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Engineering Division

Sutter Basin Pilot Feasibility Report - Environmental Impact Report / Supplemental Environmental Impacts Statement

Butte and Sutter Counties, California

GEOTECHNICAL APPENDIX

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1 INTRODUCTION

This report presents the results of the geotechnical engineering evaluation of the levees associated with the Sutter Basin Feasibility Study (SBFS). The purpose of this report is to document the geotechnical existing condition of the levees within the study area and to provide geotechnical information in support of the final array of alternatives in the Feasibility Study.

The SBFS is a Pilot Study for the Corps of Engineers to further develop Planning concepts, methodologies, and processes. This report relies heavily on the massive volume of data gathered by the California Department of Water Resources (DWR) Urban Levee Evaluation (ULE) study described in Section 1.3.2, and (for the Feather River levees) supplemented by the Sutter-Butte Flood Control Agencies (SBFCA) Feather River West Levee (FRWL) Improvement Project described in Section 1.3.3. The reports prepared for the ULE and FRWL Improvement Project are included as Enclosure M of this report, but are provided as separate files for ease of use by reviewers.

1.1 Project Description

The Sutter Basin system consists of a mainline levee system (hereafter referred to as the Sutter Basin Levee System or SBLS) surrounding the communities of Yuba City, Live Oak, Gridley, Biggs and other smaller towns in Sutter and Butte Counties, California. There are several Local Maintenance Authority (LMA) entities: Levee District (LD) 1 of Sutter County, LD 9 of Sutter County, and California Department of Water Resources (DWR) Maintenance Areas (MA) (3, 7, 13, 16, Wadsworth Canal, and Sutter Bypass). These entities maintain all levees within the study area. Other than the high ground at the Sutter Buttes and where the upstream end of the Feather River ties into the southern dike of the Thermalito Afterbay Dam, these levees completely surround and protect the interior area. Plate 1 shows the SBLS and the nearby surrounding area. The levee segments in the study area are shown on Plate 1 and are listed below:

- **Wadsworth Canal Left Levee:** Left levee (on the south bank) of the Wadsworth Canal from Project Levee Mile (PLM) 0.00 at the confluence with the Sutter Bypass upstream to PLM 4.66 at the East Interceptor Canal

- **Sutter Bypass:** Left levee (on the east bank) of the Sutter Bypass from the confluence with the Wadsworth Canal at PLM 4.40 downstream to PLM 22.11 at the confluence with the Feather River

- **Feather River – MA 3:** Right levee (on the west bank) of the Feather River from PLM 0.00 at the Sutter Bypass confluence upstream to PLM 5.19 at the downstream boundary of the LD 1 segment

- **Feather River – LD 1:** Right levee (on the west bank) of the Feather River from PLM 0.00 at the boundary of MA 3 upstream to PLM 16.65 at the downstream boundary of the LD 9 segment

- **Feather River – LD 9:** Right levee (on the west bank) of the Feather River from PLM 0.00 at the LD 1 boundary upstream to PLM 6.24 at the downstream boundary of the MA16 segment

- **Feather River – MA 16:** Right levee (on the west bank) of the Feather River from PLM 0.00 at the LD 9 boundary upstream to PLM 4.09 at the downstream boundary of the MA 7 segment

- **Feather River – MA 7**: Right levee (on the west bank) of the Feather River from PLM 0.00 at the MA 16 boundary upstream to PLM 12.07 at the downstream boundary of the Hamilton Bend segment

- **Feather River – Hamilton Bend Area**: Right levee (on the west bank) of the Feather River from PLM 0.00 at the MA 7 boundary upstream to PLM 1.20 at the Thermalito Afterbay outlet channel.

- **Cherokee Canal – MA 13:** Left levee (on the south bank) of the Cherokee Canal from PLM 9.90 at the Southern Pacific Railroad bridge upstream to PLM 6.10 at the Western Canal crossing (this partial segment is not part of the ULE program)

At the request of the local sponsor, a second levee system was added to the project. The area within this levee system is referred to as “the Sutter triangle” because the protected area is roughly triangular-shaped between two levee segments and the high ground of the Sutter Buttes and it includes the small town of Sutter. This system is also shown on Plate 1. This levee system is not part of the ULE program. The levee segments of this system are:

- **Wadsworth Canal Right Levee:** Right levee (on the north bank) of the Wadsworth Canal from PLM 0.00 at the Sutter Bypass confluence upstream to PLM 4.66 at the West Interceptor Canal

- **Sutter BypassUpstream of Wadsworth Canal:** Left levee (on the east bank) of the Sutter Bypass from the high ground at the Sutter Buttes at PLM 0.00 downstream to the confluence of the Wadsworth Canal at PLM 4.31

Photos of the levee segments covered under the ULE program are in Enclosure A of this Geotechnical Appendix.

1.2 General Levee Descriptions

This section provides a general description of the geometry, soil conditions, performance history, and modifications/improvements of each levee segment. This section is a summary of the more detailed descriptions found in chapter 2 of the ULE Phase 1 Preliminary Geotechnical Evaluation Report (P1GER) for levee segments covered under the ULE program, Appendix C of the DWR Non-Urban Levee (NULE) Geotechnical Assessment Report (GAR) for the Cherokee Canal, and Appendix E of the NULE GAR report for the Sutter triangle levees. Explorations conducted for the ULE program are presented in detail in the ULE Phase 1 Geotechnical Data Report (P1GDR) and Supplemental Geotechnical Data Report (SGDR). Explorations conducted for the Feather River West Levee (FRWL) Improvement project are presented in detail in the Geotechnical Data Report (GDR). Tables summarizing the performance history and modifications/improvements of the SBLS from Periodic Inspection Report No. 1 (2010) are in Enclosure B of this Geotechnical Appendix.

1.2.1 Wadsworth Canal Left Levee

1.2.1.1 General Description

The Wadsworth Canal and its associated levees were constructed by the State of California in 1924. The levees were constructed primarily with soil excavated from the canal. The canal was deepened and the levees were enlarged by the Corps of Engineers in 1942. The left levee of the Wadsworth Canal extends from PLM 0.00 at the confluence with the Sutter Bypass to the northeast (upstream) to PLM 4.66 at the East Interceptor Canal. The levee crest elevation varies between 60 feet NAVD at the downstream end to 65 feet NAVD at the upstream end. The levee height varies between 6 feet at the upstream end and 26 feet at the downstream end. The crest width varies between 12 feet at the upstream end and 27 feet at the downstream end. The waterside slope varies between 3H:1V and 3.5H:1V. The landside slope varies between 2H:1V and 2.5H:1V. There is a relatively flat bench 10 to 35 feet wide between the waterside levee toe and the excavated canal sideslopes. There are a few houses and outbuildings near the landside levee toe over the downstream half-mile of the levee segment.

1.2.1.2 Soil Conditions

The levee soils consist of interbedded lean clay, fat clay, sand, and silty sand. Sand and silty sand are the dominant soils over the downstream 1.4 miles of the levee segment. Clay soils dominate in the upstream 3.3 miles of the levee. The levee is founded on Basin deposits, generally 4 to 9 feet thick, consisting mostly of lean and fat clay with occasional lenses of silt and sand. The Modesto Formation underlies the Basin deposits. The upper contact of the Modesto Formation is characterized by very stiff to hard clays, called “hardpan” locally. Below the hardpan, the Modesto Formation consists of silt, lean clay, and fat clay, with 1 to 9 foot thick layers of sand and silty sand.

1.2.1.3 Performance History and Levee Improvements

Excessive through-levee seepage, underseepage, and boils occurred between PLM 0.00 and 1.25 during the 1986 and 1997 flood events. The Corps constructed a soil-cement-bentonite cutoff wall between PLM 0.00 and 0.57 in 2008. The depth of the cutoff wall varied between 42 and 63 feet.

1.2.2 Sutter Bypass Left Levee (downstream of Wadsworth Canal)

1.2.2.1 General Description

The Sutter Bypass was constructed in 1924 by the State of California to convey flood water diverted from the Sacramento River. This levee was constructed primarily with soil excavated from the bypass channel. This segment was raised and enlarged by the Corps in 1941-1942. The levee extends from the confluence with the Wadsworth Canal left bank levee at PLM 4.4 to the south (downstream) along the left bank of the Sutter Bypass to the confluence with the Feather River right bank levee at PLM 22.12. The levee crest elevation varies from 52 feet NAVD at the downstream end to 60 feet NAVD at the upstream end. The levee height varies between 14 and 22 feet with an average height of 19 feet. The crest width varies between 17 and 30 feet. The waterside slope varies between 3H:1V and 4H:1V and the landside slope varies between 2.7H:1V and 4H:1V. There is a 1-foot high, 50-foot wide berm at the landside levee toe, with a drainage ditch located at the toe of the berm over most of this segment. There are a few houses and outbuildings located near the landside levee toe. There is a small bench with an access road between the waterside levee toe and the sideslope of the excavated channel.

1.2.2.2 Soil Conditions

The levee soils consist mostly of lean and fat clays with occasional lenses of silt, sand, and silty sand up to 4 feet thick. Subsurface soil conditions are variable over the Bypass alignment, due to the geomorphology of the levee alignment cutting across numerous historic small drainage channels at approximately 90 degree angles. The foundation consists of a clay blanket 10-60 feet thick, with the layer thickness generally lower towards the downstream end of the segment. A portion of the clay blanket is cemented at some locations, locally called “hardpan”. There are pockets of sand and silty sand within the clay blanket, varying between 4 and 20 feet thick. The top of some of these pockets is 6 feet below the top of the impervious blanket layer. A widespread sand, silty sand, and gravel layer is underneath the clay blanket.

1.2.2.3 Performance History and Levee Improvements

The landside toe area of this levee segment was under water during the 1955 flood due to the break in the Feather River levee near Shanghai Bend. Performance history and levee improvements during subsequent floods are detailed below.

- During the 1958 flood numerous types of levee distress were observed between PLM 5.4 and 13 (McClatchy Road to Gilsizer Slough). Ground heaving with mud flowing from the center of the mounds, accompanied by longitudinal cracking and movement towards the drainage ditch was observed along the landside toe berm and sand boils were observed in the drainage ditch. Longitudinal cracking along the landside levee slope about 4 feet below the crest was observed at the downstream portion of this reach. During the flood, sandbag rings were constructed at the five worst locations. After the flood, the Corps of Engineers constructed a 2-foot wide, 15-foot deep toe drain trench.
- During the 1986 flood, wavewash and county maintenance equipment induced erosion at the waterside toe and halfway up the waterside slope between PLM 20 and 22.37. This erosion was repaired by the Corps of Engineers under a PL84-99 action.
- During the 1986 flood, seepage and boils in the landside drainage ditch occurred between PLM 12.7 and 14.6 (Gilsizer Slough to Everglade Road). The Corps of Engineers constructed a pervious toe drain and overlying stability berm at this site under Sacramento River Flood Control Project Phase II Levee Reconstruction, Site 12.
- Heavy seepage and soil heaving was observed between PLM 4.4 and 5.4 (Wadsworth Canal to McClatchy Road) during the 1997 flood. After the flood the Corps of Engineers constructed a toe drain trench and berm at this location under a PL84-99 rehabilitation action.
- Seepage occurred at PLM 17.6 during the 1997 flood. The Corps of Engineers constructed a 2-foot wide, 5-foot deep pervious toe trench with an overlying stability berm at this site in 2001 under a PL 84-99 rehabilitation action.
- During the 1997 flood seepage, boils, and a sinkhole were observed in a “berm” (an abandoned railroad embankment) on the landside of the levee between PLM 21.88 and 22.07 (Feather River confluence to 1,000 feet upstream). The Corps of Engineers constructed a pervious vertical drain in the berm in 2001 under a PL84-99 rehabilitation action.

1.2.3 Feather River – MA 3

1.2.3.1 General Description

The Feather River is a meandering river that flows from the Sierra Nevada Mountains down to the confluence with the Sacramento River. The MA 3 levee segment goes along the right bank of the Feather River from PLM 0.00 at the Sutter Bypass left bank levee to the north (upstream) to PLM 5.19. This levee segment was originally constructed by local residents, date unknown. The Corps enlarged the portion of the segment downstream of the Highway 99 bridge in 1942. The Corps of Engineers also relocated a landside drainage ditch 20 to 50 feet away from the toe over the upstream (northern) half of the segment in 1962. The ditch is 5 feet wide at the bottom and 6 feet deep, with 1.5H:2V sideslopes. The levee crest elevation varies between 52 feet NAVD at the downstream end to 66 feet about half a mile downstream of the upstream end of the segment. The levee height varies between 18 and 26 feet, with an average height of 22 feet. The crest width varies between 20 and 30 feet. The waterside slope varies between 1.6H:1V and

2.5H:1V. The landside slope varies between 1.5H:1V and 3H:1V. There are a few outbuildings near the landside levee toe. The waterside bench between the levee toe and the riverbank varies between 300 and 3,700 feet wide. There are waterside borrow pits near the levee toe at some locations.

1.2.3.2 Soil Conditions

The levee soils consist mostly of alternating layers of silty sand and silt, with lesser amounts of lean clay and sandy clay. The foundation consists of a sandy clay/clay/sandy silt blanket 1 to 50 feet thick. In general, the blanket layer thickness decreases moving upstream along the segment. There is no hardpan within the blanket layer. The underlying pervious layer consists of sand, silty sand, and gravel.

1.2.3.3 Performance History and Levee Improvements

The performance history and levee improvements on this segment are listed below.

- A breach was reported between approximate PLM 3.64 and 3.70, date unknown (pre-1955).
- The levee was deliberately cut and reconstructed between PLM 0.72 and 0.82 to drain floodwater from the levee breach near Shanghai Bend after the 1955 flood.
- After the 1986 flood, a “sinkhole” was discovered at the waterside levee toe at PLM 1.2. The sinkhole was 30 feet long, 10 feet wide, and 10 feet deep. No distress was observed on the landside during the flood at this location. The sinkhole was filled with gravel.
- Heavy seepage, mostly running clear, was observed over much of the segment during the 1986 and 1997 floods. Boils were observed in a few isolated locations, primarily in the bottom of the drainage ditch along the upstream half of the segment. After the 1997 flood, pervious toe drains with overlying stability berms were constructed by the Corps of Engineers between PLM 2.28 and 2.43 (Sacramento River Flood Control Project Phase II Levee Reconstruction, Site 11) and between PLM 3.46 and 3.83(PL84-99 rehabilitation).
- During the 1997 flood heavy seepage into the drainage ditch caused sloughing of soil into the ditch between PLM 3.76 and 4.45. During the flood, the ditch was filled with sandbags placed on plastic sheeting. After the flood the ditch was converted to a pervious toe drain.
- Erosion of the waterside levee slope occurred at two locations during the 1997 flood and one location during the 2006 flood. These locations were repaired after the floods.

1.2.4 Feather River – LD 1

1.2.4.1 General Description

The LD 1 segment of the Feather River goes along the right bank of the Feather River from PLM 0.00 at the upstream end of the MA 3 segment to the north (upstream) to PLM16.65 at the downstream end of the LD 9 segment. The city of Yuba City is adjacent to the upstream 6 miles of this segment. This levee segment was originally constructed by local residents, date unknown. The Corps raised and enlarged the segment in the late 1930's. The crest elevation varies between 62 feet NAVD at the downstream end and 88 feet NAVD about 200 feet downstream of the upstream end of the segment. The levee height varies between 19 and 25 feet, with an average height of 22 feet. The crest width varies between 15 and 22 feet. The waterside slope varies between 2H:1V and 3.5H:1V. The landside slope varies between 1.8H:1V and 3.1H:1V. There are waterside borrow pits near the levee in some locations. Within Yuba City buildings, swimming pools, and retaining walls have been built at and near the landside levee toe and, in some cases, cutting into the landside levee slope. The waterside bench between the levee toe and the riverbank varies from about 30 to 4,500 feet wide.

1.2.4.2 Soil Conditions

The levee soils consist of sandy silt, sandy clay, and clay with occasional zones of silty sand downstream of Star Bend (PLM 0.00 to 5.7) and sand, silty sand, and clayey sand with some zones of sandy silt and sandy clay upstream of Star Bend. The foundation soils are highly variable and consist of a clay, sandy clay and sandy silt blanket between 2 and 62 feet in thickness. Occasional, discontinuous zones of the blanket are cemented into hardpan. The blanket layer overlies a sand and gravel pervious layer that is up to 45 feet thick.

1.2.4.3 Performance History and Levee Improvements

The performance history and levee improvements within this segment are listed below.

- The levee breached just upstream of Shanghai Bend in the southern part of Yuba City (approximate PLM 11.0 to 12.4) in 1909, 1911, and 1955. The 1955 breach resulted in 38 fatalities. The Corps reconstructed the levee to the landside of its original alignment and installed relief wells in 1955-1957. Boils in this area required sandbagging during the 1986 flood. The City of Yuba City installed additional relief wells between the old relief wells in the southern portion of the relief well area in 1991 to reduce seepage into a new housing development. Floodfighting was required for boils in the northern portion of the relief well area during the 1997 flood. The Corps of Engineers installed new relief wells between the original relief wells in the northern portion of the relief well area in 2000.
- Seepage and boils occurred at Shanghai Bend during the 1986 and 1997 floods. The levee alignment had a horseshoe bend at this location. The Corps of Engineers constructed a setback levee with a 25-foot deep cutoff wall through the foundation after the 1997 flood under a PL84-99 action. The setback levee cut off the horseshoe bend.
- During the 1986 flood the landside levee slope became saturated and bulged within downtown Yuba City (approximate PLM 14.00 to 15.5) , and water flowed through cracks in pavement and floor slabs near the levee toe. Yuba City constructed an emergency berm during the flood. A permanent stability berm was constructed after the flood by LD 1. After the 1997 flood the Corps constructed a cutoff wall 40 to 55 feet

deep between PLM 12.76 and 14.54. The southern portion of the wall was constructed under the Marysville/Yuba City Levee Reconstruction Project. The northern portion of the wall was constructed under a PL84-99 rehabilitation action.

- Waterside levee erosion occurred near the Fifth Street Bridge in Yuba City during the 1986 and 1997 floods (PLM 14.27 to 14.57). The erosion was repaired with riprap.
- Floodfighting was required for seepage and boils just north of Star Bend during the 1986 and 1997 floods (PLM 4.56 to 5.42). The Corps of Engineers installed relief wells after the 1997 flood. Light seepage was noted between the upstream 3 relief wells during the 2006 flood.
- LD 1 constructed a setback levee with a 40 to 65-foot deep soil-bentonite cutoff wall through the foundation in 2008 at Star Bend (PLM 3.76 to 4.58). The setback cut off a 90-degree bend in the levee alignment.
- During the 1997 flood the waterside levee slope became unstable between PLM 5.55 to 5.66. The instability was believed to be due to a rapid drawdown situation after the levee on the opposite bank of the Feather River breached. The Corps repaired the slope after the flood under a PL 84-99 rehabilitation action.
- At PLM 1.5, a crack in the levee occurred during the 1986 flood and boils occurred during the 1997 flood. The Corps constructed a stability berm after the 1997 flood under a PL84-99 rehabilitation action after the 1997 flood.

1.2.5 Feather River – LD 9

1.2.5.1 General Description

The LD 9 segment goes along the right bank of the Feather River from PLM 0.00 at the upstream end of the LD 1 segment to the north (upstream) to PLM 6.24 at the downstream end of the MA 16 segment. An active railroad embankment crosses the levee alignment at the LD1/LD 9 boundary. The railroad embankment is about 4 feet lower than the levee; this opening is sandbagged during flood events. This levee segment was originally constructed by local residents, date unknown. Portions of this segment were raised and enlarged by the Corps in 1933 and 1944. The levee crest elevation varies between 83 feet NAVD at the downstream end and 91 feet NAVD near the upstream end of the segment. The levee height varies between 11 and 21 feet, with an average height of 19 feet. The crest width varies between 16 and 25 feet. The waterside slope varies between 1.9H:1V and 3H:1V. The landside slope varies between 1.4H:1V and 2.6H:1V. An irrigation canal (Sutter Butte Canal) is adjacent to the landside levee toe over a portion of this segment. The Sutter Butte Canal is about 30 feet wide at the bottom and varies between 5 and 8 feet deep. Smaller, localized drainage ditches are at the landside levee toe in some areas where the Sutter Butte Canal is not adjacent to the toe. There are also a few houses and outbuildings near the landside levee toe. Width of the waterside bench between the levee toe and the riverbank varies between 5 and 3,800 feet.

1.2.5.2 Soil Conditions

The levee soils consist of silt, sandy silt, sandy lean clay with occasional silty sand. The clay soils predominate at the downstream end of the segment and the silty and sandy soils predominate towards the upstream end of the segment. The foundation soils consist of a sandy clay/sandy silt blanket of variable thickness (average thickness 12 feet), sometimes cemented into a hardpan, overlying a sand/silty sand pervious layer. The pervious layer has some gravel lenses in the downstream half of the segment.

1.2.5.3 Performance History and Levee Improvements

The performance history and levee improvements on this segment are listed below.

- Boils were observed at PLM 2.12 and 3.60 during the 1955 flood. The landside levee slope at those locations was rebuilt with a gravel filter.
- Boils were observed between PLM 4.63 and 4.91 in 1957.
- Several waterside "sinkholes" appeared near the levee toe between PLM 0.1 and 0.46 in 1966. The sinkholes were attributed to improper removal of tree stumps and they were filled in.
- A "sinkhole" was observed at PLM 0.81 in 1986; documentation does not show the location (waterside or landside).
- Seepage and boils were observed in an irrigation ditch at PLMs 3.75 and 5.13 during the 1986 flood.
- Trench drains were placed at the landside levee toe between PLM 3.0 and 3.83 and between PLM 4.33 and 4.9 by LD 9 in 1992. The trenches were 4-5 feet deep and 2 feet wide and consisted of a geotextile lining around drain rock, with a perforated PVC pipe near the bottom of the trench.
- Seepage and boils were observed at several locations during the 1997 flood. At most of these locations, LD 9 personnel report that seepage quantities were "significantly higher" than during previous flood events. These locations included PLM 0.0 and 1.3; PLM 2.80 and 3.07; and PLM 4.50 and 5.30. There is no evidence of post-flood repairs at those locations. Boils also occurred between PLM 2.43 and 2.59; the Corps of Engineers constructed a toe drain with a concrete V-ditch collector at this location in 1998.
- Heavy seepage into the Sutter Butte Canal caused landside levee toe sloughing into the canal and a longitudinal crack in the levee slope between PLM 5.44 and 5.65 during the 1997 flood. The canal was filled with gravel during the flood. The canal was relocated into a pipeline away from the levee toe after the flood.

1.2.6 Feather River – MA 16

1.2.6.1 General Description

The MA 16 segment goes along the right bank of the Feather River from PLM 0.00 at the upstream end of the LD 9 segment to the north (upstream) to PLM 4.09 at the downstream end of the MA 7 segment. This levee was originally constructed by local residents, date unknown. Portions of this segment were raised by the Corps in 1933 and 1944. The levee crest elevation varies between 91 feet NAVD at the downstream end to 96 feet NAVD at the upstream end. The levee height varies between 7 and 14 feet, with an average height of 10 feet. The crest width varies between 15 and 25 feet. The waterside slope varies between 1.9H:1V and 3.2H:1V. The landside slope varies between 1.3H:1V and 3H:1V. The Sutter Butte Canal is adjacent to the landside levee toe over a portion of this segment. The Canal is about 30 feet wide at the bottom and the depth varies between 5 and 8 feet. There are a few houses and outbuildings near the landside levee toe. The waterside bench between the levee toe and the riverbank varies between 30 and 3,100 feet wide.

1.2.6.2 Soil Conditions

The levee soils consist mostly of sandy silt, with some zones of sandy clay. The foundation consists of a clay/sandy silt blanket, at some locations cemented into hardpan, between 0 and 50 feet thick (average thickness about 20 feet) overlying a pervious sand layer. The pervious layer contains gravel in the upstream half of the segment.

1.2.6.3 Performance History and Levee Improvements

There is no documented history of performance problems and subsequent levee modifications to this segment.

1.2.7 Feather River – MA 7

1.2.7.1 General Description

The MA 7 segment goes along the right bank of the Feather River from PLM 0.00 at the upstream end of the MA 16 segment to the north (upstream) to PLM 12.07 at the downstream end of the Hamilton Bend segment. This levee was originally constructed by local residents, date unknown. The Corps reconstructed this levee in 1954. The levee crest elevation varies between 96 feet NAVD at the downstream end and 135 feet NAVD at the upstream end. The levee height varies between 5 and 22 feet, with an average height of 15 feet. The crest width varies between 15 and 25 feet. The waterside slope varies between 1.9H:1V and 3.2H:1V. The landside slope varies between 1.3H:1V and 3H:1V. The Sutter Butte Canal is adjacent to the landside levee toe over a portion of this segment. The Canal has a bottom width of about 30 feet and is between 5 and 8 feet deep. There are a few houses and outbuildings near the landside levee toe. The waterside bench between the levee toe and the riverbank varies between 5 and 4,800 feet wide. Dredge tailings, consisting primarily of cobbles and gravel, have been placed on the waterside bench over the upstream 4 miles of the segment.

1.2.7.2 Soil Conditions

The levee soils consist mostly of sandy silt, with some zones of sandy clay and occasional lenses of sand. The foundation consists of a blanket of clay/sandy in the southern portion of the segment and silt/silty sand in the northern portion of the segment. Thickness of the blanket varies between 0 and greater than 80 feet; the average thickness is about 15 feet, and in general the thickness decreases moving upstream along the segment. The pervious layer consists of sand and gravel. The pervious layer is almost entirely gravel upstream of PLM 3.2.

1.2.7.3 Performance History and Levee Improvements

The performance history and levee improvements/modifications to this segment are listed below.

- During the 1955 flood, waterside slope erosion caused the west span of the Gridley bridge across the Feather River to collapse, although the levee did not breach due to floodfighting efforts. The levee and the bridge were rebuilt by local residents after the flood. The bridge was replaced by a new bridge about 450 feet downstream of the old bridge in the mid-1960's.
- Boils were observed between PLM 0.1 and 0.9 during the 1955 flood event.
- During the 1986 flood, seepage and boils were observed between PLM 2.68 and 2.82. The Corps constructed a 50-foot deep cutoff wall at this location (plans not found).
- Seepage and boils were also observed between PLM 9.7 and 9.8 during the 1986 flood.
- The Corps raised the levee about 2 feet and reworked the landside slope between PLM 9.9 and 10.4 in 1998 under the Sacramento River Flood Protection Project Phase II System Evaluation.
- During the summer dry season, when the Feather River is low and the Sutter Butte Canal is full, seepage flows from the canal and exits on the waterside of the levee between PLM 0.0 and 1.0.

1.2.8 Feather River – Hamilton Bend

1.2.8.1 General Description

The Hamilton Bend segment goes along the right bank of the Feather River from PLM 0.00 at the upstream end of the MA 7 segment to the north (upstream) to PLM 1.20 at the Thermalito Afterbay outlet channel. This levee was originally constructed by the Corps of Engineers in 1947. The Corps raised the levee in the mid-1950's and widened the downstream mile of the levee in 1963. The Sutter Butte Canal crosses the levee alignment at PLM 1.05-1.06. A concrete headgate structure was built across the canal alignment. The headgate structure is 36 feet tall, 50 feet long, and 13.5 feet wide. The headgate structure was abandoned after construction of the upstream Oroville Dam in 1968. The levee crest elevation varies between 134 feet NAVD at the downstream end and 139 feet NAVD at the upstream end. The levee height varies between 3 and 24 feet, with an average height of 14 feet. The crest width is 15-20 feet upstream of the headgate structure and 60-70 feet downstream of the headgate structure. The waterside slope varies between 2H:1V and 2.5H:1V. The landside slope varies between 1.5H:1V and 3H:1V. There is one small outbuilding near the landside levee toe. The waterside bench between the levee toe and the riverbank varies between 50 and 1,100 feet wide. The downstream 0.8 miles of the segment was built through dredge tailings piles. The dredge tailings consist of silty sand, gravel, and cobbles and are higher than the levee crest elevation at some locations.

1.2.8.2 Soil Conditions

The levee is constructed of clay upstream of the headgate structure and silty sand, gravel, and cobbles (dredge tailings) downstream of the headgate structure. There is a thin clay blanket underlying less than half of this levee segment. The pervious layer consists of silty sand, gravel, and cobbles (dredge tailings) about 80 feet thick.

1.2.8.3 Performance History and Levee Improvements

This segment experienced seepage and boils downstream of the headgate structure during the 1955 flood, which was floodfought by local residents. This portion of the levee was widened by the Corps of Engineers in 1963. The Sutter Butte Canal Headgate Structure was also floodfought for overtopping during the 1955 flood; the crest elevation of the structure is lower than the crest elevation of the adjacent levee, and water was reportedly 16 inches below the crest of the structure.

1.2.9 Cherokee Canal (MA 13)

1.2.9.1 General Description

The Cherokee Canal and its associated levees were constructed by the Corps of Engineers in the late 1950's. The Cherokee Canal is located in the northwest portion of the project area. The Canal discharges water into the Butte Sink, a low-lying area between the Sacramento River and the Sutter Buttes. The entire canal is 23.1 miles long; this Feasibility Study only includes the left bank levee from PLM 9.90 at the Southern Pacific Railroad bridge to the northeast (upstream) to PLM 6.10 at the Western Canal confluence. This portion of the levee would flood the town of Biggs if it breached or overtopped. The levee height is 6-10 feet and the crest width is 10-20 feet. The waterside slope varies between 3H:1V and 3.5H:1V and the landside slope varies between 2.5H:1V and 3H:1V. An irrigation ditch is present at the landside toe.

1.2.9.2 Soil Conditions

The levee is constructed of lean and fat clay, silt, and elastic silt. The foundation soils consist of a silt and sandy silt blanket between 3 and 19 feet thick, overlying a pervious layer of silty sand, clayey sand, and clean sand. Where the pervious layer consists of clean sand, it generally contains silt lenses that are 2-4 feet thick.

1.2.9.3 Performance History and Levee Improvements

This section of levee overtopped during the 1986 flood due to debris buildup at the railroad bridge. There is no documentation of any other performance problems during floods or improvements/modifications to this portion of the segment.

1.2.10 Wadsworth Canal Right Levee

1.2.10.1 General Description

This levee segment extends from PLM 0.00 at the confluence with the right bank levee of the Sutter Bypass to the northeast (upstream) along the right bank of the Wadsworth Canal to PLM 4.66 at the West Interceptor Canal. The levee height varies between 20 feet at the downstream end to 5 feet at the upstream end. The crest width is 10-20 feet. The waterside slope varies between 3H:1V and 3.5H:1V. The landside slope varies between 2H:1V and 2.5H:1V. A small drainage canal is located at the landside levee toe over most of this segment. The DWR Sutter Maintenance Yard is located at the landside of the levee immediately south of Highway 20.

1.2.10.2 Soil Conditions

There are no known explorations in this levee segment. Since the canal is fairly small (about 300 feet from levee crest centerline to levee crest centerline), it is anticipated that soil conditions would be similar to the left bank levee of the Wadsworth Canal, discussed in section 1.2.1.2 of this report.

1.2.10.3 Performance History and Levee Improvements

This segment has experienced seepage and boils between PLM 0.00 and 2.4 during floods, and waterside bank erosion encroaching into the levee toe has been reported at PLM 3.2. No modifications or improvements have been constructed on this segment.

1.2.11 Sutter Bypass Upstream of Wadsworth Canal

1.2.11.1 General Description

This levee segment extends along the right bank of the Sutter Bypass from PLM 0.00 at high ground at the Sutter Buttes to the southeast (downstream) to the confluence of the right bank levee of the Wadsworth Canal at PLM 4.31. The levee height varies between 15 feet at the upstream end and 23 feet at the downstream end. The crest width is 20 feet. The waterside slope varies between 3.5H:1V and 4H:1V and the landside slope varies between 2.5H:1V to 3H:1V. A project pump plant at PLM 2.7 pumps interior drainage water over the levee into the Bypass. There are some outbuildings near the landside toe of the levee. There is also a drainage canal on the landside of the levee. The canal is located 15 to 50 feet from the landside toe and is about 5 feet deep and 12 feet wide at the bottom.

1.2.11.2 Soil Conditions

There are no existing explorations on this levee segment.

1.2.11.3 Performance History and Levee Improvements

Moderate seepage, not carrying material, was reported at PLM 2.68 during the 1997 flood; no remedial action was undertaken. Also during the 1997 flood, heavy seepage and soil heaving occurred between PLM 3.7 to 4.3. A pervious toe drain and overlying stability berm were constructed by the Corps of Engineers under a PL84-99 rehabilitation action.

1.3 Previous Geotechnical Studies

Over the years, many geotechnical studies have been performed on the Sutter Basin, particularly on the Feather River levee in the vicinity of Yuba City. The major geotechnical engineering studies for the Sutter Basin area in recent years include:

- The original Sutter Basin Feasibility Study without project condition report (i.e. F3 report) completed in 2004.
- The State of California Department of Water Resources (DWR) Urban Levee Evaluation (ULE), ongoing.
- The Sutter-Butte Flood Control Agency (SBFCA) Feather River West Levee (FRWL) Improvement Project, ongoing.
- The State of California DWR Non-Urban Levee Evaluation (NULE), ongoing.

1.3.1 Initial Sutter Basin Report

The initial Sutter Basin report had a different study area, and included all levees within Sutter County, as identified in Sutter County, California Feasibility Study Feasibility Scoping Meeting (F3 Milestone) Report (2004). That study area did not include any complete levee systems; the levees studied stopped at the Sutter County boundaries. That assemblage of segments and partial segments did not fully protect any one area completely, and relied on levees that were not located within Sutter County. The initial study did not include any new explorations and it was done before the DWR ULE program began, so it relied on a limited amount of historical explorations and reports.

1.3.2 DWR Urban Levee Evaluation (ULE)

DWR initiated the ULE study in 2007. The ULE study, which is ongoing, is a major geotechnical evaluation of all the levee systems protecting urban areas within the Central Valley of California. The definition of the Urban Levees for the DWR project is a levee system protecting more than a population of 10,000. The Sutter Basin part of the ULE program includes most of the same levee segments as this project. The levees along the right bank of the

Cherokee Canal, the right bank of the Wadsworth Canal, and the left bank of the Sutter Bypass upstream of Wadsworth Canal were not included in the ULE program because they do not protect a large enough population. The DWR ULE study was partially funded by this project as in-kind services for geotechnical explorations and geotechnical deterministic evaluations.

1.3.3 SBFCA Feather River West Levee (FRWL) Improvement Project

SBFCA is a consortium of Sutter and Butte Counties, the Cities of Yuba City, Live Oak, Gridley, and Biggs, and Levee Districts 1 and 9 of Sutter County. The agency was formed in 2007 to finance and construct regional levee improvements. The FRWL Improvement Project's goal is to improve the 44 miles of the right bank levee of the Feather River from the Thermalito Afterbay outlet to the confluence with the Sutter Bypass under a Section 408 permit. The FRWL Improvement Project design team works closely with the DWR ULE team and much of the geotechnical design of the FRWL Improvement project is based on the geomorphic study and the explorations and laboratory testing done for the ULE project. The FRWL project did additional explorations and laboratory testing to fill in data gaps at specific locations and to have the information necessary for a design project. The additional data, including subsurface investigation and deterministic analyses provided by the study for the FRWL Improvement Project was also used for development of this Feasibility Study geotechnical evaluation. Currently the FRWL Improvement Project is at 90 % design for what has been designated the Contract C area (between Shanghai Bend and Live Oak, LD 1 PLM 11.1 to MA16 PLM 2.87). SBFCA plans to construct that contract in 2013. The areas designated as Contract A1 (Laurel Avenue to Star Bend, MA3 PLM 3.64 to LD 1 PLM 3.76), Contract B (Star Bend to Shanghai Bend, LD1 PLM 4.56 to LD 1 PLM 10.62), and Contract D (Live Oak to Thermalito Afterbay, MA 7 PLM 0.00 to Hamilton Bend PLM 1.20) are at the 65% design level.

1.3.4 DWR Non-Urban Levee Evaluation (NULE)

The NULE study, which is ongoing, is a major geotechnical evaluation of all the levee systems protecting non-urban areas within the Central Valley of California. The levees along the Cherokee Canal, the right bank levee of the Wadsworth Canal, and the left bank levee of the Sutter Bypass upstream of the Wadsworth Canal (between PLM 0.00 and 4.31) are included in the DWR NULE study. The DWR NULE study is just beginning; a Geotechnical Assessment Report (GAR), a collection of existing exploration, geomorphology, and levee performance information was prepared in April 2011, but no new explorations have currently taken place.

1.4 Use of DWR ULE/NULE and SBFCA FRWL Improvement Project Studies

This geotechnical report uses the ULE/NULE and SBFCA FRWL Improvement Project reports as the source for the majority of data upon which this geotechnical evaluation is based. The purpose of this report is not to duplicate the efforts of the ULE/SBFCA project reports, especially due to the massive volume of data gathered by those projects and the restraints of the Feasibility Pilot Study process. This report is an independent evaluation (analysis and conclusions); though in many cases the evaluations of the ULE/SBFCA projects were used to facilitate this report. The geotechnical explorations, the geomorphic study, and the deterministic analyses developed in the ULE/SBFCA projects were used as the basis for the geotechnical risk-based analysis in this Feasibility Study. The design plans and cost estimate for the SBFCA project were used to prepare the design and cost estimates for the alternatives carried forward in this study.

2 PROJECT DATUM AND STATIONING

The project study used existing topographical data, which was previously developed for the Sacramento/San Joaquin Rivers Comprehensive Study (Comp Study). The Comp Study topography was surveyed using the National Geodetic Vertical Datum of 1929. Comp Study elevations which were not surveyed in NAVD88 were converted to NAVD88 as discussed in section 2.2 of this report.

2.1 Horizontal Control

Index point locations are referenced in relation to the Project Levee Mile (PLM) system as provided in paragraph 1.1 and with northing and easting coordinates. All northing and eastings provided herein are referenced to the NAD83 Horizontal datum. All horizontal coordinates are projected to the California State Plane Zone II coordinate system. PLMs have been established by the Project Sponsor and the LMAs and are shown in the existing Operations and Maintenance (O&M) Manuals. The SBFCA FRWL Improvement Project has set up a stationing system along the FRWL that is used in their plans and design reports. Table 2-1 shows a correlation between PLMs and the SBFCA stationing for each levee segment on the Feather River.

Table 2-1 PLM and SBFCA Stationing

LMA	PLM	SBFCA Station
MA 3	0.00	10+00
	5.11	280+60
LD 1	0.00	280+60
	16.65	1131+00
LD 9	0.00	1131+00
	6.24	1460+40

LMA	PLM	SBFCA Station
MA 16	0.00	1460+40
	4.09	1674+30
MA 7	0.00	1674+30
	12.07	2303+50
Hamilton Bend	0.00	2303+50
	1.20	2367+00

2.2 Topographical Base

As required by ER 1110-2-8160 all elevations provided herein are referenced to the NAVD88 vertical datum at an appropriate level of accuracy. The geotechnical deterministic and risk-based analysis done for this report used topography originally developed by the Sacramento/San Joaquin Rivers Comprehensive Study (Comp Study), with the exception of Cherokee Canal. The Comp Study topography was collected in the NGVD 29 vertical datum. The Comp Study topography was converted to the NAVD 88 vertical datum by HJW GeoSpatial, Inc. (2010) following the requirements in ER 1110-2-8160 and the uncertainty in the conversion has been recognized in the study results. The report on the datum conversion is in Enclosure C of this report. The conversion between NGVD29 and NAVD88 ranges from -2.3 to -2.4 feet over the study area. The levee crest elevations at the index point locations were verified with the levee crest elevations in the National Levee Database, which was surveyed in the NAVD 88 vertical datum. The Cherokee Canal channel and levee cross-sections use LiDAR data provided by DWR.

3 PAST FLOODS AND LEVEE PERFORMANCE

Significant high water events in recent years include the Floods of 1986 and 1997. Significant seepage and large sand boils were observed along the right bank of the Feather River, left bank of the Wadsworth Canal, and Sutter Bypass levees during these two flood events. A catastrophic levee break occurred on the right bank of the Feather River near Shanghai Bend at the southern end of Yuba City during the 1955 flood event with severe economic effects and the loss of 38 lives. Moderately high water events occurred in 1995 and 2006. Light seepage and pin boils developed during these two events. Section 1.2 of this report gives a summary of the levee performance history during flood events. More detailed information may be found in chapter 2 of the ULE P1GER report for levee segments covered under the ULE program and Appendix E of the NULE GAR report for the Sutter triangle levees. Tables summarizing the performance history and constructed modifications/improvements of the SBLS from Periodic Inspection Report No. 1 (2010) are provided in Enclosure B of this report. Table 3-1 shows the peak river stage for the Feather River at Yuba City gage station. The peak stage of record for the Feather River occurred during the December 1955 Flood with a peak stage of about 78.5 feet at the Yuba City gage station.

Table 3-1 Peak River Stages at the Feather River Yuba City Gage

Event	**Peak River Stage at the Feather River Yuba City Gage (Elevation in feet, NAVD 1988)**
1986	75.3
1995	66.8
1997	77.5
1999	52.3
2000	51.8
2006	66.8

Note: Normal river level elevation is 36 to 37 feet.

4 GEOLOGY, GEOMORPHOLOGY, GROUNDWATER, AND SEISMICITY

The following discussion of regional and local geology was taken from Chapter 3 of the ULE P1GER report, with minor modifications.

4.1 Regional Geology

The levees within the SBFS are located in the central portion of the Sacramento Valley, which comprises the northern half of the Great Valley Geomorphic Province of California. The SBFS includes levees near the center of the Sacramento Valley (Wadsworth Canal and Sutter Bypass), levees near active river channels (Feather River) as well as some levees near the east margin of the valley (Feather River North). The Wadsworth Canal sits on the flanks of the Sutter Buttes, a small isolated volcanic range located in the center of the Sacramento Valley.

The Sacramento Valley lies between the northern Coast Ranges to the west and the northern Sierra Nevada to the east, and was a depositional basin throughout most of the late Mesozoic and Cenozoic. An estimated 2+ vertical miles of sediments were deposited in the Sacramento area during cyclic transgressions and regressions of a shallow sea that once inundated the valley. This thick sequence of clastic sedimentary rock units, derived from erosion of the adjoining easterly and westerly highlands from the Late Jurassic, with interspersed Tertiary volcanics, form the bedrock units now deeply buried in the mid-basin areas of the valley. These bedrock units were covered by coalescing alluvial fans during the Pliocene-Pleistocene era by the major ancestral rivers of the Sacramento Valley (Sacramento, Feather, Yuba, Bear, and American) that funneled huge volumes of sediments into the Sacramento basin. Periodically, volcaniclastic sediments and tuffs were also broadly deposited during this time. Late Pleistocene and Holocene alluvial deposits now cover the low-lying areas, consisting mainly of reworked fan and stream materials that were deposited by meandering rivers prior to the construction of the existing flood control system.

From the mid-1800’s to the early 1900’s, hydraulic mining in the Sierra Nevada introduced massive amounts of mining debris into the Feather, Yuba, and Bear Rivers. This debris backfilled river channels and raised channel elevations. Mining debris was also transported and deposited atop the more consolidated Pleistocene-to Holocene-age deposits during numerous flood events. Many (although not all) levees along the major ancestral rivers of the Sacramento Valley were deliberately built near the river channel to increase the flow velocity during flood events to “flush out” the mining debris. Much (although not all) of that debris has been flushed out, resulting in the rivers eroding the bottom and sides of their channels. Erosion of the bottom and sides of the river channel tends to worsen underseepage over time.

4.2 Local Geology

State-sponsored regional geologic mapping, at a scale of 1:250,000, of the Chico Quadrangle (Saucedo and Wagner, 1992) and Sacramento Quadrangle (Wagner, Jennings, Bedrossian, and Bortugno, 1987) indicate within the project area geologically recent (Quaternary-age) alluvial/fluvial deposits fill the floodplains of the Sacramento and Feather Rivers. These deposits consist of Holocene Alluvium and Holocene Basin Deposits, as well as late Pleistocene alluvial fan and terrace deposits of the Modesto and Riverbank Formations. Per Helley and Harwood (1985), these Quaternary deposits are variably dissected and overlain by younger Quaternary (Historical) deposits consisting of channel, floodplain, and artificial fill (levees and mine tailings). The Historical deposits date from approximately 1800 to 1937.

Cropping out adjacent to the younger fluvial deposits of the north-westerly portion of the project area are the Pleistocene volcanic rock and sediments of the Sutter Buttes. To the north and east of the younger fluvial deposits are outcrops of Plio-Pleistocene volcanic sediments informally known as the "Tuffs of Oroville." Additionally, fluvial deposits of the Pliocene Laguna Formation occur as scattered outcrops located at the base of the easterly-adjoining highlands and are the oldest geologic unit in the local project vicinity.

The generalized geologic map units in the project region are detailed below with descriptions typically summarized from Helley and Harwood (1985). The map symbol for each mapped geologic unit is noted after the formal name of the unit. Typically, each younger geologic unit is deposited into broad channels cut into the progressively older geologic units. Map symbols used on plan and profile drawings in the DWR ULE reports and SBFCA FRWL Improvement Project are slightly different since they are based on the geomorphologic mapping (see paragraph 4.3 below), which is more detailed.

- **Holocene (Historical) Artificial Fill (L, T).** Historical Artificial Fills (less than 150-years old) are culturally-emplaced deposits of varying amounts of clay, silt, sand, and gravel from local sources. These deposits include levee structures (L) and dredge tailings (T). Dredge tailings are located along the northern-most reach of the Feather River.
- **Holocene (Historical) Alluvial Deposits (Ra).** Historical Alluvial Deposits occur generally between the Feather River channel levees and on the land side. These deposits also occur in historical stream and overflow channels that transport high stage water flow across the ground surface outboard of the levees. The deposits consist of silt and sand with traces of gravel. The upper few feet of these deposits (particularly those derived from the Feather River) are anticipated to be filled with debris derived from upstream hydraulic mining activities.
- **Holocene Alluvium (Qa).** Holocene (<11,000 years before present [ybp]) alluvial deposits are present within the floodplains of the Sacramento River, Feather River, and lesser streams and water courses in the area. These sediments were deposited into and long channels incised into the older latest-Pleistocene formations of similar fluvial origin. The alluvium includes sand and gravel deposits of the active and abandoned stream

channels, as well as sands, silts, and minor gravel lenses deposited as overbank deposits during high-water stages.

- **Holocene Basin Deposits (Qb).** Basin Deposits primarily dominate in the western portion of the project area. The Wadsworth Canal and a majority of the Sutter Bypass are underlain by these deposits. Closer to the Feather River, Basin Deposits are less extensive and are mapped as scattered exposures overlying the Modesto and Riverbank formations. These fluvial sediments consist of fine-grained silt and clay, and represent the distal facies of Holocene alluvial deposits in the area.
- **Pleistocene Modesto Formation (Qm).** The youngest Pleistocene fluvial deposits in the project area are termed the Modesto Formation. The Modesto Formation is exposed at the ground surface predominantly along the landside of most of the Feather River levees and to a lesser degree along the eastern edge of the Sutter Buttes. This formation underlies the younger Basin Deposits east of Sutter Buttes and most of the Feather River channel alluvium. These deposits are recognized by distinct alluvial terraces, alluvial fans and abandoned channel ridges that lie topographically above and outboard of the Holocene alluvial deposits. The Modesto Formation generally consists of unconsolidated gravel, sand, silt and clay and can be divided into two members; an upper member (12,000 to 26,000 ybp) showing poor soil development and a lower member (30,000 to 50,000 ybp) characterized by thicker soils with a developing argillic B horizon. Hardpan may be present near the top of each member of the Modesto Formation as an indicator of the upper boundary with the overlying alluvium. On the western side of the Feather River, the Modesto Formation is a common source of fresh water, as coarse-grained deposits are thick and have good hydraulic connection.
- **Pleistocene Riverbank Formation (Qr).** The Riverbank Formation has limited exposures on the western side of the Feather River at the extreme northern and southern ends of the river. However, the Riverbank Formation is predominantly separated from the Feather River alluvial deposits by the Modesto Formation. The Riverbank Formation consists primarily of gravel, sand and silt and can typically be distinguished from the Modesto by topographic position and presence of a thick, well-developed soil horizon. The Riverbank Formation is subdivided into the Upper and Lower members, each associated with periods of geologic stability (Helley and Harwood, 1985). Hardpan may be present near the top of each member. The Riverbank formation is estimated to be 130,000 to 450,000 years old. The Riverbank Formation can be a source for fresh water in the Feather River North area, but the interbedded nature of the fine- and coarse-grained units and the varying degree of connectivity between coarse-grained beds may limit groundwater production at some locations.
- **Pleistocene Sutter Buttes Volcanic Sediments (QPs) and Rocks (QPv).** These deposits primarily occur as a peripheral topographic ring that surrounds the Sutter Buttes and are separated from the Basin Deposits of the SBFS area by the Modesto and Riverbank Formations. These deposits consist predominantly of volcaniclastic sediments and lahars and to a lesser degree andesitic rocks.

- **Plio-Pleistocene Tuffs of Oroville (QPto).** The “Tuffs of Oroville” consist of interbedded volcaniclastic deposits of gravel, sand, and tuff (volcanic ash) deposited to the south and west of the town of Oroville.

4.3 Geomorphology

A detailed geomorphology study was conducted for the Wadsworth Canal, Sutter Bypass, and Feather Rivers under the DWR ULE program. Results of that study are in Enclosure D of this report. A preliminary geomorphology study of the Cherokee Canal and the Sutter Triangle levees has been conducted under the DWR NULE program, although no reports or maps will be prepared until a more detailed study of those levees has been conducted.

4.4 Groundwater

The SBFS project area is located along the eastern boundary of the East Butte and Sutter Subbasin, which comprises part of the greater Sacramento Valley Groundwater Basin. Historical mine tailing deposits and historical to Holocene stream channel and floodplain deposits upon which the levees were constructed are highly permeable and allow for large volumes of groundwater recharge within the subbasin.

DWR maintains an internet water data library (http://www.water.ca.gov/waterdatalibrary/), which includes groundwater depth measurements from monitoring wells throughout the state. The DWR data and additional information from the Butte County Groundwater Management Plan indicate groundwater within the project area typically flows from north to south. The data also show relatively stable groundwater elevations within the last 40 to 60 years, although there is a typical seasonal fluctuation in groundwater elevation with peaks typically in the winter to spring months and drops in the summer and fall months. The winter and spring peaks indicate groundwater recharge during the precipitation season, and the summer and fall drops indicate the lack of rainfall and extraction of groundwater for domestic and irrigation uses. The DWR water data library indicates groundwater levels within the project area vary between 3 and 30 feet below ground surface. Pore-pressure dissipation tests completed during cone penetration test (CPT) advancement show groundwater levels in the Feather River south area vary between 10 and 32 feet below ground surface and groundwater levels in the Feather River north area vary between 8 and 46 feet below ground surface. Groundwater levels are expected to fluctuate due to variations in precipitation, nearby river stage, irrigation, and withdrawal.

4.5 Seismicity

The SBFS project area is located in a relatively low area of seismic risk for California. Nearby faults, such as those of the Foothills Fault System are located 10 to 20 miles east in the foothills of the Sierra Nevada; however, these faults are considered to have a small likelihood for seismic activity, and may be considered active over very long periods. A portion of this fault was active after the first filling of Oroville Dam. The Great Valley Fault System is a blind thrust fault on the western edge of the Sacramento Valley, about 34 miles from the SBFS levees. This fault system is also considered to be active.

5 GEOTECHNICAL RISK-BASED ANALYSIS METHODOLOGY

The following sections discuss the risk-based geotechnical analyses performed to evaluate the existing levees. Effects of utility penetrations, encroachments, erosion and animal activity were considered in the engineering judgment portion of the analysis. Underseepage and slope stability analyses were primarily used to develop the expected performance of each reach.

5.1 Risk Based Analysis Method

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. 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 evaluation of the existing conditions of each levee unit. 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 slope stability and underseepage piping stability. Through-levee seepage is discussed in section 5.4 of this report.

The final result of the FOSM is a reliability index, Beta (β), representing the amount of standard deviation of the performance function by which the expected value exceeds the limit equilibrium state. The limit equilibrium state for the slope stability and underseepage piping stability was defined using a factor of safety of 1.0. The standard deviation and variance of the performance function are calculated from the standard deviation and variance of the foundation and embankment parameters using the Taylor's series method based on a Taylor's series expansion of the performance function about the expected values. The partial derivatives were calculated numerically using an increment of plus and minus one standard deviation centered on the expected mean value. The variance of the performance function was obtained by summing the products of the partial derivatives of the performance function considering the variance of the corresponding parameters. For the existing condition of the levee, the probability of failure Pr(f) was expressed as a function of the river water elevation and other factors including soil strengths, permeabilities, and subsurface stratification. Reliability (R) is defined as:

$$R = \left(1 - \Pr(f)\right)$$

The combined geotechnical conditional probability of poor performance, considering the probability of poor performance due to underseepage failure, slope stability and judgment probability is

$$\Pr(f) = 1 - ((1-\Pr(f)_{us})*((1-\Pr(f)_{st}) * (1-\Pr(f)_{jd}))$$

Where: $Pr(f)$ = total probability of poor performance

$Pr(f)_{us}$ = probability of poor performance due to underseepage

$Pr(f)_{st}$ = probability of poor performance due to slope stability (in steady state condition)

$Pr(f)_{jd}$ = judgment probability of poor performance

A set of conditional-probability-of-poor performance versus floodwater-elevation graphs were developed as related to underseepage piping, stability and judgment.

The probability of geotechnical poor performance of a levee is conditional on the uncertainties associated with hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties in the @RISK model. This is accomplished, for economic purposes, through estimation of two index elevations for each levee reach within the study area. These index elevations are defined as follows:

The Probable Non-Failure Point (PNP) is the water elevation below which it is highly likely that the levee would not fail.

The Probable Failure Point (PFP) is the water elevation above which it is highly likely that the levee would fail.

The terms "highly likely that the levee would fail" is defined by the ETL as having 85% probability of occurrence. Therefore, the probability of failure at the PNP is 15% and the probability of failure at the PFP is 85%. It should be noted that only for flood damage economic studies the probability of poor performance curve at 85% should be assumed to correspond to PFP and probability of poor performance of 15% shall be assumed to correspond to PNF. These performance curves shall be not used to predict failure for studies other then flood damage economic studies.

The Probability of Poor Performance of the stability of a slope (P_{rf}) is defined as the probability that the critical failure surface could be loaded to the limit equilibrium state. This infers the slope is loaded to its maximum capacity.

The reaches to be analyzed were selected considering the soil profiles based on the available subsurface investigation and the deterministic calculations on cross sections where the geotechnical conditions were considered critical (such as thin impervious blanket, thick aquifer, soft material in the levee or top foundation layer).

5.2 Underseepage Reliability

Subsurface conditions based on past investigations as described previously, geomorphology of the area, and past history were considered in selection of the most critical reaches of each levee unit. Generally the conditions in the Sutter Basin consist of a natural waterside berm of sands and silts and a natural top blanket consisting of silts grading to clay in the basin with the permeability decreasing with distance to the basin. The impervious blanket thickness, soil types (for determination of the permeability ratio), and aquifer thickness were considered for selection of the cross section representing a characteristic reach of every levee unit. Existing subsurface investigation showing the location of the borings, soil profiles and boring logs are those used by the ULE study and the FRWL Improvement Project and are provided in Enclosure J. The expected value, standard deviation and the coefficient of variation of the permeability ratios between the horizontal permeability through the sandy aquifer and vertical permeability of the impervious blanket, blanket thickness and thickness of the sandy aquifer for each reach and for the each index point are provided in Enclosure D. The information was based on soil profiles developed using existing subsurface investigation. Underseepage analysis was performed using the blanket theory as described in the Corps ETL 1110-2-556. Finite element analyses using SEEP2D program part of the GMS version 6 were developed to independently check the blanket theory results. Statistical analysis was used for each reach in determination of the coefficients of variation of the permeability ratios an, blanket thickness and thickness of the underlying aquifer as shown on Enclosure C. A critical gradient $i_c = 0.80$ was used considering the unit weight of the blanket 112pcf. The unit weight of the blanket was considered the same for all sections analyzed. The standard deviation for the blanket thickness and the thickness of the sand layer in the foundation was calculated for each index point of each levee unit and was used in underseepage reliability evaluation. The phreatic line obtained by the SEEP2D program using the finite element method was also used in the stability analysis for each water elevation.

Reliability analysis was performed using Taylor’s Series Method. In the Taylor method, random variables are quantified by their expected values, standard deviations, and correlation coefficients. The variables used in the generalized equation for underseepage analysis are shown on Figure 5-1 and are as follows:

$$X_3 = \sqrt{z * d * \left(\frac{k_f}{k_b}\right)}$$

$$i_o = \frac{h_0}{D_b} = \frac{H\left(\frac{k_f}{k_b} D_f D_b\right)}{D_b\left[C_R \frac{e^{2L_R} - 1}{e^{2L_R} + 1} + x_2 + \left(\frac{K_f}{K_{bL}} D_f D_{bL}\right)\right]}$$

Where

$$h_0 = \frac{H * L_e}{L_1 + L_2 + L_e}$$

$$L_1 = C_R * \frac{e^{2L_R} - 1}{e^{2L_R} + 1}$$

$$Pr(F) = P(i_{critical} < i_o)$$

Where: X_3 = C_R = effective exit distance

z = D_b = blanket thickness (it is assumed that the blanket thickness is the same on the waterside and landside D_b=D_{bl}=D_{br})

d = D_f = aquifer thickness

k_f = horizontal permeability of the aquifer

k_b = vertical permeability of the blanket

k_f/k_b = permeability ratio between horizontal permeability of the aquifer and vertical permeability of the impervious blanket

L_R = actual length of the riverside blanket

L_L = actual length of landside blanket assumed infinite (∞)

H = total head on levee

h_0 = water head at the landside levee toe

i_0 = upward seepage gradient through the landside blanket

L_1 = effective length of the waterside blanket

L_2 = X_2 = base width of natural blanket beneath the levee embankment (X_2)

L_e = C_R when the landside length of the natural blanket is assumed infinite

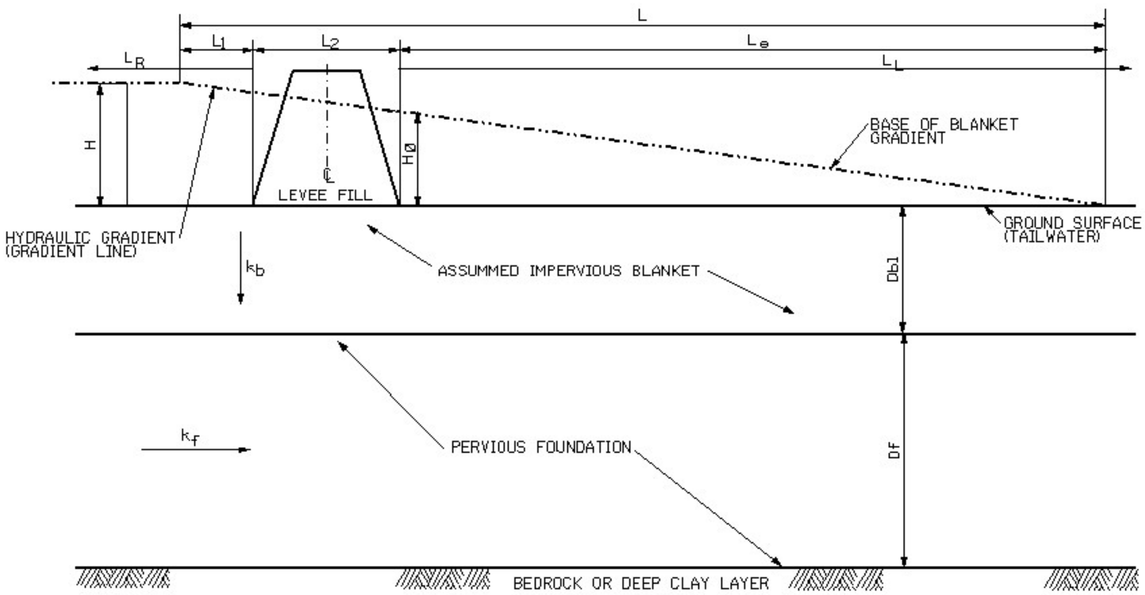

Figure 5-1 Underseepage Analysis – Blanket Theory

Seepage gradients were calculated for a range of river stages from the landside toe elevation to the top of the levee. From previous studies, the Taylor Series method appears to be more conservative and appropriate for this level of investigation. The index points for the risk analysis were selected based on deterministic analyses using the final element method developed by the SEEP2D program.

Permeability ratios between the horizontal permeability of the sand layer and vertical permeability of the impervious blanket, used in the blanket R&U analyses were based on a statistical analysis of the permeability of different foundation soils shown in the borings within the considered reach based the subsurface investigation and recommendations of the previous studies listed in Section 1.3.

The Sacramento District has developed a standard EXCEL workbook for the preparation of fragility curves. The mean, standard deviation, and coefficient of variation for the blanket thickness, pervious layer thickness, and the permeability ratio (horizontal permeability of the pervious layer divided by the vertical permeability of the blanket layer) are calculated for each index point using nearby soil borings on the first sheet in the workbook. Those parameters are then transferred to the second sheet in the workbook for the actual blanket theory calculations, with one potential exception. If the standard deviation is higher than the mean, then the mean minus one standard deviation will be a negative number, which is not physically possible. In that case, the coefficient of variation will be over 100. To avoid this problem, the workbook limits the coefficient of variation to a maximum of 98. If the coefficient of variation is 98, then the standard deviation is recalculated on the second sheet in the workbook based on the maximum allowed coefficient of variation. For this study, the permeability ratio was the parameter most often impacted by a large standard deviation. Soils deposited by river systems are highly variable compared to engineered materials like concrete and steel, and high standard deviations are not unexpected when dealing with soil materials.

Exit gradients calculated using blanket theory are highly sensitive to blanket layer thickness. For meandering river systems, it is not uncommon to have large differences in the blanket layer thickness; in this study, two soil borings along the Feather River show blanket layer thicknesses greater than 100 feet. Unusually large blanket layer thickness values will significantly impact the mean and cause the standard deviation to be large. To eliminate this effect, soil borings near the index points with unusually large blanket layer thickness (over 31 feet) were not used to calculate the random variables and statistical parameters. Soil borings with blanket layer thickness greater than 31 feet are the exception and not the rule in the SBLS. Basing the calculations on the borings with the thinner blanket layer is more representative of the conditions that are likely to cause an underseepage failure.

The detailed results of the underseepage reliability analyses for each index point are provided in Enclosure E.

5.3 Slope Stability Reliability

A sensitivity study was done to determine which parameters in the slope stability calculations were most influential. For this study, those variables are soil strength and unit weights of the soil in the levee embankment and in the foundation. Statistical descriptors for these variables were determined using available site-specific information and published statistical data. The piezometric lines or pore pressures for each water elevation were determined using the finite element program SEEP2D.

5.3.1. Cases Analyzed and Methods Used for Analyses

The cases analyzed for stability risk analyses considered long-term conditions with steady state seepage along the landside slope of the levee. The phreatic surface developed for the steady state condition was determined using the SEEP2D finite element computer program developed as part of the GMS. The limit equilibrium computer program “UTEXAS4” was used to perform the stability analyses. Circular failure surfaces were assumed and the embankment was modeled as homogeneous. All analyses consisted of running a search routine to identify the critical failure surface using the Spencer's Method. Five random variables were defined for each unit. Stability analyses were performed for different assumed river stages. Details of the results of the stability risk analysis are provided in Enclosure E.

5.3.2 Soil Strength Parameters

Soil strength parameters used in the stability analyses were the drained soil parameters as determined in previously studies listed. The coefficient of variation for soil strength parameters and unit weight of the fill material in the levee or the top impervious blanket were obtained using methodologies outlined in ETL 1110-2-556, and those proposed by Harr in the "Reliability-Based Design in Civil Engineering", and Duncan in the "Manual for Geotechnical Engineering Reliability Calculations". Cohesion of 50 psf was considered for the embankment fill when the levee embankment was constructed of sandy materials, to avoid shallow slipping surfaces close to the infinite slope failure.

5.4 Through Seepage Analysis

The through-levee seepage performance mode was not included in the development of fragility curves. The levees along the Wadsworth Canal and the Sutter Bypass are comprised mostly of clays and sandy clays with some zones of silty sand. The west levee of the Feather River is variable with regards to soil type. The Maintenance Area 3 segment and the southern portion of the Levee District 1 segment are composed of sandy silt and sandy clay. The upstream portion of the Levee District 1 segment is composed of silty sand, sandy silt, and some zones of sand. The Levee District 9, Maintenance Area 16, and Maintenance Area 7 segments are composed of sandy silt and sandy clay. The Hamilton Bend segment is composed of gravel and sand, with a fines content between 2 and 10 percent, but this segment has a crest width of about 60 feet. The Cherokee Canal levee is composed mostly of clay, silt, and sandy silt, with occasional small pockets of silty sand. Clay, sandy clay, and sandy silt levees are unlikely to experience through-levee seepage distress due to the low permeability of those soils. While silty sand and sand levees can be susceptible to through-levee seepage distress, there is almost no history of this performance mode causing levee distress in the two levee systems covered under the SBFS. The only reported incidence of distress due to through-levee seepage in these levees was saturation and bulging of the landside levee slope of the Feather River in downtown Yuba City during the 1986 flood. A landside stability berm was constructed in this area by the local sponsor after that flood, and a cutoff wall was constructed through the levee in this area by the Corps of Engineers after the 1997 flood. Due to the mosty fine-grained levee soils, the lack of history of through-levee seepage distress, the levee improvements in the only reported location of through-levee seepage distress, and the enlarged levee section of the pervious Hamilton Bend segment, it was determined that through-levee seepage is unlikely to be a major contributor to the probability of failure of the levees in this study.

5.5 Judgment Base Reliability Analysis

A judgment based conditional probability function was based on existing conditions of the levee such as the impact of encroachments, vegetation, existing cracks and holes due to animal burrows, and based on 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 pretty significant in the analyzed areas. Impact of erosion on the levee performance was also included in the judgmental reliability analysis. The judgment-based curve is included for each analyzed levee cross section and in the combined curve.

In June 2009, an expert elicitation was conducted for the purpose of developing the geotechnical judgment portion of the curves for the American River Common Features project for various conditions of the levee regarding vegetation, encroachments, rodents activity, penetrations and erosion. This expert elicitation was conducted in accordance with ETL 1110-2-561, "Appendix E, Expert Elicitation in Geological and Geotechnical Applications" 31 January 2006. The members of the expert elicitation team were highly recognized professional specialists, representing the LMAs of those levee systems, and specialists in erosion and in geotechnical issues. The expert elicitation focused on the judgment part of the geotechnical risk and uncertainty curves for the flood control structures. The expert elicitation was conducted over a three-day period in which the most representative reaches of each basin of the study were discussed. The expert elicitation team discussed and reached consensus on the impact of different factors of the judgment curve, such as:

a) the vegetation on the levees and within the levee right of way
b) penetrations through the levee and foundation
c) encroachments into the levee and levee right-of-way
d) erosion of the riverbank and waterside slopes of the levee
e) animal burrows

The expert elicitation also concluded that up to a certain water elevation, the risk of poor performance as determined by analyses or considered in the judgment portion of the curves may not necessarily coincide with the risk of failure. Based on historical performances of the levees, it appears that the risk of failure as determined in the analyses may be conservative and the poor performance of a levee may not lead to a catastrophic levee failure, even if damage to the levee embankment needs to be repaired after the flood to bring the levee back to the pre-flood performance. Consequently, the risk of catastrophic failure may be reduced based on the historical past performance, and consequently the curves may be altered. The conclusion reached by the panel was that the risk of failure as a function of stage of the river may be reduced by 50% when the river reached 4-5 feet above the landside toe, by 30% when the river stage is up to 8-9 feet above the landside levee toe, and by 10% when the river reaches 11-12 feet above the landside toe. Geotechnical R&U curves for poor performance and risk of failure considering the conclusions of the expert elicitation were provided for further analysis in HEC-FDA. The judgmental curves for the existing conditions are provided in Enclosure E.

5.6 Combined Reliability Analysis

The total conditional probability of failure as a function of floodwater elevation has been developed by combining the probability of failure functions for three failure modes; underseepage, slope instability, and judgment. The reliability is the probability of no failure due to each mode considered in the calculations. The analysis also assumed that no flood fighting is employed.

5.7 Soil Parameter Selection

The soil parameters used for underseepage and slope stability were assigned function of the type of the materials in the upper impervious blanket and in the aquifer underneath. The expected values were selected based on design charts in the EM 1110-2-1913, values used by the geotechnical report for the Feather River West Levee Improvement Project, and on values recommended by the Guidance for the DWR ULE program. All presumptive relationships were considered when selecting initial values (note that they are all very similar); however, seepage modeling was performed by varying these presumptive values to produce flow nets, which realistically match the generalized subsurface conditions. Therefore, the final values utilized may not be the same as the presumptive value used to begin the modeling process.

5.8 Reach Selection

Reaches were selected based on a geotechnical evaluation of the subsurface profile boring logs. The critical points are locations in each project phase that have the most significant risk of failure. The reaches were chosen to represent sections of levee with similar levee and foundation conditions.

Reaches were identified using the following criteria:

- Similar foundation blanket type (e.g. clay, silt, etc, a change may indicate a different reach)
- Similar foundation blanket or pervious layer thickness (e.g. the absence of a blanket or the absence of a pervious layer constitutes a difference reach)
- Significant changes in layer thickness, such those that result of a geomorphology difference (e.g. soil deposits with a major change in layer heterogeneity)

Experience has also shown that smaller reaches provide more consistent means and standard deviations, which represent actual geomorphological related subsurface conditions.

6 EVALUATION OF EXISTING GEOTECHNCIAL CONDITIONS

The SBLS levees completely encircle the Sutter Basin (Plate 1). The levee system is approximately 65 miles long. This levee system is broken into reaches by the ULE P1GER report, except for the Cherokee Canal levee, which is not included in the ULE study.

The Sutter Triangle levees tie into high ground at both ends at the Sutter Buttes (Plate 1). This levee system is approximately 10 miles long. This system is not included in the ULE study.

The height of the levees ranges from 4to 30 feet. The landside slopes are typically2H:1V. Portions of the system have landside slopes as steep as 1.5H:1V. The waterside slopes are typically 2.5H:1V to 4H:1V, with a typical slope of 3H:1V. The levee crown width ranges from 10 to 70 feet and are typically 16 to 20 feet. In the highly urbanized area of Yuba City, the Levee District 1 Sutter County segment has buildings, fences, roads, and utility poles located in close proximity to the levee. In rural areas, unpermitted fence encroachments are common. More detailed descriptions of the levees in the ULE study are in Section 2 of the ULE P1GER report. More detailed descriptions of the levees in the NULE study are in Appendix C (Cherokee Canal) and Appendix E (Sutter Triangle) of the NULE GAR report.

6.1 Past Performance and Levee Modifications/Improvements

Section 1.2 of this report gives a brief summary of the levee performance history during flood events. More detailed information may be found in chapter 2 of the ULE P1GER report for levee segments covered under the ULE program and Appendix E of the NULE GAR report for the Sutter triangle levees. Tables summarizing the performance history and constructed modifications/improvements of the SBLS from Periodic Inspection Report No. 1 (2010) are in Enclosure B of this report.

6.2 Explorations

The ULE P1GDR (2008) and SGDR (2010) and the SBFCA GDR (2012) reports summarize explorations for the majority of the project. The Cherokee Canal left levee, Wadsworth Canal right levee, and the Sutter Bypass left levee upstream of Wadsworth Canal are included in the DWR NULE project, but no NULE explorations have been conducted at this time. Nine soil borings were conducted for the Cherokee Canal left levee for this Feasibility Study. The boring logs are included in Enclosure F of this report.

6.3 SBLS Seismic Evaluation

The fragility curves were developed following ETL 1110-2-566, which does not include a seismic component. The Sacramento District has developed a draft manual for seismic deformation evaluation of levees which is currently under review by Corps Headquarters, the Risk Management Center (RMC) and the Engineering Research and Development Center (ERDC) for nationwide implementation. The recommendation of the proposed manual is to design for seismic loading only levees loaded permanently by high water levels. Levees subject to intermittent loading will only be evaluated for liquefaction potential and seismic deformation. We recommend the Sponsor identify an emergency borrow area to reconstruct the levee within 3 months to a limited level of protection (10% (1/10) Annual Chance Exceedance (ACE)). Additional information on the seismic evaluation is given in section 8.6.3 of this report.

6.4 Deterministic Analysis

Deterministic slope stability and foundation underseepage analyses were conducted at thirteen cross-sections along the Sutter system corresponding to the reaches identified in Paragraph 6.1 as the basis for the risk based analyses and to identify the index points for each reach. In some cases, the deterministic analyses were conducted at the index points for this study and in other cases the deterministic analyses were conducted a short (half-mile or less) distance away from the index point. The purpose of the deterministic analysis was to verify the deterministic analyses conducted under the two outside studies and as a “check” for the fragility curves prepared under this study.

The Groundwater Modeling System (GMS) computer program was used for all analysis seepage and stability analysis. GMS is a model wrapper program that incorporates a graphical user interface (GUI). The modeler builds the subsurface model in GMS and the subsurface model is exported to the appropriate analysis module. The GMS program includes modules such as SEEP2D for finite-element seepage modeling, and UTEXAS4 for slope stability analysis.

For this project, topographic cross-sections were cut from triangulated irregular network (TIN) 3D surfaces using AutoCAD, which derived the TIN surface from the Comp Study topography identified in Section 2.0. This CADD cross section was combined with boring stick logs exported to CADD format from the gINT boring log program. The combined CADD file containing the appropriate boring logs and topographical cross-section was imported into the GMS program. Once in GMS the modeler imported the topography layer from CADD to object layers in GMS.

Enclosure G presents the graphical output from the deterministic analyses.

7 GEOTECHNICAL FRAGILITY CURVES

7.1 Reach Selection

The ULE P1GER (2008) report divides the SBLS into forty reaches. Reach selection grouped portions of the levee into segments with similar performance, previous repairs, or geomorphology. For the purposes of the SBFS, we grouped the ULE-developed reaches into ten larger reaches with similar subsurface conditions, and added one reach each for the Cherokee Canal left levee, Wadsworth Canal right levee, and Sutter Bypass left levee upstream of Wadsworth Canal, which are not in the ULE study. The resulting thirteen reaches were developed in consultation with the project hydraulic engineer and economist. Note that the levee reaches used to develop the fragility curves are different than the levee segments listed in section 1.1 of this report. The levee segments are determined by levee maintaining agency, while the levee reaches were developed for analysis based on the criteria listed in section 5.6 of this report.

The thirteen reaches are as follows:

Wadsworth Canal (left): ULE A and B

Sutter Bypass Upper: ULE C, D, E, and F

Sutter Bypass Lower: ULE H, I, J, K, and L

Feather River South (near Bear) = ULE M, N, O, P,Q ,and R

Feather River Abbot/Connor Lakes: ULE S, T1, and T2

Feather River Shanghai Bend: ULE U, V, W, X, Y, Z1, Z2, Z3, and Z4

Feather River North LD9: ULE AA, BB, CC, DD, and EE

Feather River North Middle: ULE FF, GG, HH, II, JJ, KK, and LL

Feather River North Canal: ULE JJ, KK, and LL

Feather River North Hamilton Bend: ULE MM, NN, OO, QQ, RR, and SS

Cherokee Canal: ULE Not applicable

Wadsworth Canal (right): ULE Not applicable

Sutter Bypass (upstream) : ULE not applicable

Table 7-1 provides a location summary of the index points associated with these reaches. The index point locations are shown on Plates 2 through 9 of this report. These reaches are described in subsections of 7.2 through 7.14, including a short explanation of the rationale of the reach selection.

Table 7-1. Index Point Location Information

Index Point Number	Reach Name	PLM	NAD83_N	NAD83_E
1	Wadsworth Canal (left)	0.84	2170954.8600	6629916.3000
2	Sutter Bypass Upper	6.20	2158851.0000	6631970.0000
3	Sutter Bypass Lower	17.30	2113476.9763	6655398.0817
4	Feather River South (near Bear)	4.92	2106963.5800	6679261.2400
5	Feather River Abbott/Connor Lakes	3.99	2127081.8143	6676331.1294
6	Feather River Shanghai	9.31	2156078.1800	6673804.9800
7	Feather River North LD9	0.52	2188213.8800	6668679.4100
8	Feather River North Middle	0.90	2224154.3700	6664999.3400
9	Feather River North Canal	2.90	2233626.2500	6664328.5400
10	Feather River North Hamilton Bend	0.51	2288660.9600	6662820.2400
11	Cherokee Canal	9.50	2301045.9480	6637006.2610
12	Wadsworth Canal (right)	0.50	2168750.0000	6627910.0000
13	Sutter Bypass (upstream)	4.0	2168110.0000	6626590.0000

7.2 <u>Wadsworth Canal (left)</u>

The section describes the reach selection and modeling criteria for the Wadsworth Canal left levee from the Sutter Bypass (PLM 0.00) upstream to the East Interceptor Canal (PLM 4.66).

<u>Reach Selection:</u> The ULE report identifies two reaches for this levee: Reaches A and B. In Reach A, the levee is shorter, and the height reduces toward the upstream end. Reach B south of Franklin Road is a taller levee and is characterized by relatively thin sand layers layers in the foundation. The critical index point for the Wadsworth Canal left bank levee was considered towards the downstream (taller) end since the hydraulic head is greater and more susceptible to underseepage than a shorter levee. The statistical parameters for blanket and pervious layers were developed using soil types identified in soil borings from the ULE P1GER report. There is a forty to fifty foot deep, 3000 feet long, soil-cement-bentonite cutoff wall at the southern end of this reach. The index location is situated upstream from the cutoff wall at PLM 0.83.

Selection of Cross-Section: The cross-section at PLM 0.83, was selected to represent the portion of the reach not protected by a cutoff wall; however, the statistical variation of the parameters of the blanket and sand (thickness and permeability) considered all borings within the reach, including those in the area with a cut-off wall.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the impervious blanket in the vicinity of the selected index point varies between 3 and 10.5 feet and consists of clay and silt blanket overlaying a silty sand aquifer 2-11 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-2.

Table 7-2 Permeabilities, Wadsworth Canal (left)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Thin Clay Blanket	$3.9x10^{-4}$	1.12	$9.8x10^{-4}$	0.28	4
Thick Clay Blanket	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.0071	4
Silty Sand (SM)	$9.8x10^{-3}$	2.8	$9.8x10^{-4}$	0.28	10
Sand with Silt (SP-SM)	$4.02x10^{-3}$	11.4	$4.02x10^{-4}$	1.14	10

The standard deviation and coefficient of variation for the permeability ratios (between the horizontal permeability of the sandy aquifer and vertical permeability of the impermeable upper blanket), blanket thickness and sandy aquifer thickness are summarized in Table 7-3 as follows.

Table 7-3 Random Variables, Wadsworth Canal (left)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	7	2	29

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permeability Ratio (k_f/k_b)	493	455	92
Foundation Sand thickness (d)	7	3	43

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-4

Table 7-4 Stability Random Variables, Wadsworth Canal (left)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	31	4	12
Levee Unit Weight γ (pcf)	125	6	5
Blanket Layer Cohesion c (psf)	150	50	33
Blanket Layer Unit Weight γ(pcf))	115	6	5
Blanket Layer Φ(degrees)	28	3	12

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-1. The underseepage performance mode accounts for most of the risk of failure for this levee segment. This is consistent with the performance history of excessive seepage and boils over the downstream portion of this levee, where it is tallest.

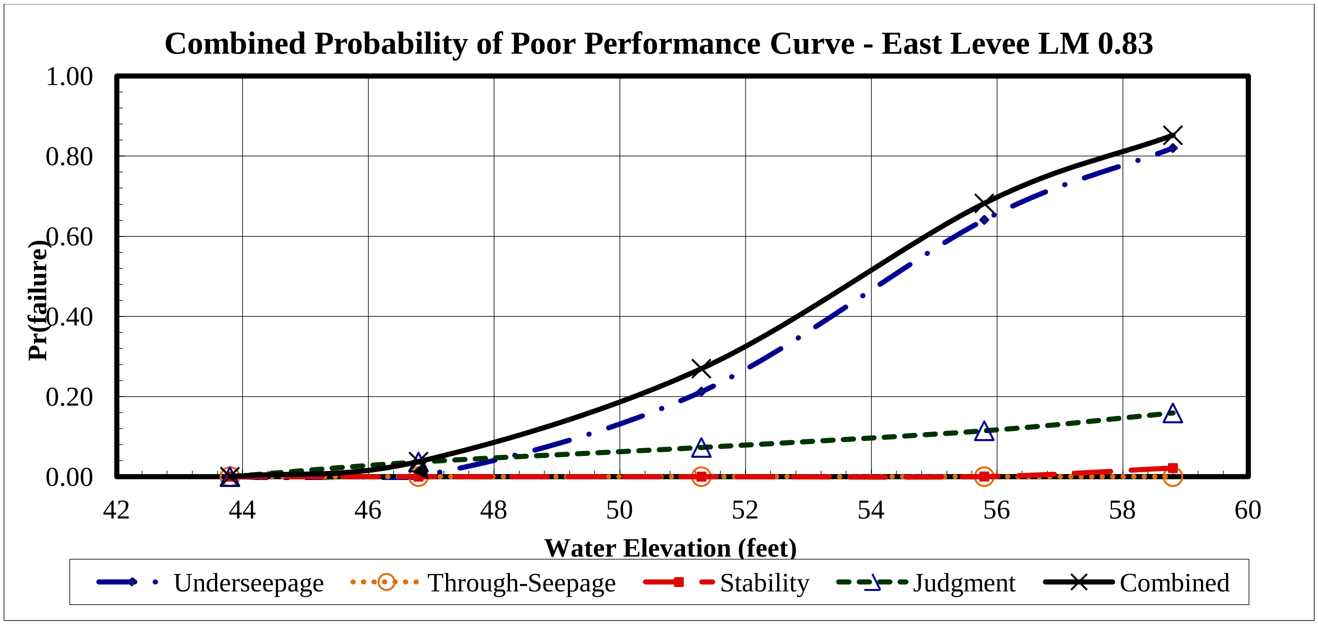

Figure 7-1 Combined R&U, Wadsworth Canal (left)

7.3 Sutter Bypass; Upper Reach

The section describes the reach selection and modeling criteria for the Sutter Bypass Upper Reach, from PLM 4.40 at the Wadsworth Canal downstream to PLM 12.7 at Gilsizer Slough.

Reach Selection:

The Sutter Bypass has experienced many seepage problems over the years. The channel was blasted out of the hardpan clay and the levees were built adjacent to the cut. This reduces the waterside blanket to almost zero. The Bypass channel crosscuts the geology of the region, with the orientation of the natural flood overbank deposits from the Feather, Yuba and Bear Rivers passing roughly perpendicular to the channel and levee. The results are that the Sutter Bypass has similar subsurface conditions and performance during flood events over the entire segment.

Within the Sutter Bypass, geomorphically, Gilsizer Slough stands out as a place to separate reaches. Gilsizer Slough is an historically major drainage in the Sutter Basin, and intersects the bypass near the midpoint. For this reason, the bypass was divided into two reaches; the Upper Reach and the Lower Reach.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the impervious blanket in the vicinity of the upper reach index point varies between 4.8 and 14 feet and consists of clay, sandy clay, and occasional silty sand overlaying a sand and silty sand aquifer 3-10 feet thick. The borings used for the R&U analyses for the upper reach are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-5.

Table 7-5 Permeabilities, Sutter Bypass (Upper)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay/Sandy Clay Blanket (CL)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silty Sand Blanket (SM)	$4.2x10^{-4}$	1.2	$1x10^{-4}$	0.3	4
Silty Sand Aquifer(SM)	$1x10^{-3}$	3	$1x10^{-4}$	0.3	10
Sand with Silt Aquifer (SP-SM)	$3.5x10^{-3}$	10	$3.5x10^{-4}$	1	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-6 as follows.

Table 7-6 Random Variables, Sutter Bypass (Upper)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	10	4	40
Permeability Ratio (k_f/k_b)	725	650	90
Foundation Sand thickness (d)	10	6	60

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-7

Table 7-7 Stability Random Variables, Sutter Bypass (upper)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	31	4	12
Levee Unit Weight γ (pcf)	125	6	5
Blanket Layer Cohesion c (psf)	150	50	33
Blanket Layer Unit Weight γ(pcf))	115	6	5
Blanket Layer Φ(degrees)	28	3	12

<u>Probability of Failure Curve:</u> The combined fragility curve is shown in Figure 7-2. The underseepage performance mode accounts for most of the risk of failure for this levee reach. This is consistent with the performance history of excessive seepage, boils and ground heaving near the toe of this levee.

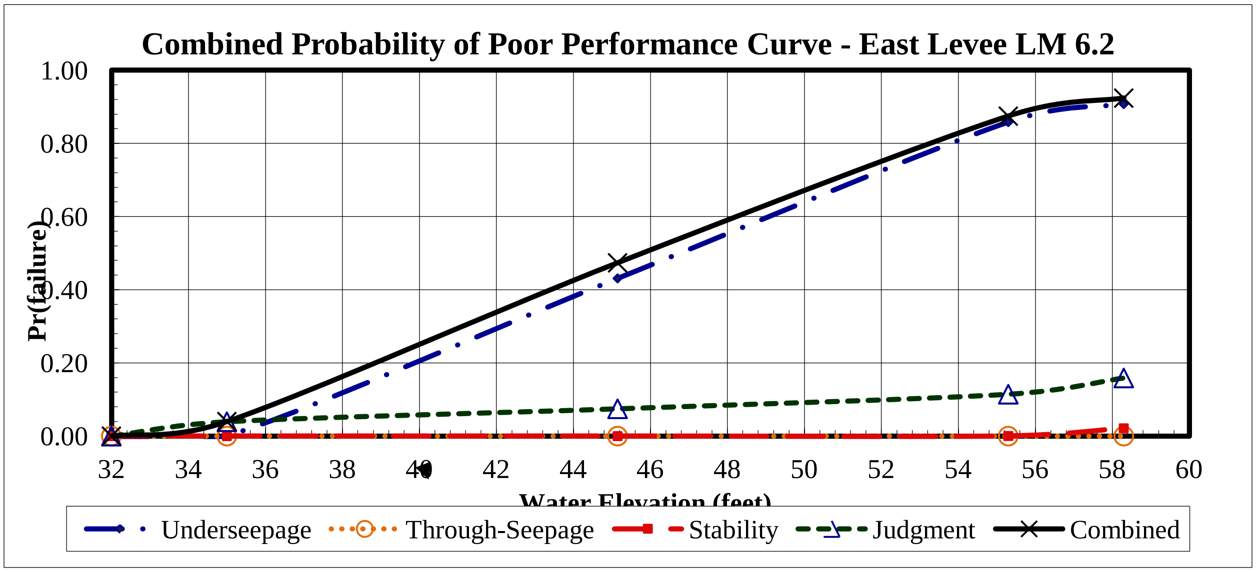

Figure 7-2 Combined R&U, Sutter Bypass (Upper)

7.4 Sutter Bypass (Lower)

The section describes the reach selection and modeling criteria for the Sutter Bypass Lower Reach.

Reach Selection:

The Sutter Bypass has experienced many seepage problems over the years. The channel was blasted out of the hardpan clay and the levees were built adjacent to the cut. This reduces the waterside blanket to almost zero. The Bypass channel crosscuts the geology of the region, with the orientation of the natural flood overbank deposits from the Feather, Yuba and Bear Rivers passing roughly perpendicular to the channel and levee. The results are that the Sutter Bypass has similar subsurface conditions and performance during flood events over the entire segment.

Within the Sutter Bypass, geomorphically, Gilsizer Slough stands out as a place to separate reaches. Gilsizer Slough is an historically major drainage in the Sutter Basin, and intersects the bypass near the midpoint. For this reason, the bypass was divided into two reaches; the Upper Reach and the Lower Reach.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the impervious blanket in the lower reach varies between 4 and 26.4 feet and consists of clay, silt, sandy clay, and sandy silt overlaying a sand and silty sand aquifer 3-28 feet thick. The borings used for the R&U analyses for the lower reach are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-8.

Table 7-8 Permeabilities, Sutter Bypass (Lower)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay/Sandy Clay Blanket (CL)	9.8×10^{-6}	0.028	2.5×10^{-6}	0.007	4
Silty Sand Blanket (SM)	1.4×10^{-4}	0.4	3.5×10^{-5}	0.1	4
Silty Sand Aquifer(SM)	1×10^{-3}	3	1×10^{-4}	0.3	10
Sand with Silt Aquifer (SP-SM)	3.5×10^{-3}	10	3.5×10^{-4}	1	10
Sand Aquifer (SP)	9.8×10^{-3}	28	9.8×10^{-4}	2.8	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-9 as follows.

Table 7-9 Random Variables, Sutter Bypass (Lower)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	15	10	67
Permeability Ratio (k_f/k_b)	1117	1095	98
Foundation Sand thickness (d)	13	13	98

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-10.

Table 7-10 Stability Random Variables, Sutter Bypass (lower)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	31	4	12
Levee Unit Weight γ (pcf)	125	6	5
Blanket Layer Cohesion c (psf)	150	50	33
Blanket Layer Unit Weight γ(pcf))	115	6	5
Blanket Layer Φ(degrees)	28	3	12

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-3. The underseepage performance mode accounts for most of the risk of failure for this levee reach.

This is consistent with the performance history of excessive seepage, boils and ground heaving near the toe of this levee.

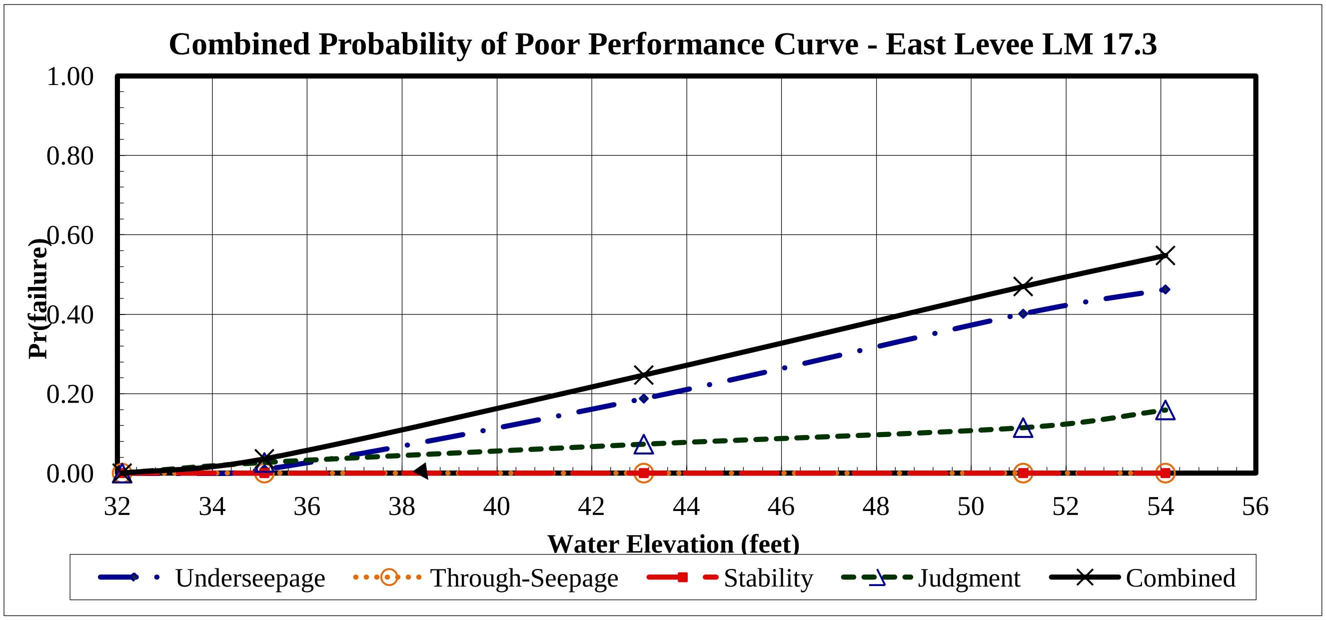

Figure 7-3 Combined R&U, Sutter Bypass (Lower)

7.5 Feather River South (near Bear)

The section describes the reach selection and modeling criteria for the Feather River South (near Bear) reach.

Reach Selection:

This reach is characterized by geomorphology dominated by the interaction between the Bear and Feather River systems since the Pleistocene, and therefore is named after the Bear River.

Selection of Cross-Section: The cross-section FEATHER RIVER SOUTH (near Bear) was selected to represent the most vulnerable location in the reach and is located at PLM 4.92.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the impervious blanket in the vicinity of the index point varies between 14 and 26 feet and consists of silt, clay, sandy silt, and sandy clay overlaying a sand, gravel, and silty sand and gravel aquifer 17-89 feet thick. The borings used for the R&U analyses for the upper reach are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-11.

Table 7-11 Permeabilities, Feather River South (near Bear)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Thick Clay/Sandy Clay/Sandy Silt Blanket (CL, ML)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Sandy Silt Blanket (ML)	$1.97x10^{-4}$	0.56	$4.9x10^{-5}$	0.14	4
Silty Sand Aquifer(SM)	$3.5x10^{-4}$	1	$3.5x10^{-5}$	0.1	10
Sand Aquifer	$3.5x10^{-3}$	10	$3.5x10^{-4}$	1	10
Sand and Gravel Aquifer (SP-SM, GW-GM)	$1x10^{-2}$	30	$1x10^{-3}$	3	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-12 as follows.

Table 7-12 Random Variables, Feather River South (near Bear)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	22	5	23
Permeability Ratio (k_f/k_b)	2172	1977	91
Foundation Sand thickness (d)	73	28	38

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-13

Table 7-13 Stability Random Variables, Feather River South (near Bear)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	1	5
Levee Unit Weight γ (pcf)	120	6	5
Blanket Layer Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	120	6	5
Blanket Layer Φ(degrees)	31	4	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-4. The underseepage performance mode accounts for much of the risk of failure for this levee reach. This is consistent with the performance history of excessive seepage and boils during flood events.

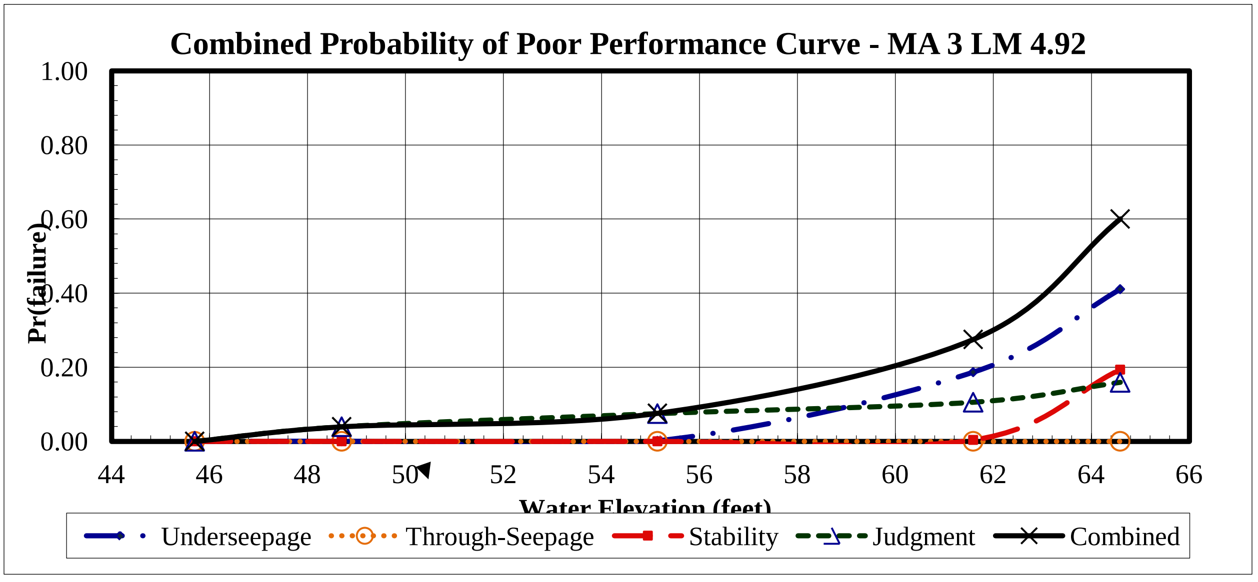

Figure 7-4 Combined R&U, Feather River South (near Bear)

7.6 Feather River South – Abbot/Conner Lakes Reach

The section describes the reach selection and modeling criteria for the Feather River South – Abbot/Connor Lakes reach.

Reach Selection:

This reach of levee was constructed near historic marshland (Abbot and Connor Lakes). The levees here have typically had underseepage problems. Relief wells and a recent setback levee with a cutoff wall in the foundation was constructed to mitigate for hydraulic and underseepage concerns over a portion of this reach; however most of this reach does not have any seepage mitigation measures in place. This reach is located from PLM 6.25 (LD1) downstream to PLM 2.74 (LD1) and is about 4.4 miles long.

Selection of Cross-Section: The cross-section was selected to represent the portion of the reach most likely to breach. This section is located at PLM 3.99.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 5 and 25.5 feet and consists of clay, sandy clay, and silty sand overlaying a silty sand and gravel aquifer 3-97 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-14.

Table 7-14 Permeabilities, Feather River South (Abbott/Connor Lakes)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Thin or Sandy Clay Blanket (CH, CL, ML)	1.4×10^{-5}	0.04	3.5×10^{-6}	0.01	4
Clay Blanket (CL)	9.8×10^{-6}	0.028	2.5×10^{-6}	0.007	4
Silty Sand Blanket (SM)	3.5×10^{-4}	0.4	3.5×10^{-5}	0.1	4
Silty Sand Aquifer (SM)	9.8×10^{-4}	2.8	9.8×10^{-5}	0.28	10

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Sand Aquifer	$3.5x10^{-3}$	10	$3.5x10^{-4}$	1	10
Sand and Gravel Aquifer (SP-SM, GW-GM)	$2.5x10^{-2}$	70	$2.5x10^{-3}$	7	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-15 as follows.

Table 7-15 Random Variables, Feather River South (Abbott/Connor Lakes)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	14	9	66
Permeability Ratio (k_f/k_b)	10526	10315	98
Foundation Sand thickness (d)	26	25	98

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-16

Table 7-16 Stability Random Variables, Feather River South (Abbott/Connor Lakes)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	1	5

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Unit Weight γ (pcf)	120	6	5
Blanket Layer Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	120	6	5
Blanket Layer Φ(degrees)	31	4	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-5. The underseepage performance mode accounts for much of the risk of failure for this reach. This is consistent with the performance history of excessive seepage and boils during flood events.

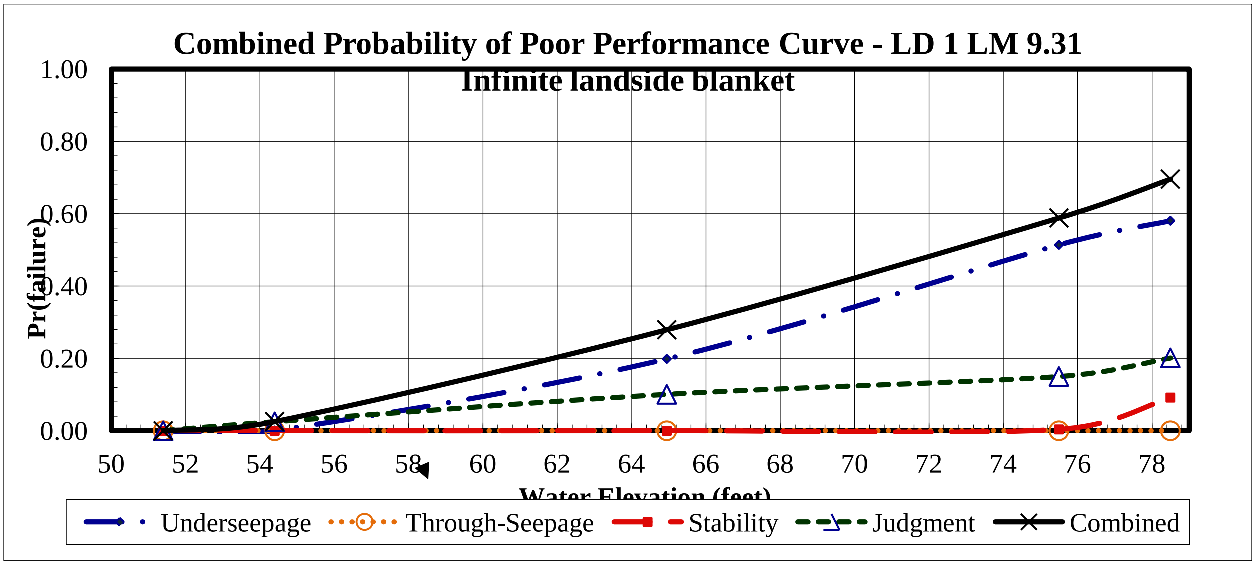

Figure 7-5 Combined R&U, Feather River South (Abbott/Connor Lakes)

7.7 Feather River South – Shanghai Reach

The section describes the reach selection and modeling criteria for the Feather River South – Shanghai Reach.

Reach Selection:

This segment represents the reach nearest to Yuba City where several previous levee strengthening projects have been completed from PLM 16.65 (LD1) downstream to PLM 6.25 (LD1) and is about 9.9 miles long. Levee strengthening projects include relief wells just upstream of Shanghai Bend, a cutoff wall upstream of the relief wells (with a slight overlap), and a stability berm within downtown Yuba City. This levee breached just north (upstream) of Shanghai Bend at approximate PLM 11.8 during the 1955 flood. Portions of this reach have had no levee strengthening projects.

Selection of Cross-Section: The cross-section at PLM 9.31 of LD1 was selected to represent the portion of the reach not protected by a cutoff wall or relief wells; however, the parameters of the blanket and sand thickness were partially derived from the borings in these areas solely for the purpose of developing reasonable statistical parameters.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 8.5 and 31 feet and consists of clay, silt, and sandy silt overlaying a sand and silty sand aquifer 2.5-30 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-17.

Table 7-17 Permeabilities, Feather River South (Shanghai)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay Blanket (CL)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silt/Clay Blanket (CL, ML)	$1.4x10^{-5}$	0.04	$3.5x10^{-6}$	0.01	4
Sandy Silt Blanket (ML)	$1.9x10^{-4}$	0.56	$4.9x10^{-5}$	0.14	4
Silty Sand Aquifer (SM)	$9.8x10^{-4}$	2.8	$9.8x10^{-5}$	0.28	10
Silty Sand Aquifer (SM)	$4.2x10^{-4}$	1.2	$4.2x10^{-5}$	0.12	10
Sand Aquifer (SP)	$4.9x10^{-3}$	14	$4.9x10^{-4}$	1.4	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-18 as follows.

Table 7-18 Random Variables, Feather River South (Shanghai)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	17	9	53
Permeability Ratio (k_f/k_b)	773	758	98
Foundation Sand thickness (d)	15	13	87

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-19.

Table 7-19 Stability Random Variables, Feather River South (Shanghai)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	1	5
Levee Unit Weight γ (pcf)	120	6	5
Blanket Layer Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	120	6	5
Blanket Layer Φ(degrees)	31	4	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-6. The underseepage performance mode accounts for much of the risk of failure for this reach. This is consistent with the performance history of excessive seepage and boils during flood events.

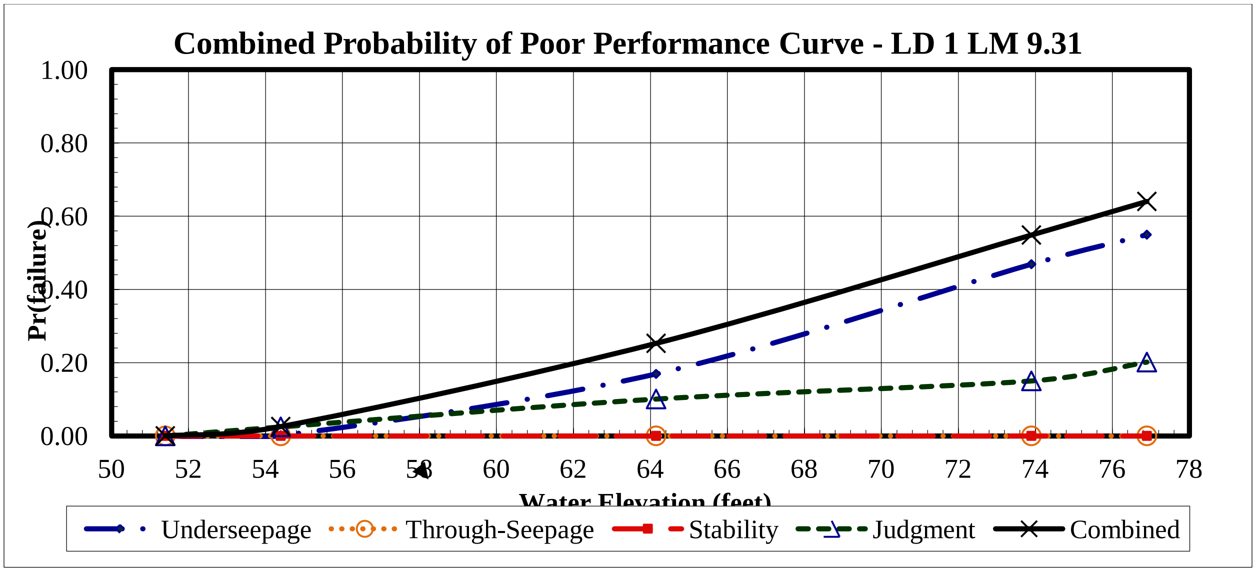

Figure 7-6 Combined R&U, Feather River South (Shanghai)

7.8 Feather River North – LD9 Reach

The section describes the reach selection and modeling criteria for the Feather River North – LD9 Reach.

Reach Selection:

This reach was selected to model the reach north of Yuba City that has experienced distress during flood events but has not had previous seepage or stability improvements, and is not adjacent to the Sutter Butte Canal. This reach is identified as PLM 5.6 downstream to PLM 0.0 of LD9, and is 5.6 miles long.

Selection of Cross-Section: The cross-section was selected to represent the portion of the reach most likely to breach based on previous screening analysis, and is located at PLM 0.52 of Levee District 9.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 7.5 and 20 feet and consists of clay, silt, and sandy clay overlaying a sand and silty sand aquifer 3.5-37 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-20.

Table 7-20 Permeabilities, Feather River North – LD9

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay Blanket (CL)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silt/Clay Blanket (CL, ML)	$1.9x10^{-4}$	0.56	$4.9x10^{-5}$	0.14	4
Sandy Clay Blanket (SC)	$1.9x10^{-3}$	5.6	$4.9x10^{-4}$	1.4	4
Silty Sand Aquifer (SW-SM)	$4.9x10^{-3}$	14	$4.9x10^{-4}$	1.4	10
Sand Aquifer (SW)	$9.8x10^{-3}$	28	$9.8x10^{-4}$	2.8	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-21 as follows.

Table 7-21 Random Variables, Feather River North – LD9

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	13	5	38
Permeability Ratio (k_f/k_b)	1730	1217	70
Foundation Sand thickness (d)	28	17	61

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-22.

Table 7-22 Stability Random Variables, Feather River North – LD9

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	4	13
Levee Unit Weight γ (pcf)	120	48	40
Blanket Layer Cohesion c (psf)	150	11	7
Blanket Layer Unit Weight γ(pcf))	120	16	13
Blanket Layer Φ(degrees)	31	12	40

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-7. The underseepage performance mode accounts for much of the risk of failure for this reach. This is consistent with the performance history of excessive seepage, and boils during flood events.

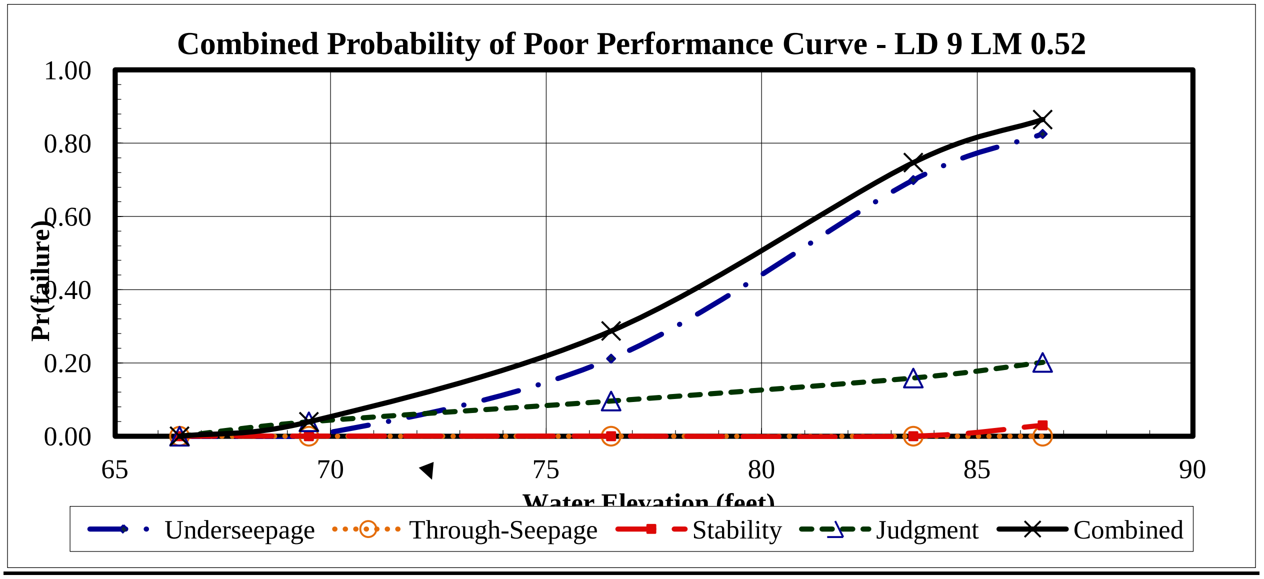

Figure 7-7 Combined R&U, Feather River North – LD9

7.9 Feather River North – Middle Reach

The section describes the reach selection and modeling criteria for the Feather River North – Middle Reach.

Reach Selection:

This reach was selected to model the reach within the agricultural area north of Yuba City that has not experienced any distress during flood events and is not adjacent to the Sutter Butte Canal. This reach is identified as PLM 2.8 downstream to PLM 0.0 of MA 16 and PLM 6.24 downstream to PLM 5.6 of LD9, and is approximately 3.44 miles long.

Selection of Cross-Section: The cross-section at PLM 0.09 of MA 16 was selected to represent this reach.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 3 and 15 feet and consists of silty clay and clay overlaying a sand and silty sand aquifer 1.5-30 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-23.

Table 7-23 Permeabilities, Feather River North - Middle

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay/Silt Blanket (CL, ML)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Thin Clay/Silt Blanket (CL, ML)	$4x10^{-5}$	0.114	$1.1x10^{-5}$	0.0284	4
Silty Sand Aquifer (SM)	$4.9x10^{-3}$	14	$4.9x10^{-4}$	1.4	10
Sand Aquifer (SP-SM)	$9.8x10^{-3}$	28	$9.8x10^{-4}$	2.8	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-24 as follows.

Table 7-24 Random Variables, Feather River North - Middle

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	6	5	76
Permeability Ratio (k_f/k_b)	2633	1742	56
Foundation Sand thickness (d)	16	8	53

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-25.

Table 7-25 Stability Random Variables, Feather River North - Middle

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	31	4	12
Levee Unit Weight γ (pcf)	125	6	5
Blanket Layer Cohesion c (psf)	150	50	33
Blanket Layer Unit Weight γ(pcf))	115	6	5
Blanket Layer Φ(degrees)	28	3	12

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-8. The underseepage performance mode accounts for much of the risk of failure for this levee reach.

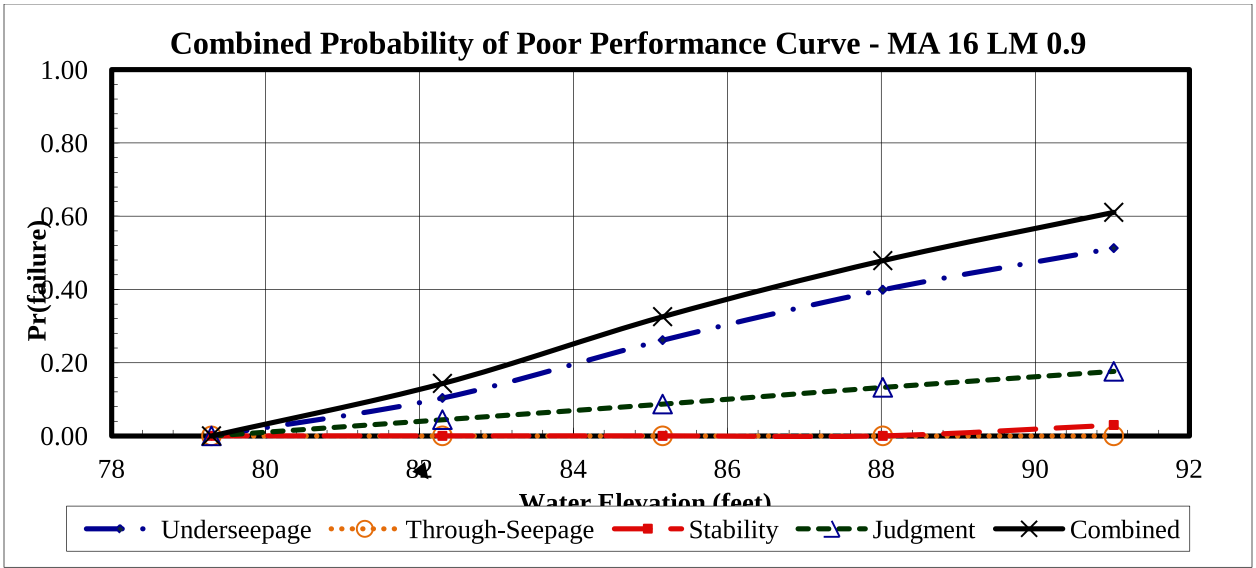

Figure 7-8 Combined R&U, Feather River North - Middle

7.10 Feather River North – Canal Reach

The section describes the reach selection and modeling criteria for the Feather River North-Canal Reach.

Reach Selection:

This reach represents the portion of the North Feather River levee that is abutted by the Sutter Butte Canal from PLM 2.65 downstream to PLM 0 of MA7, and PLM 4.09 downstream to PLM 2.8 of MA16. This reach is approximately 3.94 miles long.

Selection of Cross-Section: The cross-section selected to represent the levee with the irrigation canal is at PLM 2.9 of MA 16.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 8 and 20 feet and consists of silt, clay, sandy silt, and sandy clay overlaying a sand and silty sand aquifer 3-18 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-26.

Table 7-26 Permeabilities, Feather River North - Canal

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Thin Clay/Silt Blanket (CL, ML)	$3.9x10^{-4}$	1.12	$9.8x10^{-5}$	0.28	4
Clay/Silt Blanket (CL, ML)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silty Sand/Clayey Sand Blanket (SC, SM)	$1.9x10^{-4}$	0.56	$4.9x10^{-5}$	0.14	4
Silty Sand Aquifer (SM)	$4.9x10^{-3}$	14	$4.9x10^{-4}$	1.4	10
Sand Aquifer (SP-SM)	$9.8x10^{-3}$	28	$9.8x10^{-4}$	2.8	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-27 as follows.

Table 7-27 Random Variables – Feather River North - Canal

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	13	5	41
Permeability Ratio (k_f/k_b)	1053	1032	98
Foundation Sand thickness (d)	12	5	45

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-28.

Table 7-28 Stability Random Variables – Feather River North Canal

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	1	5
Levee Unit Weight γ (pcf)	120	6	5
Blanket Layer Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	120	6	5
Blanket Layer Φ(degrees)	31	4	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-8. The underseepage and judgment performance modes account almost equally for the risk for this levee reach. In addition to the canal adjacent to the landside toe, there are numerous culverts through this levee, many of which were installed in the 1940's.

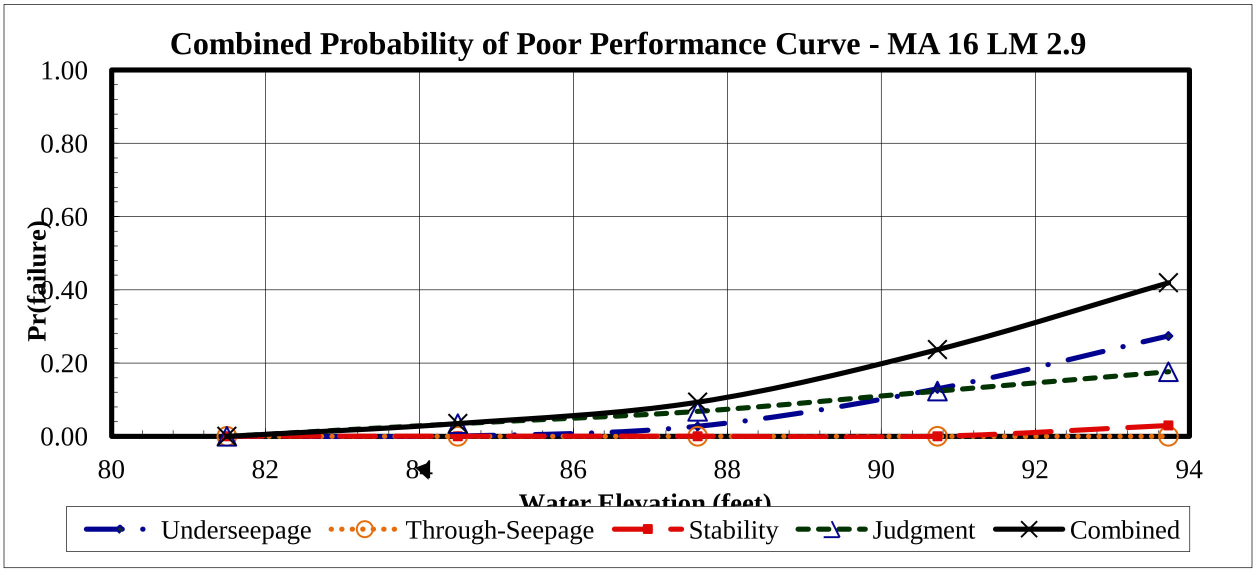

Figure 7-9 Combined R&U, Feather River North - Canal

7.11 Feather River North – Hamilton Bend Reach

The section describes the reach selection and modeling criteria for the Feather River North-Hamilton Bend Reach.

Reach Selection:

This reach is characterized by deposits of dredge tailings (silty and clayey gravel to small cobble size) beneath the levee from roughly PLM 1.20 downstream to PLM 0 (MA 7/Hamilton Bend Unit) and PLM 12.07 downstream to 2.65 (MA 7) for a total of roughly 8.8 miles. The MA 7/Hamilton Bend Unit levee was mostly constructed from the dredge tailings.

Selection of Cross-Section: The cross-section selected to represent the portion of the segment most likely to breach is where the levee is relatively tall at MA 7/Hamilton Bend Unit PLM 0.51.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 8.5 and 12 feet and consists of clay, sandy silt, sandy clay, and clayey sand overlaying a silty sand and gravel aquifer 10-60.5 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-29.

Table 7-29 Permeabilities, Feather River North – Hamilton Bend

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay Blanket (CL)	9.8×10^{-6}	0.028	2.5×10^{-6}	0.007	4
Sandy Silt/Sandy Clay Blanket (CL, ML)	1.4×10^{-5}	0.04	3.5×10^{-6}	0.01	4
Clayey Sand Blanket (SC)	1.9×10^{-4}	0.56	4.9×10^{-5}	0.14	4
Silty Sand and Gravel Aquifer (SM, GM)	4.9×10^{-3}	14	4.9×10^{-4}	1.4	10
Sand/Gravel Aquifer (SP-SM, SP, GM, GP)	2.5×10^{-2}	70	2.5×10^{-3}	7	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-30 as follows.

Table 7-30 Random Variables – Feather River North – Hamilton Bend

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	12	13	26
Permeability Ratio (k_f/k_b)	11742	11507	98
Foundation Sand thickness (d)	30	21	70

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-31.

Table 7-31 Stability Random Variables – Feather River North – Hamilton Bend

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	29	1	5
Levee Unit Weight γ (pcf)	120	6	5
Blanket Layer Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	120	6	5
Blanket Layer Φ(degrees)	31	4	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-10. The underseepage performance mode accounts for almost all of the risk for this levee reach. This is consistent with the foundation conditions of a thin blanket overlying a highly pervious sand and gravel layer.

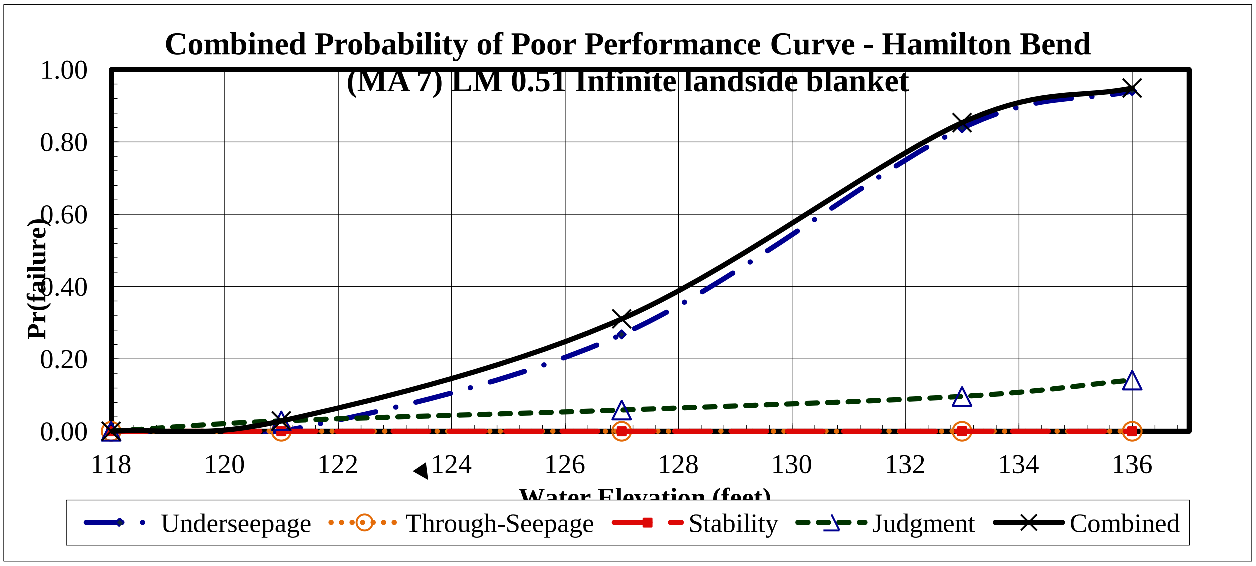

Figure 7-10 Combined R&U, Feather River North – Hamilton Bend

7.12 <u>Cherokee Canal</u>

The section describes the reach selection and modeling criteria for the Cherokee Canal left levee from PLM 9.9 upstream to PLM 6.10.

<u>Reach Selection:</u>

This reach consists of the entire portion of the levee that is within the SBFS. Boring logs on this levee from the Corps' Feasibility Study are in Enclosure D.

<u>Selection of Cross-Section:</u> The cross-section at PLM 9.50 was selected to represent the reach. It is the location of the boring most downstream, and at the location of the tallest portion of the levee. This location overtopped in 1986 due to flooding induced by debris build-up at the railroad bridge immediately downstream.

<u>Underseepage Reliability Analysis:</u> The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. The thickness of the blanket varies between 3 and 19 feet and consists mostly of silt and sandy silt with some low-plasticity clay overlaying a sand, silty sand, and clayey sand aquifer 1.5-14.5 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-32.

Table 7-32 Permeabilities, Cherokee Canal

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay Blanket (CL)	9.8×10^{-6}	0.028	2.5×10^{-6}	0.007	4
Silt Blanket (ML)	1.4×10^{-5}	0.04	3.5×10^{-6}	0.01	4
Sandy Silt/Clayey Sand Blanket (ML, SC)	1.9×10^{-4}	0.56	4.9×10^{-5}	0.14	4
Silty Sand/Clayey Sand Aquifer (SM, SC)	3.5×10^{-4}	1	3.5×10^{-5}	0.1	10
Sand Aquifer (SP, SW)	3.5×10^{-3}	10	3.5×10^{-4}	1	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-33 as follows.

Table 7-33 Random Variables – Cherokee Canal

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	8	5	63
Permeability Ratio (k_f/k_b)	417	409	98
Foundation Sand thickness (d)	9	5	56

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-34. For this reach, the levee cohesion was varied instead of the foundation cohesion. The foundation consists mostly of silt and sandy silt soils with low to no cohesion.

Table 7-34 Stability Random Variables – Cherokee Canal

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	28	1	5
Levee Unit Weight γ (pcf)	120	6	5
Levee Cohesion c (psf)	150	17	11.5
Blanket Layer Unit Weight γ(pcf))	115	6	5
Blanket Layer Φ(degrees)	29	3	11.5

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-11. The underseepage performance mode accounts for almost all of the risk for this levee reach, although the risk is lower than other reaches in the study. This is consistent with the low levee height and the lack of poor underseepage performance history with this portion of levee.

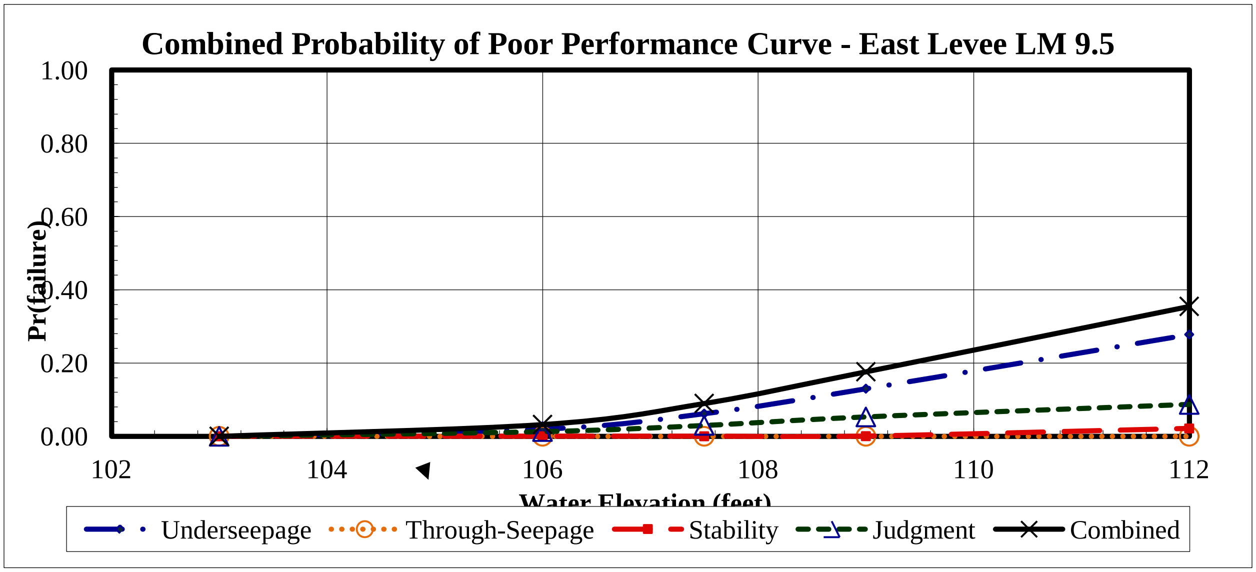

Figure 7-11 Combined R&U, Cherokee Canal

7.13 Wadsworth Canal (right)

There are no existing soil borings along the right levee of the Wadsworth Canal. This levee was added to the Feasibility Study recently at the request of the local sponsor, and this levee is included in the NULE program, which has not had explorations conducted. This levee only protects agricultural land – the small town of Sutter is located on high ground at the base of the Sutter Buttes – so economic damages from a breach would be insufficient to justify Federal interest in modifying or strengthening the levee. Due to the lack of potential economic damages and the time/funding restraints of the 3-3-3 Pilot Study, we decided to base the fragility curve on existing borings on the left levee of the Wadsworth Canal. The canal is about 300 feet wide (levee crest centerline to levee crest centerline).

Reach Selection: One reach, from PLM 0.00 to 4.66, was used for this levee, due to the lack of subsurface data.

Selection of Cross-Section: The cross section at PLM 0.50 was selected to represent this reach. This levee increases in height from about 5 feet at the upstream end to 20 feet at the downstream end, and the left bank levee has experienced seepage and boils during floods at the downstream end.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. Since this index point is located downstream of the index point on the left side of the canal, the underseepage statistical parameters for this reach were determined using more borings within the cutoff wall extent on the left side. The thickness of the blanket varies between 4 and 10 feet and consists of clay, silt, sandy clay, and sandy silt, overlaying a sand, silty sand, and clayey sand aquifer 3-28 feet thick. The borings used for the R&U analyses are shown in the analyses spreadsheet and the permeabilities are summarized in Table 7-35.

Table 7-35 Permeabilities, Wadsworth Canal (Right)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay Blanket (CL)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silt Blanket (ML)	$4.9x10^{-5}$	0.14	$1.2x10^{-5}$	0.035	4
Silty Sand Blanket (ML)	$4.9x10^{-4}$	1.4	$1.2x10^{-4}$	0.35	4

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Silty Sand Aquifer (SM)	$4.9x10^{-4}$	1.4	$4.9x10^{-5}$	0.14	10
Sand with Silt/Clay Aquifer (SP-SC, SP-SC)	$4.9x10^{-3}$	14	$4.9x10^{-4}$	1.4	10
Sand Aquifer (SP)	$9.8x10^{-3}$	28	$9.8x10^{-4}$	2.8	10

The standard deviation and coefficient of variation for the permeability ratios, blanket thickness and sandy aquifer thickness are summarized in Table 7-36 as follows.

Table 7-36 Random Variables – Wadsworth Canal (Right)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	7	2	29
Permeability Ratio (k_f/k_b)	1244	1219	98
Foundation Sand thickness (d)	13	11	85

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-37.

Table 7-37 Stability Random Variables – Wadsworth Canal (Right)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	32	4	13

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Cohesion c (psf)	10	4	40
Levee Unit Weight γ (pcf)	125	9	7
Blanket Layer Φ(degrees)	28	4	13
Blanket Layer Cohesion c (psf)	100	40	40

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-12. The underseepage performance mode accounts for almost all of the risk for this levee reach. This is consistent with the thin blanket layer compared to the levee height near the index point and the history of excessive seepage and boils in the downstream part of this reach.

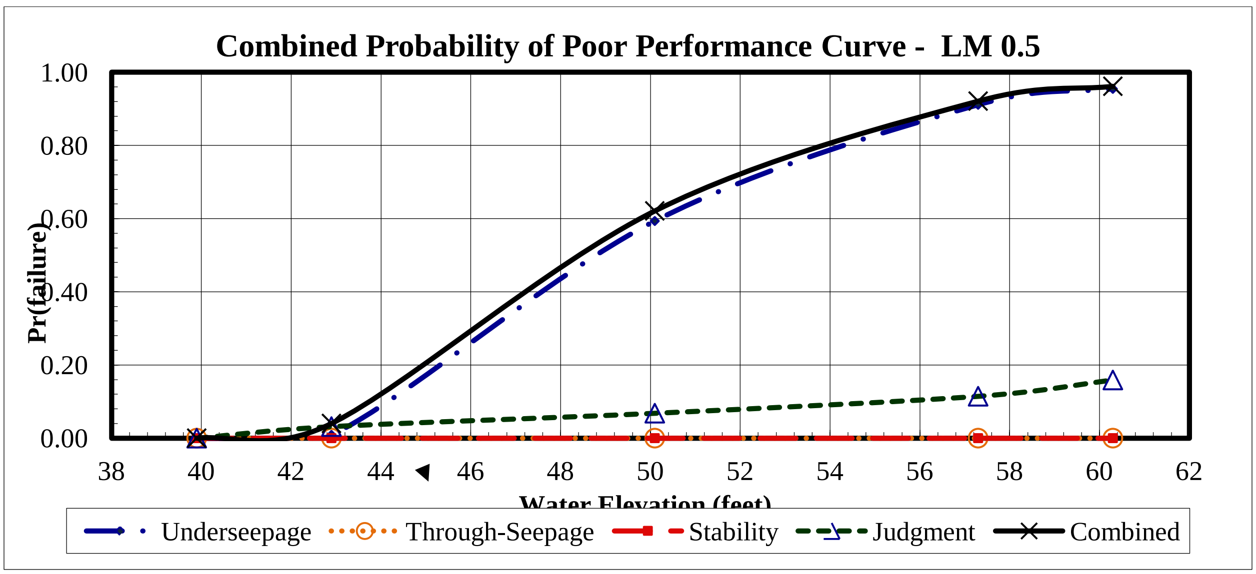

Figure 7-12 Combined R&U, Wadsworth Canal (Right)

7.14 Sutter Bypass (upstream)

There are no existing soil borings along the left levee of the Sutter Bypass upstream of Wadsworth Canal. This levee was added to the Feasibility Study recently at the request of the local sponsor, and this levee is included in the NULE program, which has not had explorations conducted. This levee only protects agricultural land – the small town of Sutter is located on high ground at the base of the Sutter Buttes – so any economic damages from a breach would be insufficient to justify Federal interest in modifying or strengthening the levee. Due to the lack of potential economic damages and the time/funding restraints of the Pilot Study, we decided to base the fragility curve on existing borings on the Sutter Bypass downstream of the Wadsworth Canal.

Reach Selection: One reach, from PLM 0.00 to 4.31, was used for this levee, due to the lack of subsurface data.

Selection of Cross-Section: The cross section at PLM 4.0 was selected to represent this reach. The levee height and levee geometry do not vary significantly over this reach, and this cross section is just upstream of the Wadsworth Canal, not far from the closest soil boring on the Sutter Bypass downstream of the Wadsworth Canal.

Underseepage Reliability Analysis: The permeabilities and the variation of the permeability ratio, blanket thickness, and the thickness of the aquifer were based on borings in the vicinity of the index point. Since there are no soil borings in this levee reach, the same borings, permeabilities, and statistical parameters were used for this reach as for the Sutter Bypass (Upper) Reach. The permeabilities are summarized in Table 7-38 and the random variables are shown on Table 7-39 (identical to Tables 7-5 and 7-6 respectively).

Table 7-38 Permeabilities, Sutter Bypass (Upstream)

Material	Horizontal permeability		Vertical Permeability		Permeability Ratio K_h/K_v
	cm/sec	ft/day	cm/sec	ft/day	
Clay/Sandy Clay Blanket (CL)	$9.8x10^{-6}$	0.028	$2.5x10^{-6}$	0.007	4
Silty Sand Blanket (SM)	$4.2x10^{-4}$	1.2	$1x10^{-4}$	0.3	4
Silty Sand Aquifer(SM)	$1x10^{-3}$	3	$1x10^{-4}$	0.3	10
Sand with Silt Aquifer (SP-SM)	$3.5x10^{-3}$	10	$3.5x10^{-4}$	1	10

Table76-39 Random Variables – Sutter Bypass (Upstream)

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket Thickness (z)	10	4	40
Permeability Ratio (k_f/k_b)	725	650	90
Foundation Sand thickness (d)	10	6	60

Stability Reliability Analysis: The expected values of the soil properties, the standard deviation, and the coefficient of variation used in the stability risk analysis are shown in Table 7-40. The stability parameters used for this reach were based solely on the closest boring to the index point, whereas the stability parameters used for the Sutter Bypass (Upper) Reach were based on borings further downstream, closer to that index point.

Table 7-40 Stability Random Variables – Sutter Bypass (Upstream)

Parameter	Expected Value (mean)	Standard Deviation	Coefficient of Variation (%)
Levee Φ(degrees)	28	3	12
Levee Unit Weight γ (pcf)	115	6	5
Blanket Layer Cohesion c (psf)	200	66	33
Blanket Layer Unit Weight γ (pcf)	115	6	5
Blanket Layer Φ(degrees)	28	3	12

Probability of Failure Curve: The combined fragility curve is shown in Figure 7-13. The underseepage performance mode accounts for almost all of the risk for this levee reach at low water surface elevations; underseepage and stability both contribute to the risk at high water elevations. Even though the same random variables were used for this reach as for the Sutter Bypass (Upper) Reach, the underseepage curve is slightly different because the levee at this index point is about 6 feet shorter than the levee at the Sutter Bypass (Upper) index point. This is consistent with the thin blanket layer compared to the levee height near the index point.

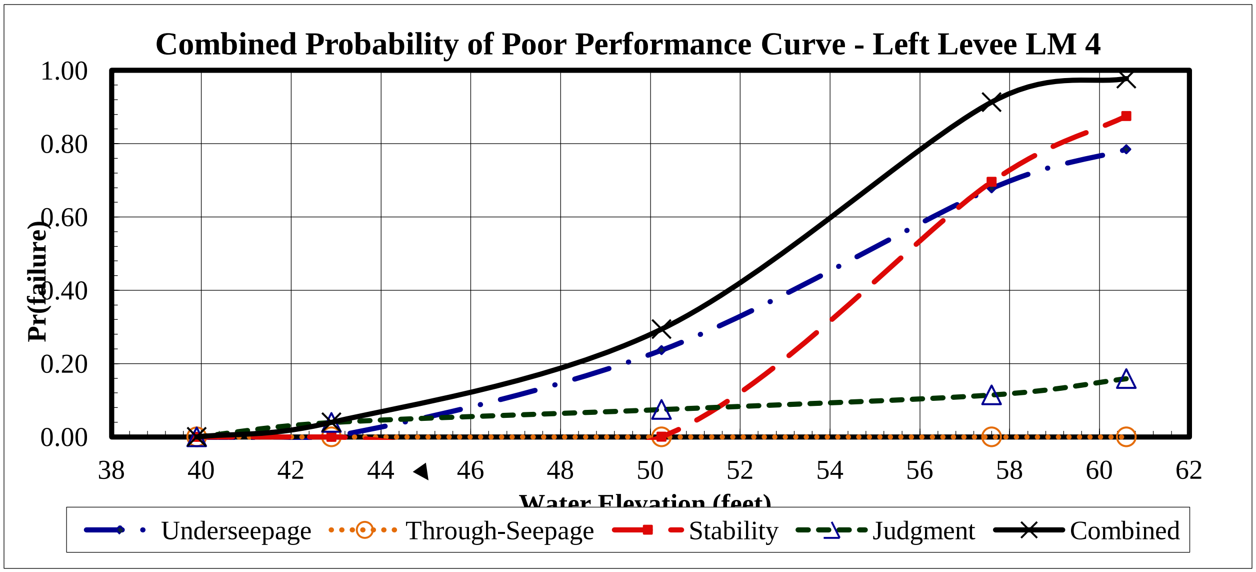

Figure7-13 Combined R&U, Sutter Bypass (Upstream)

8 GEOTECHNICAL ASPECTS OF FEASIBILITY STUDY ALTERNATIVES

8.1 General

Eight draft alternatives for reducing the flood risk within the SBFS system levees were developed as part of the Feasibility Study. The plan formulation process, including description of the draft array of alternatives, screening of alternatives, and the recommended Tentatively Selected Plan (TSP), is covered in detail in Chapter 3 of the main Feasibility Study report. During the screening of the draft array of alternatives, geotechnical recommendations for seepage and stability remediation for fix-in-place alternatives and seepage controls for non-fix in place alternatives (e.g. new ring levees, setback levees, etc.) were developed based in large part using engineering judgment. This Class 4 (reconnaissance level) approach was used to equalize the potential differences between seepage controls recommended for segments with existing geotechnical data, and those that had no subsurface data (non-fix-in-place measures). The approach assumed that cutoff walls were the primary method for seepage control, and the design of the measures (e.g. length, depth, percentage of reach, etc) was selected using judgment and the principal of most likely minimum and maximum for each value. After identifying a range, a expected mean value was selected again using judgment based on experience. This approach was used for evaluation of the draft array of alternatives only, and a memorandum documenting this action is in Enclosure H of this report. The design of the final array of alternatives was based on a Class 3 (feasibility level) conventional approach using existing subsurface explorations and deterministic seepage and stability analyses.

The remainder of this chapter in the Geotechnical Appendix to the Feasibility Study discusses the geotechnical aspects of the final array of alternatives. The final array of alternatives is:

- Alternative SB-1: No action
- Alternative SB-7: Fix-in-Place Feather River Levees: Sunset Weir to Laurel Avenue
- Alternative SB-8: Fix-in-Place Feather River Levees: Thermalito Afterbay to Laurel Avenue

8.2 Minimum Levee Design Criteria

Minimum levee design criteria from four sources (EM 1110-2-1913, Sacramento District Geotechnical Levee Practice, DWR Urban Levee Design Criteria, and the Code of California Regulations (CCR) Title 23 Division 1) is shown on Plate 10. The criteria are slightly different between the sources; in summary, the requirements are for a crest width of 10-20 feet, sideslopes of 2H:1V to 3H:1V, a landside easement of 10-20 feet, and a waterside easement of 15 feet. The Sacramento District Geotechnical Levee Practice and DWR Urban Levee criteria are for newly-constructed levees; exceptions may be allowed on a case-by-case basis for the reconstruction/improvement of existing levees. The Sacramento District allows a narrower crest width for existing levees that have improvements constructed to address seepage and stability concerns, although we generally will not go narrower than 15 feet to allow for 2-way pickup truck traffic on the levee crest. The Sacramento District also will accept whatever landside or waterside easement the local sponsor has. In cases where there is no existing landside easement, we require a minimum easement of 10 feet in urban areas. In conjunction with the Sacramento District Levee Safety Program Manager, the following minimum levee template criteria were adopted for this Feasibility Study:

- Crest width: 15 feet
- Landside slope: 2H:1V
- Waterside slope: 3H:1V where levee is being reshaped or relocated; existing slope at other locations
- Landside easement: 15 feet or the existing easement
- Waterside easement: 15 feet

The existing landside and waterside easements along the FRWL are not known at this time. Sponsors/LMAs frequently do not have records of their easements, and experience on other projects shows the work to determine the existing easements can take up to a year to complete. The Feasibility Study Real Estate maps were produced assuming a 15-foot waterside easement over the entire FRWL, a 15-foot landside easement in the urban area of Yuba City (between SBFCA stations 821+00 to 1135+00), and a 20-foot landside easement outside of Yuba City, except at one location where there is an existing 10-foot bench between the landside levee toe and the Sutter Butte Canal. At that location, a 10-foot landside easement is shown on the Real Estate Maps. The existing easements will be researched during the Planning and Engineering Design (PED) phase to develop a record of the existing easements and determine what easements need to be purchased.

8.3 Alternative SB-1: No Action

Under the No Action alternative, the existing condition of the levees as described in Chapters 1 and 7 of this report will continue into the future. The FRWL will continue to experience seepage-related distress during future flood events. Floodfighting will continue to be an expense for the LMAs and PL 84-99 rehabilitation assistance will continue to be an expense for the Federal Government (as long as the levees are active in the program). Individuals and critical infrastructure within the SBLS will remain at high risk from future flood events.

8.4 Alternatives SB-7 and SB-8 (in Relation to the SBFCA FRWL Improvement Project)

8.4.1 General

Due to the time and funding constraints of the Pilot Study process, the Feasibility Study Project Delivery Team (PDT) relied heavily on the SBFCA FRWL Improvement Project design reports for the proposed improvements, plans, quantity take-offs, and cost estimates for developing the Feasibility level design and cost estimate for Alternatives SB-7 and SB-8. Currently the SBFCA FRWL Improvement Project is at the 100% design level for the Contract C area between Shanghai Bend and Live Oak (LD 1 PLM 11.08 to MA16 PLM 2.87, SBFCA station 844+00 to 1625+00). The areas between Laurel Avenue and Shanghai Bend (MA3 PLM 3.64 to LD 1 PLM 10.66, SBFCA station 180+00 to 832+00), and Live Oak to Thermalito Afterbay outlet channel (MA 7 PLM 0.00 to Hamilton Bend PLM 1.20, SBFCA station 1625+00 to 2367+00) are at the 65% design level. The Feasibility Study design and cost estimate are based on the SBFCA 65% design submittal; the 90% design for the Contract C area was submitted in October 2012, and the Feasibility Study effort needed to get underway in July 2012 to stay on schedule. While design modifications between 65% and 90% submittals can and do occur, a 65% design submittal is more detailed than the usual Feasibility-level design, and the contingency added to the Feasibility Study cost estimate is expected to account for changes between the 65% design and the final design.

8.4.2 Remediation Measures

Where the existing levee meets the geotechnical seepage and stability criteria, due to either an existing cutoff wall or levee geometry/subsurface conditions, no remediation is needed. Where remediation is needed, cutoff walls are the primary feature of the SBFCA project for remediating geotechnical deficiencies of the existing FRWL for the following reasons:

- Cutoff walls are highly effective when constructed properly
- Cutoff walls do not require the acquisition of additional permanent real estate
- Cutoff walls do not require maintenance once constructed
- Cutoff walls constructed by the conventional open-trench method are cost comparable to landside seepage berms when the costs of additional permanent real estate and environmental mitigation associated with seepage berms are included
- Cutoff walls have minimal long-term environmental impact primarily due to their location within the existing levee footprint.

Early in their design process, the SBFCA design team evaluated two primary measures for remediation of the FRWL. In general, the measures were a fully-penetrating soil-bentonite (SB) cutoff wall and a shallow SB cutoff wall combined with a seepage berm or relief wells (both alternatives include a partial levee degrade to obtain the needed working platform width). A reach-by-reach cost comparison between the two measures showed a fully-penetrating SB cutoff wall was the least cost measure for most reaches (see Chapters 8 and 9 and Table 9-1 of the main Pre-Design Formulation Report for more information on the SBFCA cost analysis). However, site conditions dictated selection of a different measure for some reaches or portions of reaches. Those conditions are summarized in the following bullets.

- A cutoff wall with a full levee degrade is proposed where the levee has a severe burrowing rodent infestation just north of Yuba City and to prevent having to use the more expensive Deep Mixing Method (DMM) for cutoff wall construction due to depth just north of Shanghai Bend.

- Jet grout cutoff walls are proposed at locations where it is not practical to construct a conventional SB cutoff wall (i.e. bridges, railroad crossings, and the Yuba City Water Treatment Plant).
- Seepage berms by themselves are used in the northernmost end of the FRWL because a conventional SB cutoff wall may not be constructible through the cobble levee.
- Partially penetrating cutoff walls combined with seepage berms or relief wells are used in the southern end of the FRWL because fully-penetrating cutoff walls would need to be too deep to be cost-effective.

The Feasibility Study PDT modified the SBFCA 65% plans to address some minor differences between the SBFCA 408 project and the recommended Feasibility project (see section 8.5 of this report for details of the modifications). The modified SBFCA 65% plans are given in Appendix M of this report, but are provided as a separate file. Table 8-1 lists the proposed remedial measures from the SBFCA 65% plans from downstream to upstream. There are overlaps at the transitions between remedial measures (for example, an SB cutoff wall to a jet grout cutoff wall).

Table 8-1 Proposed Remedial Measures, SBFCA FRWL Improvement Project, 65% Plans

Location (PLM)	Location (SBFCA Stationing)	Remedial Measures
MA3 PLM 3.26 to MA3 PLM 4.19	180+00 to 230+00	Degrade levee to approximately half-height, SB cutoff wall (depth 27 ft), reconstruct levee, seepage berm, culvert replacement
MA3 PLM 4.19 to LD1 PLM 3.24	230+00 to 451+00	Degrade levee to approximately half-height, SB cutoff wall (depth 33 to 88 ft), reconstruct levee, culvert removal, culvert replacement
LD PLM 3.24 to LD1 PLM 3.71	451+00 to 476+00	Degrade levee to approximately half-height, SB cutoff wall (depth 41.5 ft), reconstruct levee, seepage berm, culvert removal
LD1 PLM 3.71 to LD1 PLM 4.61	476+00 to 513+50	No work (Star Bend setback levee/cutoff wall) *(Note: See paragraph 8.4.3 for additional information on the Star Bend area)*
LD1 PLM 4.61 to LD1 PLM 10.66	513+50 to 832+00	Degrade levee to approximately half-height, SB cutoff wall (depth 40 to 73 ft), reconstruct levee, new relief wells (sta 545+00 to 570+00 only), culvert removal, culvert replacement
LD1 PLM 10.66 to LD1 PLM 11.08	832+00 to 844+00	No work (Shanghai Bend setback levee/cutoff wall)
LD1 PLM 11.08 to LD1 PLM 12.21	844+00 to 898+00	Degrade entire levee, SB cutoff wall (depth 85 ft), reconstruct levee, rehabilitation or replacement of existing relief wells, culvert replacement

Location (PLM)	Location (SBFCA Stationing)	Remedial Measures
LD1 PLM 12.21 to LD1 PLM 12.70	898+00 to 923+75	Degrade levee to approximately half-height, SB cutoff wall (depth 44'), reconstruct levee, culvert replacement
LD1 PLM 12.70 to LD1 PLM 14.34	923+75 to 1006+04	Encroaching structure demolition (existing Yuba City cutoff wall)
LD1 PLM 14.34 to LD1 PLM 14.38	1006+04 to 1007+90	Jet grout cutoff wall (depth 41') at railroad and Fifth St. bridges
LD1 PLM 14.38 to LD1 PLM 14.61	1007+90 to 1023+70	Culvert removal (existing Yuba City cutoff wall)
LD1 PLM 14.61 to LD1 PLM 14.70	1023+70 to 1028+30	Stability berm, fill in unused tunnel at 10th Street bridge
LD1 PLM 14.70 to LD1 PLM 15.64	1028+30 to 1077+85	Culvert replacement (existing Yuba City cutoff wall)
LD1 PLM 15.64 to LD1 PLM 15.98	1077+85 to 1096+00	Degrade levee to approximately half-height, SB cutoff wall (depth 40 ft), reconstruct levee, culvert replacement
LD1 PLM 15.98 to LD1 PLM 16.03	1095+80 to 1098+30	Jet grout cutoff wall (depth 40 ft) at water treatment plant, culvert replacement
LD1 PLM 16.03 to LD1 PLM 16.63	1098+10 to 1129+98	Degrade levee to approximately half-height, SB cutoff wall (depth 40 to 77 ft), reconstruct levee, culvert replacement
LD1 PLM 16.62 to LD1 PLM 16.64	1129+49 to 1130+67	Jet grout cutoff wall (depth 77 ft), stoplog structure installation at railroad track crossing
LD1 PLM 16.64 to LD9 PLM 6.14	1130+20 to 1455+00	Degrade levee to approximately half-height, SB and DMM cutoff wall (depth 29 to 120 ft), reconstruct levee, culvert removal, culvert replacement *(Note: Alternative SB-7 ends at station 1433+83)*
LD9 POM 6.14 to MA 16 PLM 0.01	1455+00 to 1461+00	Degrade entire levee, SB cutoff wall (depth 35 ft), reconstruct levee
MA16 PLM 0.01 to MA16 PLM 3.15	1461+00 to 1625+00	Degrade levee to approximately half-height, SB cutoff wall (depth 26 to 62 ft) , reconstruct levee, culvert removal, culvert replacement
MA16 PLM 3.15 to MA16 PLM 4.06	1625+00 to 1673+00	Culvert replacement (short levee, no adjacent canal)
MA 16 PLM 4.06 to MA7 PLM 1.83	1673+00 to 1769+31	Degrade levee to approximately 20 to 50% of height, SB cutoff wall (depth 26 to 48 ft), reconstruct levee with shallower sideslopes, culvert replacement

Location (PLM)	Location (SBFCA Stationing)	Remedial Measures
MA7 PLM 1.83 to MA7 PLM 2.66	1769+31 to 1813+33	Culvert replacement (short levee, no adjacent canal)
MA7 PLM 2.66 to MA7 PLM 4.34	1813+33 to1900+50	Degrade levee to approximately 20 to 50% of height, SB cutoff wall (depth 18 to 73 ft), reconstruct levee, culvert removal, culvert replacement
MA7 PLM 4.33 to MA7 PLM 4.40	1900+00 to 1904+00	Jet grout cutoff wall (depth 78 ft) at Gridley bridge
MA7 PLM 4.39 to MA7 PLM 11.85	1903+50 to 2292+00	Degrade levee to approximately half-height, SB and DMM cutoff wall (depth 17 to 90 ft), reconstruct levee, culvert removal, culvert replacement
MA7 PLM 11.81 to MA7 PLM 12.06	2290+00 to 2303+00	Seepage berm (170 ft wide)
MA7 PLM 12.06 to Hamilton Bend PLM 0.51	2303+00 to 2330+75	Culvert removal (short, wide levee)
Hamilton Bend PLM 0.51 to Hamilton Bend PLM 1.20	2330+75 to 2367+00	Seepage berm (100-120 ft wide), culvert removal

The remainder of this section of this report gives locations only in SBFCA stationing, since PLMs are not shown in the SBFCA project geotechnical reports and the 65% design plans.

8.4.3 <u>Star Bend</u>

Star Bend is a 90-degree bend in the Feather River and the FRWL located approximately 8.7 miles south (downstream) of downtown Yuba City (SBFCA stations 478+50 to 512+00). As stated in paragraph 1.2.4.3 of this report, a setback levee with a 40 to 65-foot deep SB cutoff wall through the foundation was constructed at Star Bend by the LMA in 2008. The project sponsor would like to obtain reimbursement credit for the setback levee/cutoff wall under the Feasibility Study. Since the construction of setback levees with cutoff walls in the foundation is more expensive than fixing existing levees in place, the SBFS did not include a setback levee at Star Bend in any of the Study alternatives. The sponsor will only get reimbursement credit based on the cost of fixing the Star Bend levee in place by degrading the levee to approximately half it's height and constructing an SB cutoff wall with a depth of 65 feet.

8.5 Feasibility Study Modifications to SBFCA FRWL Improvement Project 65% Design Submittal

The SBFCA FRWL project design is being thoroughly reviewed at every phase for the 408 modification process. The design A/E firms (Wood Rodgers, HDR, MHM, Blackburn Consulting, and URS) do internal quality control review, and additional review is conducted by SBFCA, DWR, the Central Valley Flood Protection Board, and the Sacramento District. An Independent Panel of Experts (IPE) provides external and Safety Assurance Review (SAR). For the Feasibility Study, the PDT reviewed the 65% submittal to determine if it includes any improvements that are beyond Federal interest (and would be 100% sponsor cost) and if any improvements do not meet minimum study criteria. For the geotechnical discipline , nothing in the submittal was determined to be beyond Federal interest. However, three items, not directly geotechnical but related to levee safety, do not meet minimum Corps requirements. Those three items are discussed below.

8.5.1 Vegetation Removal

The Corps conditions for acceptance of a major/minor modification under the 408 process do not require all Corps requirements to be met. The Corps will approve proposed modifications under 408 as long as the levee integrity will not be reduced and any levee improvements may be credited to the Sponsor as long as the improvements are in agreement with the Feasibility Study approved improvement plan. The FRWL currently has mature trees on the both the levee slopes and within 15 feet of both the landside and waterside levee toes, with the majority of the trees being within 15 feet of the toes. The SBFCA plans include removal only of trees required to do the project construction. The Feasibility Study proposed plan for ETL 1110-2-571 compliance is for complete removal of all vegetation from the vegetation free zone as shown in the ETL. The PDT developed a cost estimate for removal of the woody vegetation from the levee vegetation free zone. Tree removal (and associated mitigation) costs were based on an estimate done by SBFCA. The estimated cost of ETL 1110-2-571 compliance by removal of woody vegetation is within the overall Feasibility Study cost contingency. A memorandum documenting this action is in Enclosure I of this report.

8.5.2 Landside Easement at the Sutter Butte Canal

At four locations (about 3.3 miles total length), the Sutter Butte Canal is adjacent to the landside levee toe, without a landside access easement. These locations are, from downstream to upstream:

- Station 1430+00 to 1449+00
- Station 1610+50 to 1623+00
- Station 1675+00 to 1765+80
- Station 1904+00 to 1957+00

The upstream location has a small landside bench, approximately 10 feet wide, between the levee toe and the canal. This bench can provide a 10-foot minimum required landside easement with some minor regrading. The PDT evaluated four options to obtain a landside easement at the other three locations. The options are:

1. Cut a horizontal bench into the landside levee toe at the cutoff wall degrade elevation and construct a retaining wall to replace the lower levee slope
2. Replace the levee with a floodwall
3. Realign the levee 15 feet to the waterside
4. Relocate the canal away from the levee toe

Option 3 was selected for the two downstream locations. At the third location (station 1675+00 to 1765+80), the levee is adjacent to the river at the upstream end of the location (station 1753+00 to 1765+80). Option 3 was selected for the downstream portion of this location (station 1675+00 to 1753+00) and option 4 was selected for the upstream portion of this location (station 1753+00 to 1765+80). Typical cross sections were prepared for all four locations and cost estimates were prepared for the selected option at each location. The Civil Design Appendix contains additional discussion of this issue.

8.5.3 Landside Slope Protection for Levee Superiority

The definition of levee superiority per EC 1110-2-6066 (Design of I-Walls, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Since alternatives SB-7 and SB-8 are based on an existing levee profile, the design top of levee was reviewed relative to the modeled mean water surface profiles to determine the likely initial overtopping location. The initial overtopping in the alternative SB-7 footprint will likely occur between SBFCA stations 547+00 to 604+60. The initial overtopping in the footprint only covered by alternative SB-8 will likely occur between SBFCA stations 1582+00 to 1601+00. These locations are in non-urbanized areas and initial overtopping is estimated to occur between the mean 0.5% (1/200) ACE and 0.2% (1/500) ACE events. Within these 1-mile sections, the landside levee slope will be covered with an anchored High Performance Turf Reinforcement Mat (HPTRM). This design will increase the erosion resistance of the landside levee slope at the initial overtopping locations.

8.6 Overview of SBFCA Geotechnical Analysis

8.6.1 Existing Conditions

The Geotechnical Analyses for Pre-Design Formulation Report (PFR) (Appendix D of the PFR, dated August 2011), includes seepage and slope stability analyses for the existing condition of the FRWL. The PFR divided the levee into 41 reaches. One cross section was analyzed per reach. The analyses were conducted to determine which portions of the levee require remediation and to evaluate the effectiveness of various remedial measures. Analyses were conducted at two water surfaces; the design water surface (DWS) and the hydraulic top of levee (HTOL). The DWS is defined as the 0.5% (1/200) Annual Chance of Exceedance (ACE) upstream of station 461+00 (located just downstream of the Star Bend setback levee) and the 1% (1/100) ACE downstream of station 461+00. (California state law, enacted in 2007 (SB 5), requires urban levees (defined as levees in areas of 10,000 people or more) to be designed to the 0.5% (1/200) ACE). Note that the DWS used in the SBFCA geotechnical design is not the project authorized water surface as described in the Hydraulic Appendix. The hydraulic top of levee (HTOL) is defined as the lowest of:

- The 0.5% (1/200) ACE plus 3 feet water surface upstream of station 461+00 and the 1% (1/100) ACE plus 3 feet water surface downstream of station 461+00
- The 0.2% (1/500) ACE water surface
- The levee crest elevation

One foot was added to the appropriate water surface elevations to evaluate remediation measures. The DWS+1 foot elevation was compared with the project authorized water surface at selected locations along the Feather River and the elevations agree within a foot.

For each cross section, analyses were conducted in the following order:

- Seepage at the DWS
- Seepage at the HTOL
- Landside slope stability at the DWS
- Landside slope stability at the HTOL
- Rapid Drawdown slope stability

The seepage analyses were done first due to the performance history of the FRWL, which has experienced extensive seepage-related distress but little stability-related distress during floods. If a section did not meet criteria for any analysis, it was determined to be in need of remediation, and the remaining existing conditions analyses were not conducted. Each section found to be in need of remediation was then re-analyzed with two remediation measures in place, generally a cutoff wall and a seepage berm.

Section 4.0 of the Geotechnical Analysis for the PFR describes the methodology used to select sections and parameters for seepage and stability analyses. Table 4-4 of the Geotechnical Analysis for the PFR gives seepage exit gradient criteria for the SBFCA design. Section 5.0 of the Geotechnical Analysis for the PFR discusses analysis results. Tables 5-1A through 5-41B of the Geotechnical Analysis for the PFR summarize the levee/foundation characterization and the seepage/stability analysis results for each reach. Those tables are included in Enclosure J of this report. Stick log profiles of the FRWL are given in Appendix A of the Geotechnical Analysis for the PFR. Analysis cross sections are shown in Appendix B of the Geotechnical Analysis for the PFR. The Geotechnical Analysis for the PFR is in Enclosure M of this report, but is provided as a separate file for ease of review. Most cross sections analyzed did not meet the project seepage criteria. This agrees with the deterministic and probabilistic analyses done for this study and the deterministic analyses done for the DWR ULE study, which show underseepage to be the primary contributor to poor performance of the FRWL. This also agrees with the performance history of extensive seepage-related distress on the FRWL.

8.6.2 With-Project Conditions

Two final geotechnical reports were submitted in October 2012; the Geotechnical Design Recommendations Report (GDRR) (2 volumes) and the Geotechnical Data Report (GDR). These reports cover the entire Feather River West Levee from the Sutter Bypass confluence upstream to the Thermalito Afterbay outlet channel. These reports are included in Enclosure M of this report, but provided as separate files. The final GDR includes results of all geotechnical explorations done under the SBFCA FRWL Improvement Project. The final GDRR includes a general description of the levees; past performance history; remedial measures constructed to date; seismic vulnerability analysis; and with-project seepage and slope stability analyses conducted to support the levee improvements as shown on the 65% project plans. Stick log profiles of the FRWL are given in Appendix A of the final GDRR. The stick log profiles with the final GDRR contain explorations conducted by the FRWL Improvement Project and the DWR ULE project subsequent to the submittal of the Geotechnical Analysis for the PFF. With-project seepage and slope stability analyses were conducted on one to six cross sections per SBFCA reach. Sections 6.0 and 7.0 of the final GDRR describe the methodology for the with-project seepage and slope stability analyses. Section 8.0 of the final GDRR describes the analysis results. Table 8-1 of the final GDRR summarizes results of those analyses, including sensitivity analyses conducted on several cross sections to evaluate the effects of different parameters (including blanket layer thickness, soil type, soil permeability, soil shear strength, seepage analysis boundary conditions, and cutoff wall tip elevation). This table is included as Enclosure K of this report. Appendix C of the final GDRR contains with-project seepage and slope stability analysis sections. The with-project seepage and slope stability analyses were conducted at the same water surface elevations used to evaluate remediation alternatives in the Geotechnical Analysis for the PFR (DWS+1 foot and HTOL+1 foot). All proposed alternatives for all design analysis sections meet Corps design criteria. The SBFCA project as designed meets all required Corps geotechnical design criteria.

Settlement analysis was not conducted for the FRWL Improvement Project. The FRWL is several decades old and will be rebuilt to the same height after degrade and cutoff wall installation.

8.6.3 Seismic Vulnerability Analysis

As stated in section 6.3 of this Geotechnical Appendix, the Sacramento District draft manual for seismic deformation evaluation of levees, which is currently under review for nationwide implementation, recommends designing for seismic loading only those levees loaded permanently by high water levels. Levees subject to intermittent loading (which includes all the levees in the SBLS) will only be evaluated for liquefaction potential and seismic deformation to inform sponsors of the risk level so they can identify and evaluate borrow sites for levee rebuilding after an earthquake, if necessary. Designing intermittently-loaded levees for seismic deformation is cost-prohibitive.

Seismic vulnerability analysis was conducted under both the DWR ULE and the SBFCA 408 projects. The ULE analysis covered all the levees within the SBLS, while the SBFCA analysis only covers the FRWL from Star Bend upstream to the Thermalito Afterbay outlet channel. The SBFCA 408 project analyzed more sections than the ULE project (44 versus 13, respectively) on the FRWL upstream of Star Bend because the SBFCA project is in design, while the ULE project is in the study phase. Both projects used a similar methodology for their analysis:

- A liquefaction triggering analysis was conducted using the higher of the average winter and average summer water surface elevations of the appropriate waterway. The ULE evaluation used peak ground accelerations (PGA) for the 100, 200, and 500-year earthquakes obtained from the ULE project guidance document, while the SBFCA analysis only used the 200-year earthquake PGA from the ULE guidance document.
- Post-earthquake slope stability analysis was then conducted, using undrained shear strengths for fine-grained soils and post-liquefaction undrained shear strengths for coarse-grained soils that were expected to liquefy. If the post-earthquake slope stability Factor of Safety was less than 1.0, the section was considered compromised and no additional analysis was done.
- Otherwise, a pseudo-static earthquake slope stability analysis was conducted using a seismic coefficient. If the pseudo-static slope stability Factor of Safety was greater than 1.0, then the section was considered probably uncompromised and no additional analysis was done.
- Otherwise, a Newmark-type deformation analysis was done to estimate horizontal displacement, from which vertical displacement and freeboard loss were estimated.

Based on the estimated vertical displacement, a seismic vulnerability classification (probably uncompromised, possibly compromised, probably compromised, or compromised) was determined. The ULE and SBFCA projects used slightly different criteria to determine the vulnerability classification. The ULE criteria is based on the amount of vertical deformation in feet and the remaining freeboard above a 50% (1/2) ACE water surface elevation and the SBFCA criteria is based on the amount of vertical deformation in percent of levee height and the remaining freeboard above a 10% (1/10) ACE water surface elevation. Table 8-2 compares the ULE and SBFCA seismic vulnerability classification criteria.

Table 8-2 Seismic Vulnerability Classification Criteria

Vertical Deformation		Significant Damage to internal structures (i.e. cutoff walls)	Remaining Freeboard for Post-Seismic Evaluation		Seismic Vulnerability Class
ULE (ft)	SBFCA (percent of landside levee height)		ULE (50% (1/2) ACE water surface) (ft)	SBFCA (10% (1/10) ACE water surface) (ft)	
<1	<5	No	>1	>1	Probably Uncompromised
1-3	<10	Possibly	>1	>1	Possibly Compromised
3-10	<20	Likely if existing	None	None	Probably Compromised
Unlimited (flow slide condition)	Unlimited, >10' (flow slide condition)	Yes	None	None	Compromised

Seismic vulnerability evaluation results from the SBFCA project are in Table 8-4 of the final GDRR and results from the ULE are in Tables 5-16 and 5-17 of the P1GER. SBFCA results are tabulated by reach while the ULE results are tabulated by a single station. The Sacramento District's geotechnical earthquake engineering expert reviewed the seismic vulnerability analyses performed for the ULE and SBFCA projects. His review is documented in a memorandum given in Enclosure L of this report. While there are minor differences between the ULE and SBFCA evaluations and the draft Corps of Engineers evaluation recommendations, the ULE and SBFCA evaluations are considered to be satisfactory. Table 8-3 of this report gives a comparison of the SBFCA and ULE seismic analysis results for reaches where both projects did an evaluation. The seismic vulnerability classifications are mostly the same between the two evaluations; the 3 differences are attributed to different soil borings being used for the two evaluations. As can be seen in Table 8-3 below and Table 8-4 of the final GDRR, some portions of the FRWL downstream of SBFCA station 1243+00 (located about 3 miles north of Yuba City) are predicted to experience significant deformation during a 200-year earthquake. The Sponsor is encouraged to identify emergency borrow area(s) to reconstruct the levee to a limited level of protection (10% (1/10) ACE) within 3 months after a significant earthquake.

Table 8-3 Comparison of SBFCA and ULE Seismic Vulnerability Class

SBFCA Reach	SBFCA Station Range	ULE Boring Location (SBFCA Station)	Seismic Vulnerability Class
8	598+87 to 654+75	626+31	Compromised (SBFCA)
			Possibly Compromised (ULE)
9	674+00 to 695+00	674+30	Probably Uncompromised (SBFCA and ULE)
13	845+00 to 927+00	861+17	Compromised (SBFCA and ULE)
15	954+00 to 968+00	959+03	Compromised (SBFCA and ULE)
18	1130+86 to 1136+00	1135+58	Possibly Compromised (SBFCA and ULE)
19	1213+85 to 1243+00	1229+81	Likely Compromised (SBFCA)
			Probably Uncompromised (ULE)
19	1213+85 to 1243+00	1238+43	Likely Compromised (SBFCA)
			Probably Uncompromised (ULE)
23	1503+83 to 1609+37	1508+51	Probably Uncompromised (SBFCA and ULE)
24	1609+37 to 1623+86	1615+61	Probably Uncompromised (SBFCA and ULE)
31	1902+00 to 1958+00	1907+37	Probably Uncompromised (SBFCA and ULE)
33	1989+00 to 2122+00	2076+95	Probably Uncompromised (SBFCA and ULE)
35	2182+00 to 2224+00	2187+00	Probably Uncompromised (SBFCA and ULE)
35	2182+00 to 2224+00	2212+09	Probably Uncompromised (SBFCA and ULE)

8.6.4 Conclusion

The Sacramento District recommends the SBFCA 408 project as shown on their 65% submittal plans (except for the items discussed in Section 8.4 of this report). The SBFCA geotechnical analysis is in conformance with Corps design requirements and procedures and we agree with their analysis and analysis results. The levee improvements shown on the SBFCA 65% submittal plans will perform as intended at the water surfaces used for the geotechnical analyses.

8.7 Borrow Sites

The SBFCA FRWL Improvement Project specifications include two primary material types: Type 1 levee fill, primarily used as a clay core for the reconstructed levee above the cutoff wall, and Type 2 levee fill, primarily used for shells for the reconstructed levee above the cutoff wall. Specifications for the two material types are as follows:

- Type 1 Levee Fill: USCS classification of CL, SC, or CH and; maximum particle size of 2 inches; minimum 35% by weight passing the #200 sieve; maximum liquid limit of 60; plasticity index between 12 and 40.
- Type 2 Levee Fill: Maximum particle size of 2 inches; minimum 12% by weight passing the #200 sieve; maximum liquid limit of 45.

Based on geotechnical investigations and standard practice, an approximately 20% increase should be applied to the total demand (to account for all material swell, loss and shrinkage during excavation, transportation and placement, respectively) when estimating the borrow amount needed. The SBFCA project team estimated the amount of levee degrade material that is suitable for reuse on a reach-by-reach basis. The approximate percentages of levee degrade material suitable for reuse as levee fill are shown in Table 8-4.

Table 8-4 Percentage of Levee Degrade Material Suitable for Levee Fill

SBFCA Reach	Percentage for Levee Core (Type 1)	Fraction	Percentage for Levee Shell (Type 2)	Fraction
2A-North	5	0.05	95	0.95
2B	5	0.05	95	0.95
3	5	0.05	95	0.95
4	5	0.05	95	0.95

SBFCA Reach	Percentage for Levee Core (Type 1)	Fraction	Percentage for Levee Shell (Type 2)	Fraction
5	5	0.05	95	0.95
6	5	0.05	95	0.95
7	40	0.4	60	0.6
8	0	0	85	0.85
9	0	0	55	0.55
10	0	0	70	0.7
11	0	0	100	1
12	NA	NA	NA	NA
13	0	0	95	0.95
14	NA	NA	NA	NA
15	NA	NA	NA	NA
16	NA	NA	NA	NA
17	0	0	100	1
18	15	0.15	85	0.85

SBFCA Reach	Percentage for Levee Core (Type 1)	Fraction	Percentage for Levee Shell (Type 2)	Fraction
19	30	0.3	70	0.7
20	0	0	100	1
21	0	0	100	1
22	15	0.15	85	0.85
23	0	0	90	0.9
24	0	0	100	1
25	0	0	100	1
26	0	0	100	1
27	80	0.8	20	0.2
28	15	0.15	85	0.85
29	NA	NA	NA	NA
30	0	0	95	0.95
31	30	0.3	70	0.7
32	0	0	100	1

SBFCA Reach	Percentage for Levee Core (Type 1)	Fraction	Percentage for Levee Shell (Type 2)	Fraction
33	0	0	100	1
34	0	0	100	1
35	0	0	100	1
36	0	0	100	1
37	0	0	100	1
38	0	0	100	1
39	NA	NA	NA	NA
40	60	0.6	0	0
41	60	0.6	0	0

While some of the levee soils removed during degrading will be re-used to reconstruct the levee, it is anticipated that borrow material will be needed to meet the levee fill material specifications. The SBFCA FRWL Improvement Project has identified potential borrow sites (Plate 11) and is currently in the process of sampling and testing the sites to ensure they meet material requirements.

8.8 Pipelines Crossing the Levee

There are numerous pipelines crossing the levee alignment, including electrical and telephone cables and various types of water lines (interior drainage gravity lines, interior drainage pump lines, water supply lines, and sewage disposal lines). Installation dates of the pipes vary between the 1930's and the early 2000's. Installation dates and details are not available for some of the pipes. Some of the pipes have not been maintained and some of them do not have a means of positive closure. The SBFCA design team evaluated each pipe based on installation date (known or unknown) and adherence to Corps criteria for pipelines crossing levees in EM 1110-2-1913. All pipes that are not currently being used will be removed. All active pipes known or suspected of being beyond their estimated service life will be removed and replaced to comply with Corps criteria, including routing above the design water surface for pressurized pipes and installing positive closure for all pipes. Pipes that are known to be recent installations will remain, although there may be minor modifications made to comply with Corps criteria. The Civil Design Appendix contains additional information on this issue.

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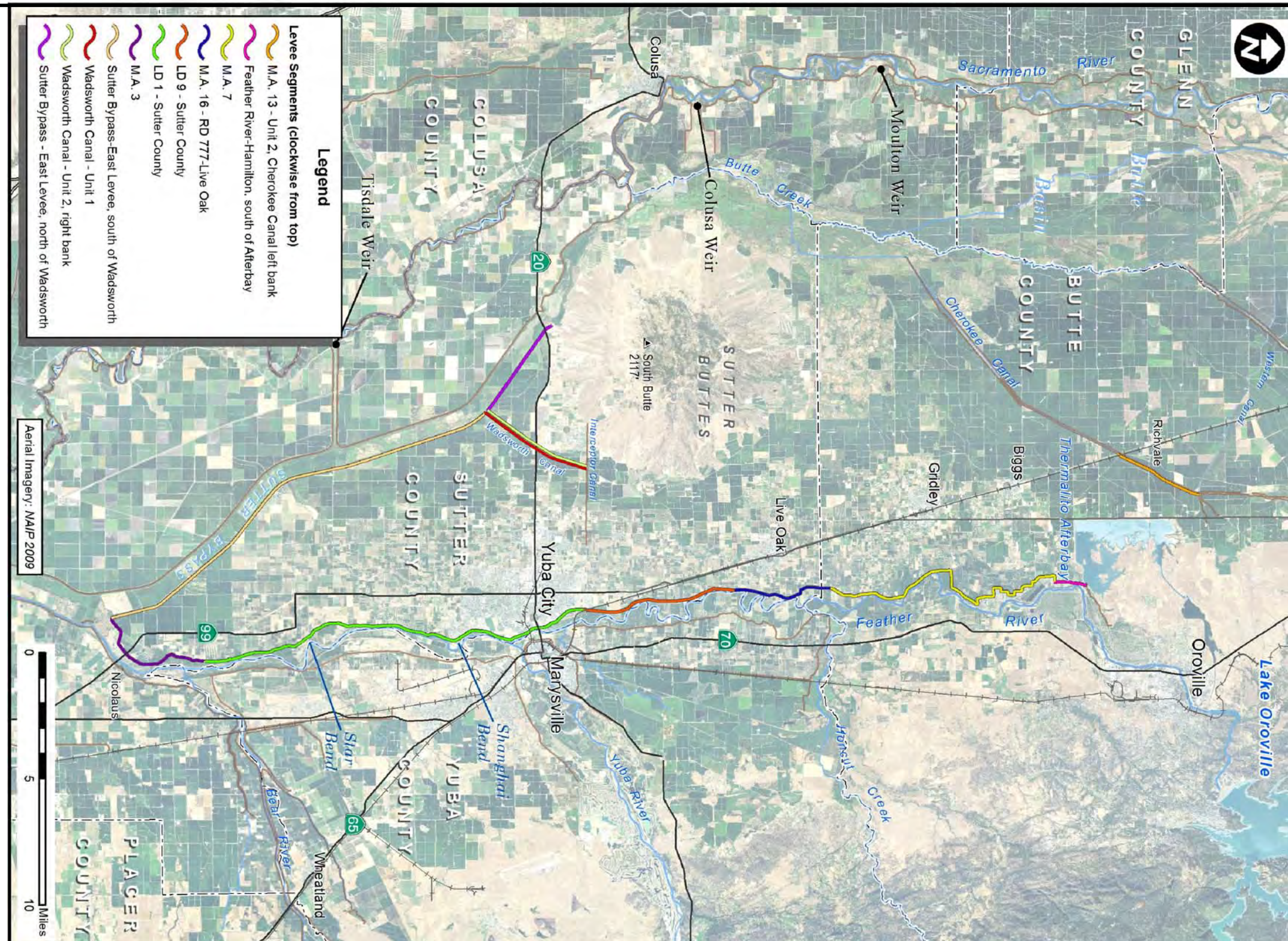
PLATES



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SEGMENT MAP

DATE:	SCALE:	
8/20/2012	1"=2500'	PLATE-1

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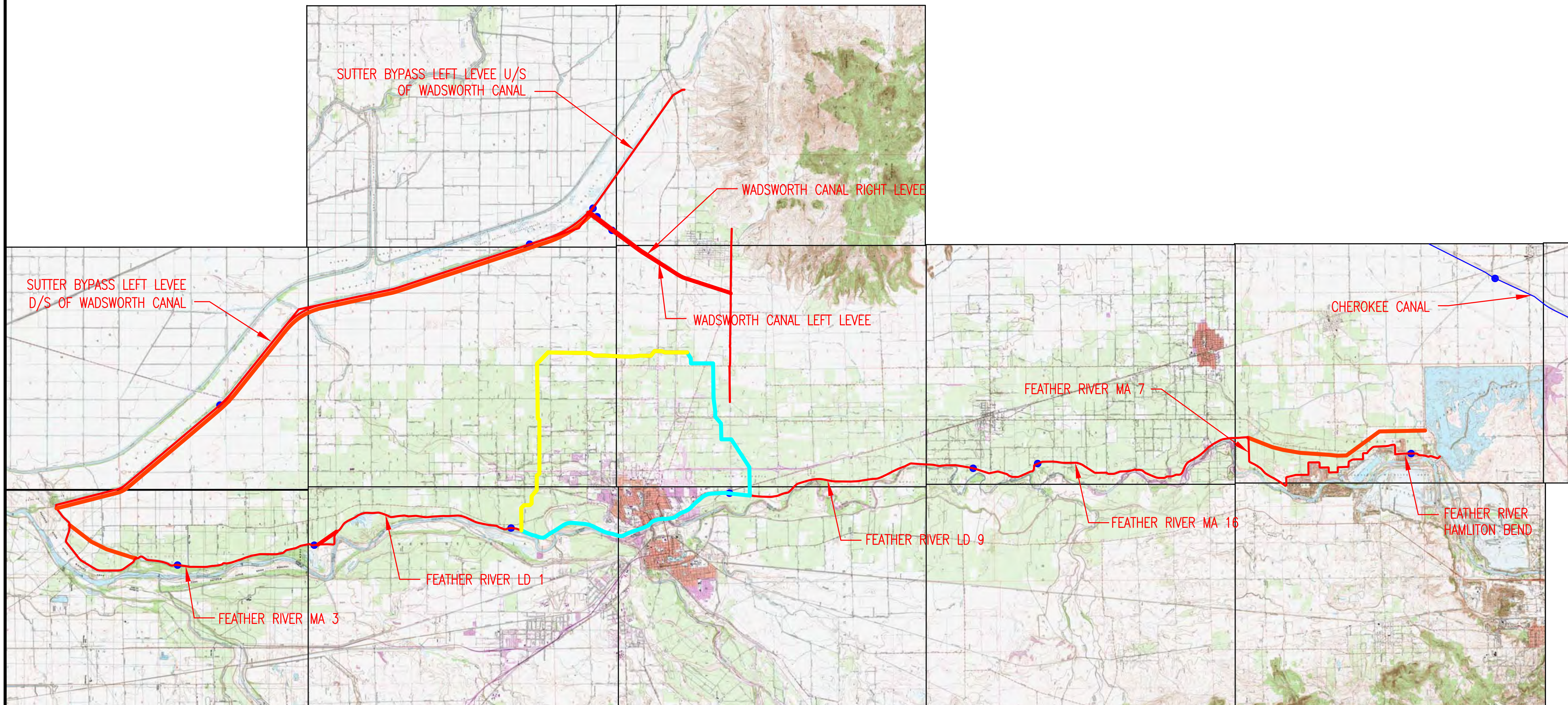

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OVERALL STUDY MAP

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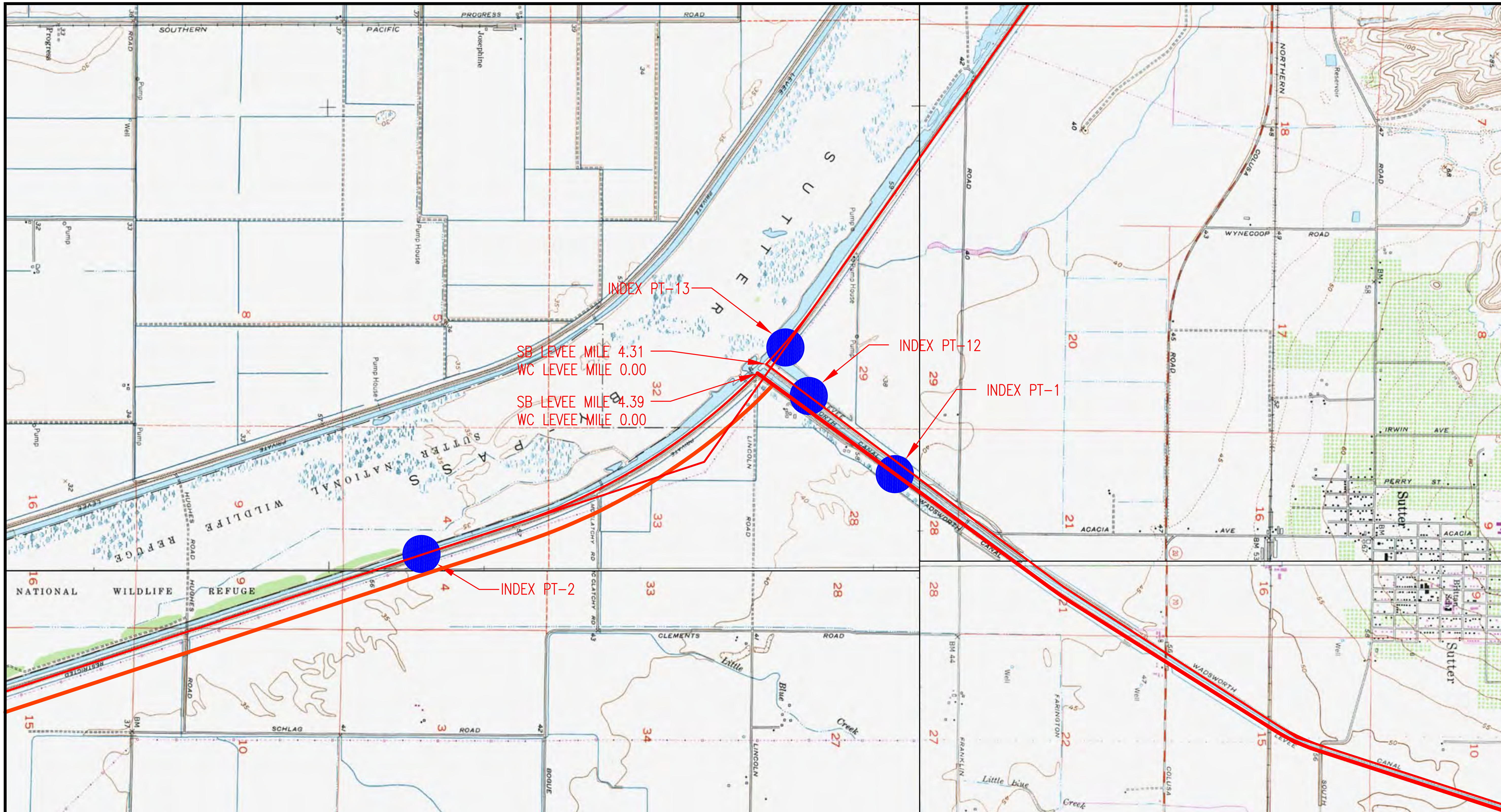

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INDEX POINTS 1,2,12,13

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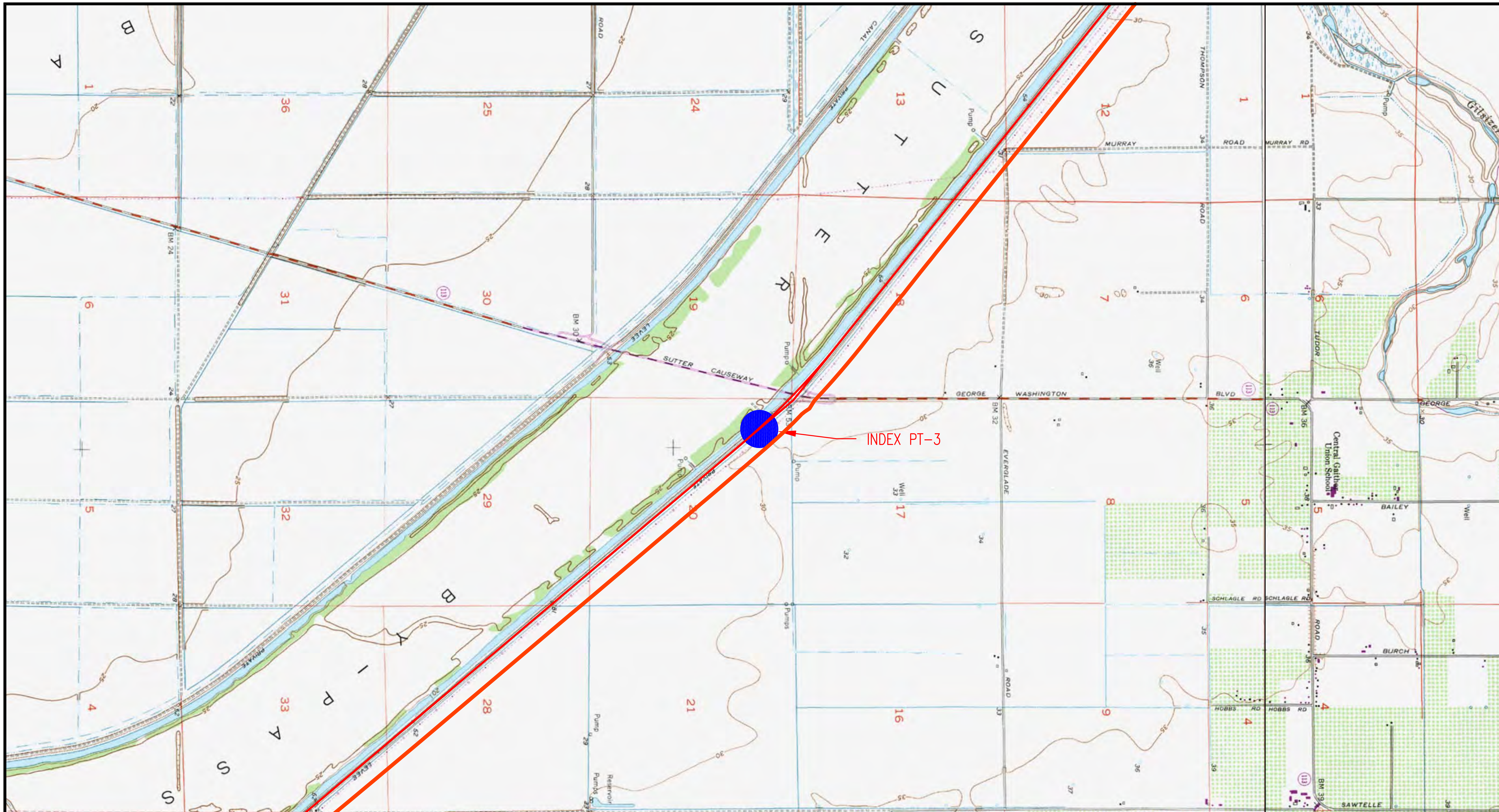

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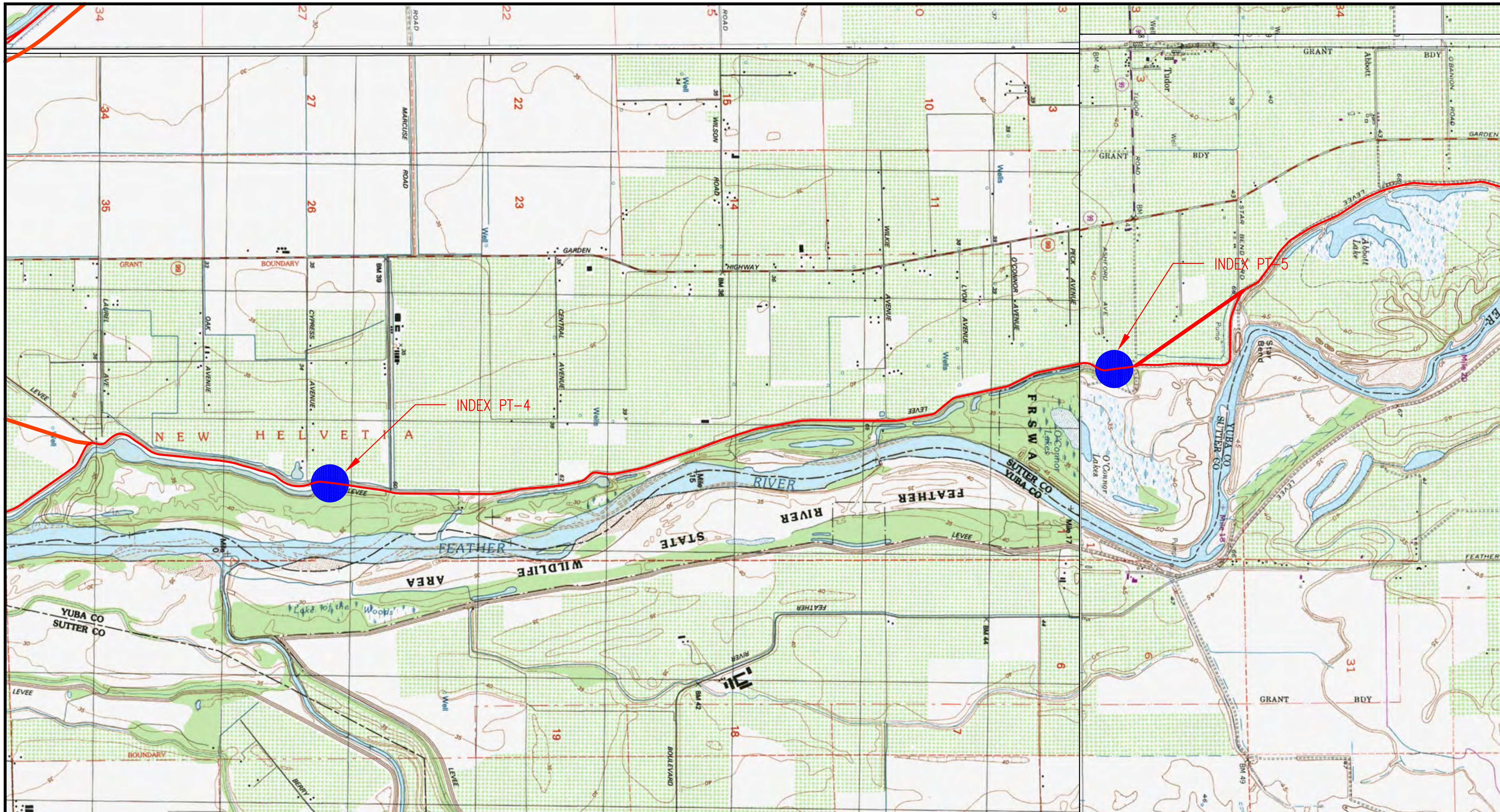

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INDEX POINT 4, 5

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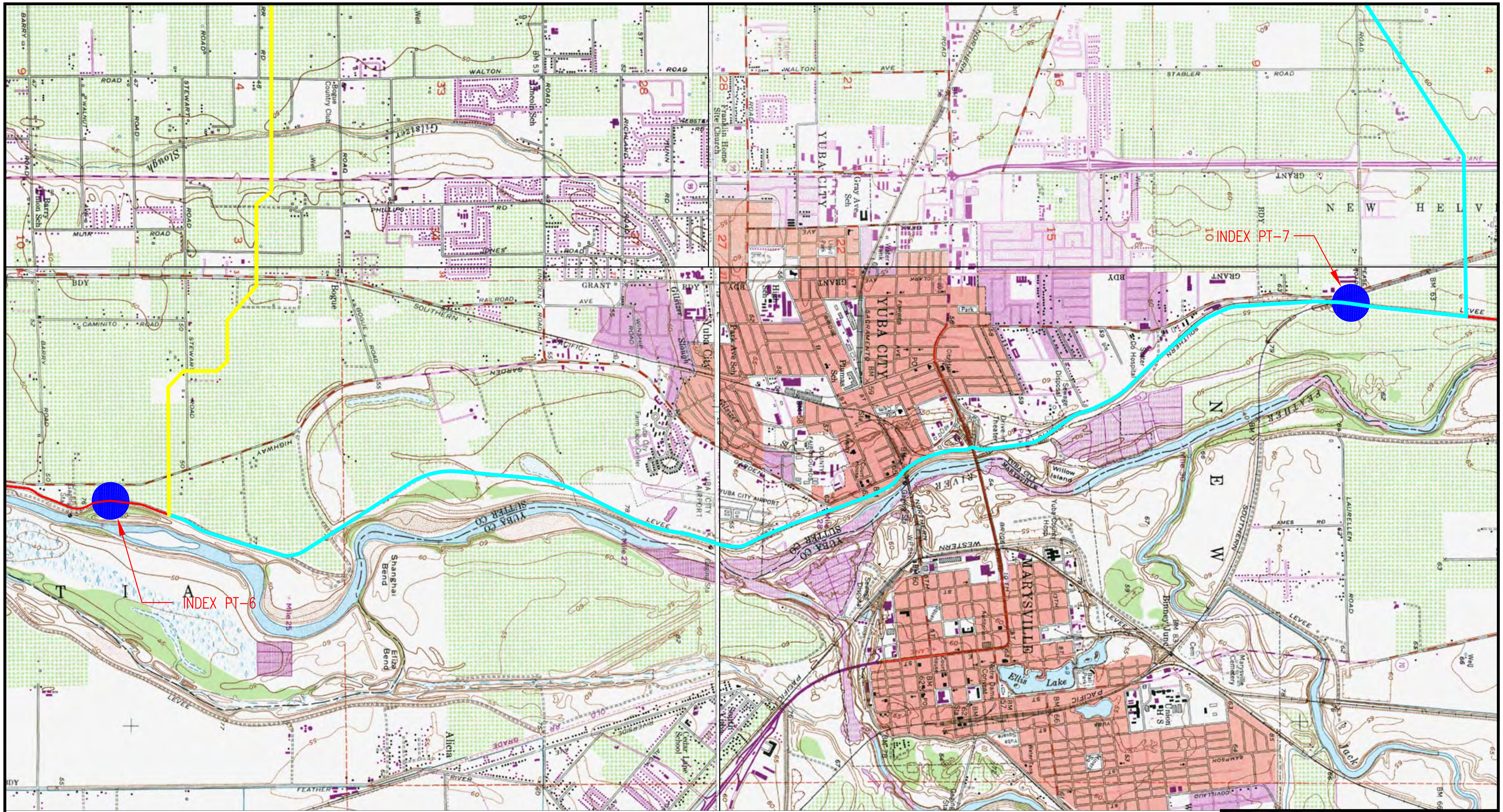
DWG-INFO



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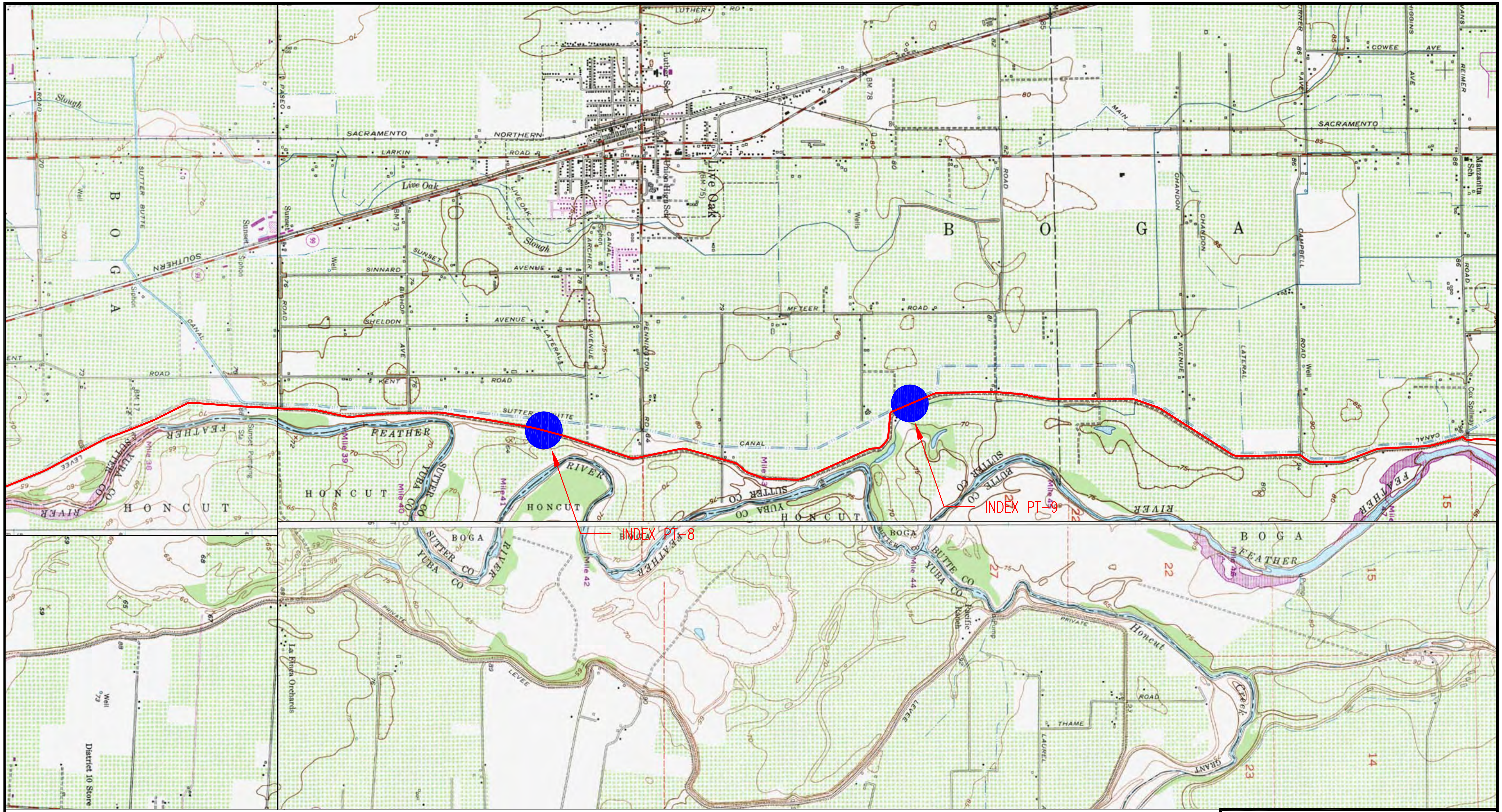

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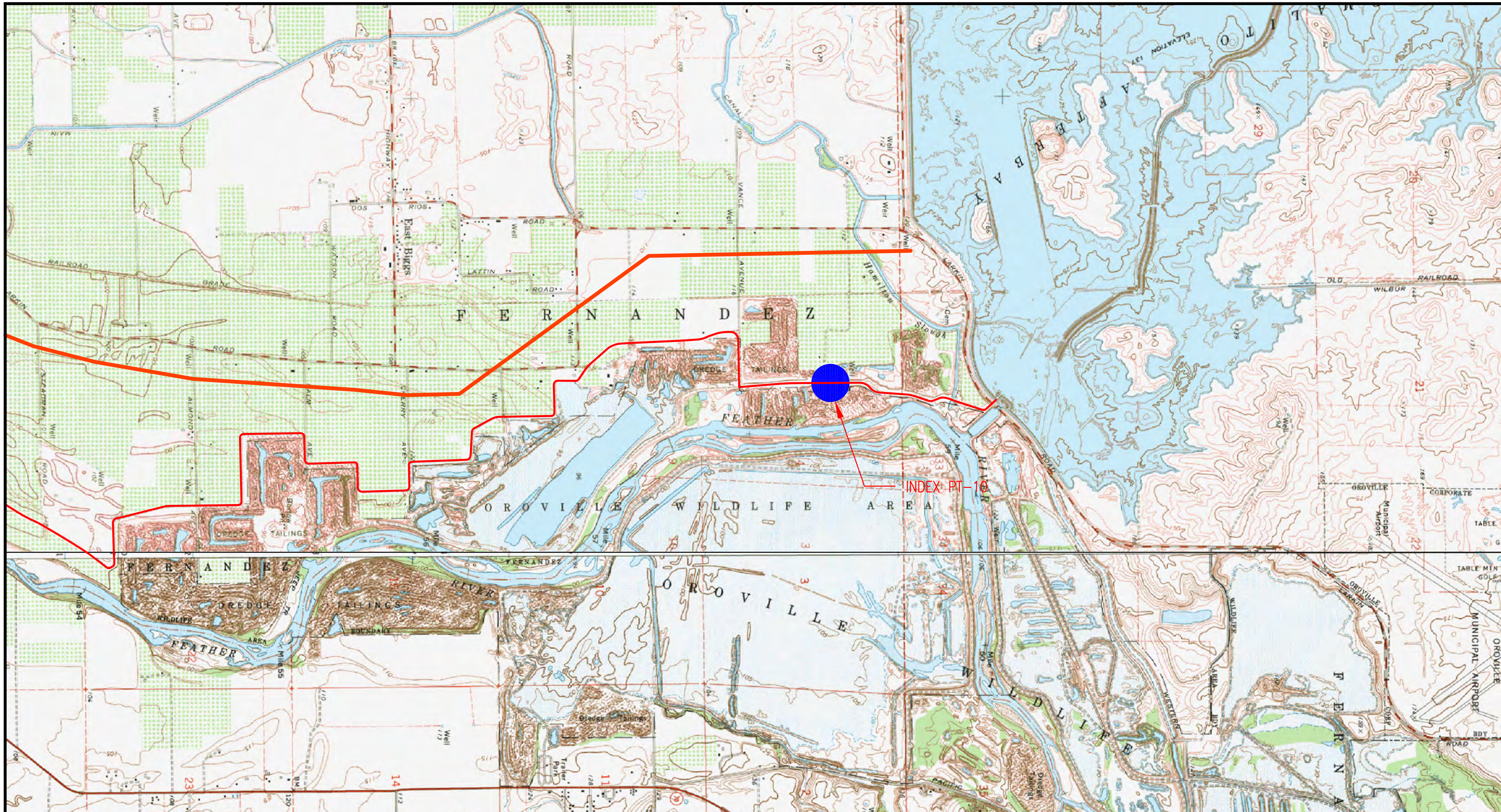
SUTTER FEASIBILITY STUDY UPDATED WITHOUT PROJECT CONDITIONS REPORT INDEX POINT 8, 9		
DATE: 8/20/2012	SCALE: 1"=2500'	PLATE-7

DWG-INFO



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INDEX POINT 10

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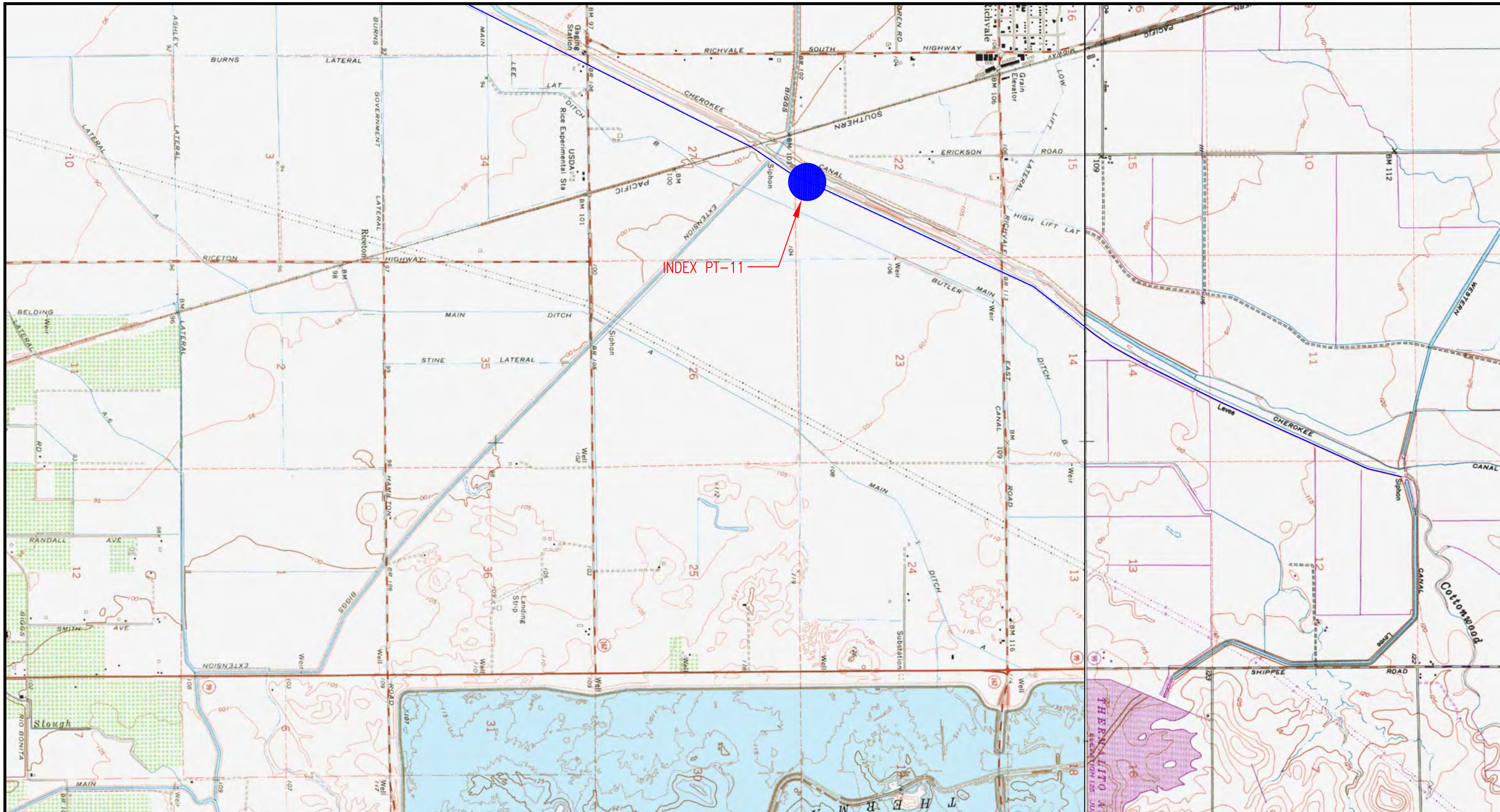

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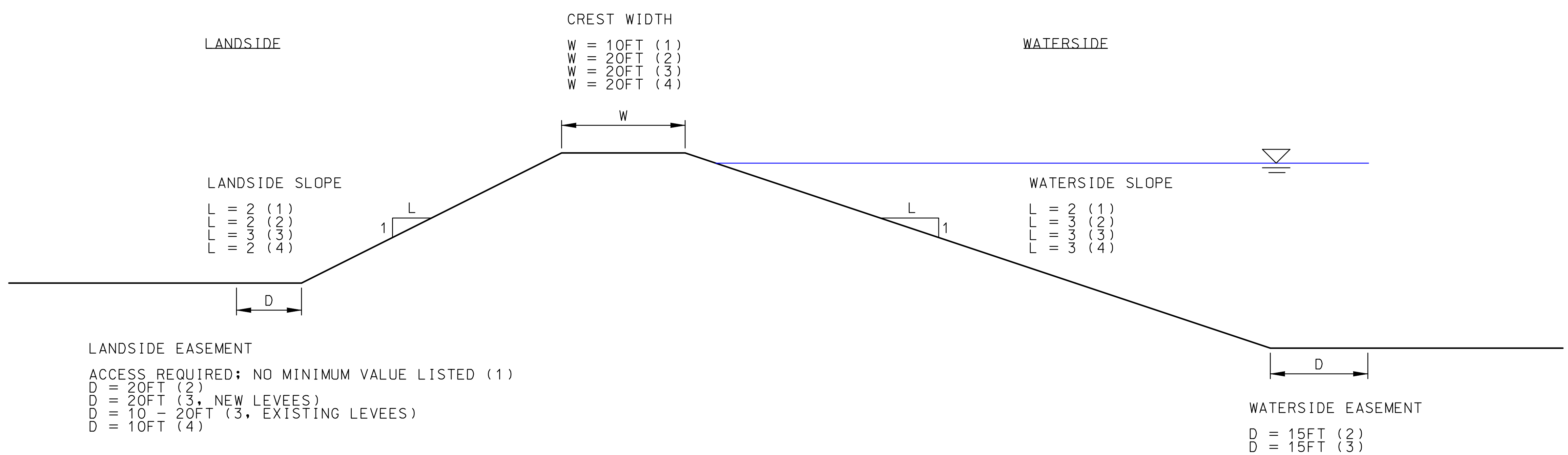

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Plate 10

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Sacramento District

Designed by: TUNG LE
Dwn by: TUNG LE
Reviewed by: -
Submitted by: -
Date: -
Spec No.: -
Design file no: -
Drawing Code: -
Rev. -

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

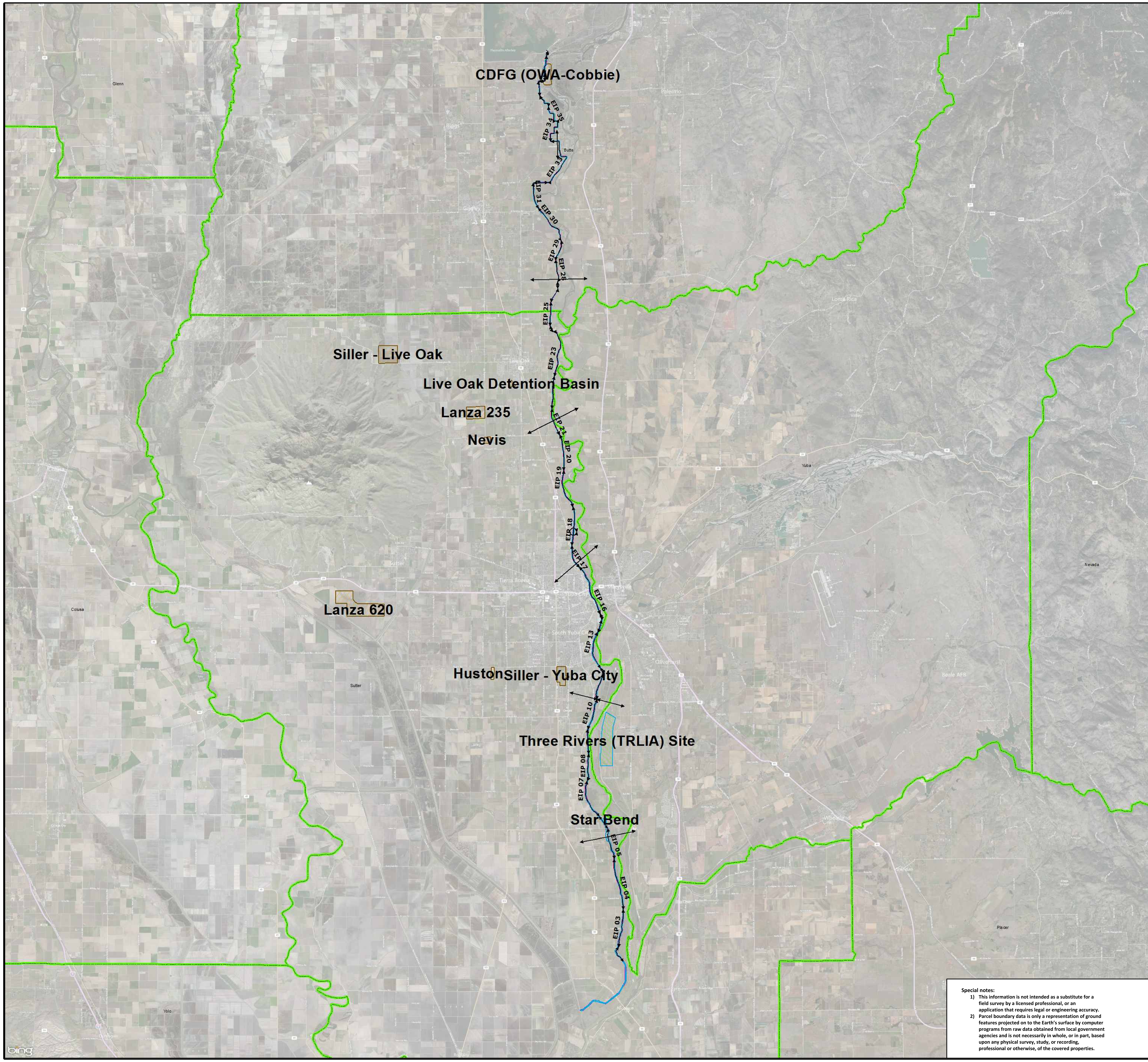
SACRAMENTO DISTRICT
IN-HOUSE DESIGN
1325 'J' STREET
SACRAMENTO, CA 95814-2922

SUTTER BUTTE COUNTY CALIFORNIA
SUTTER BASIN FEASIBILITY STUDY
CIVIL DESIGN APPENDIX
-
TYPICAL LEVEE GEOMETRIES
MINIMUM REQUIREMENTS
-

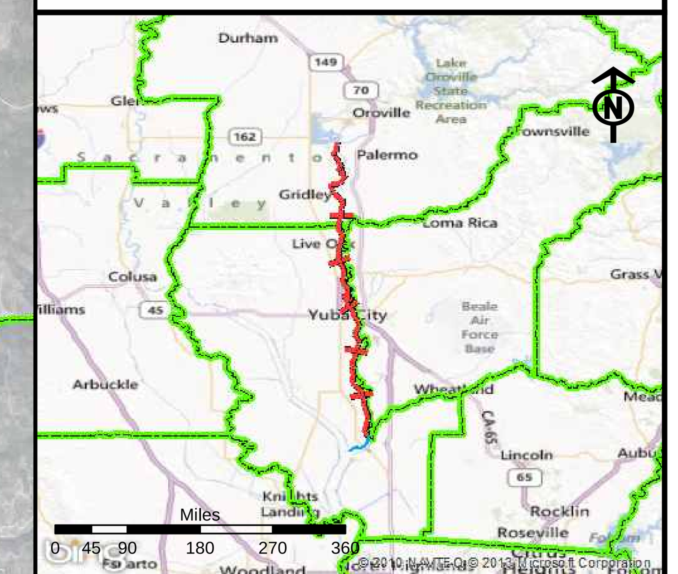
Sheet reference number:
-
Sheet - of -
AM-XX

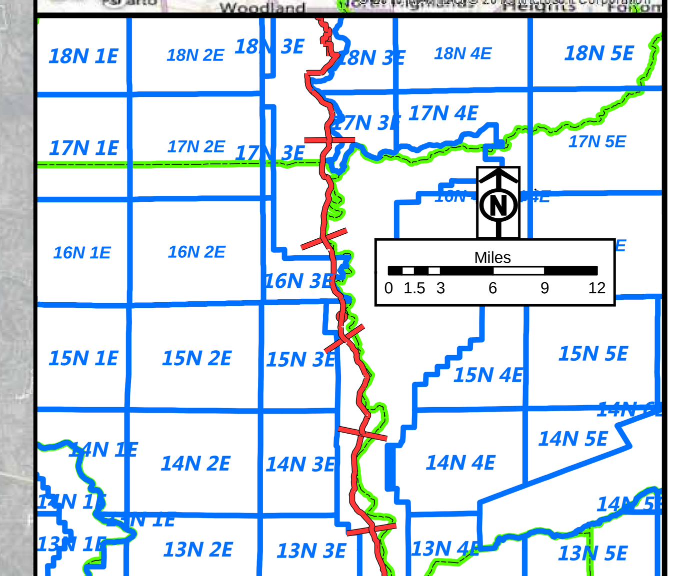
CORPS OF ENGINEERS

U.S. ARMY



Special notes:
1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.
2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.





T N

Miles
0 0.5 1 2 3 4

Coordinate System: GCS North American 1983
Datum: North American 1983
Units: Degree

M.D.B. & M.

Preliminary Draft
As Applicable

PROJECT MAP

DEPARTMENT OF THE ______
USING SERVICE US ARMY CORPS OF ENGINEERS

LOCATION OF PROJECT

STATE CALIFORNIA
COUNTY SUTTER & BUTTE
DIVISION SOUTH PACIFIC DIVISION (SPD)
DISTRICT SOUTH PACIFIC DISTRICT (SPK)
ARMY AREA ______
______ OF ______
______ OF ______

TRANSPORTATION FACILITIES

RAILROADS ______
STATE ROADS ______
FEDERAL ROADS ______
AIRPORTS ______

ACQUISITION

TOTAL ACRES ACQUIRED ______
FEE ______
PUBLIC DOMAIN: WITHDRAWN ______ / USE PERMIT ______
USE PERMIT (Other than P.D.) ______
TRANSFER: WITHDRAWN ______ / USE PERMIT ______
LEASE ______
EASEMENT RESERVED IN FEE DISPOSAL ______
LESSER INTERESTS: EASEMENT ______ / PERMIT ______ / LICENSE ______

DISPOSAL

TOTAL ACRES DISPOSED ______
SOLD ______
PUBLIC DOMAIN: WITHDRAWN ______ / USE PERMIT ______
USE PERMIT (Other than P.D.) ______
TRANSFERRED: FEE ______
LEASES TERMINATED ______
LESSER INTERESTS TERM ______
REASSIGNED ______
ACRES TO ______

LEGEND

EXCEPT FOR SPECIAL SYMBOLS SHOWN BELOW, MAP SYMBOLS ARE STANDARD IN U.S. ARMY FIELD MANUAL, FM 21-31, TOPOGRAPHIC SYMBOLS, DEC. 1968.

ACQUISITION AUTHORIZATION

Estates

Borrow
Mitigation
FPLE_Levee
OM_p
Parcels
CA_County_Boundary
Reach_ID

GENERAL PROJECT LOCATION

DEPARTMENT OF THE ARMY
OFFICE OF THE SACRAMENTO DISTRICT ENGINEER
SOUTH PACIFIC DIVSION

CARTOGRAPHER SJC
CARTO TECH. ______
CHECKED BY SJC

SUBMITTED BY
STEVE J. CAREY
CHIEF, CADASTRAL SECTION

RECOMMENDED BY
CHIEF, ACQ. & MGMT. BRANCH

REAL ESTATE
SUTTER & BUTTE COUNTIES
CALIFORNIA

SUTTER BASIN GRR
BORROW AND MITITGATION SITES
(SB8)

APPROVED BY ______ DATE ______
CHIEF, REAL ESTATE DIVISION

OFFICE, CHIEF OF ENGINEERS, WASHINGTON 25, D.C.
REMIS CODE: ______
REMIS UNIQUE ID: ______
INSTALLATION OR PROJECT NO. ______
DRAWING NO. ______

Path: K:\GIS_PROJECTS\CIVIL_WORKS\Sutter_Basin\Project\Borrow_Mit_Staging\20130710_SB_Bor_Mit-G1.mxd

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ENCLOSURES

ENCLOSURE A

LEVEE PHOTOGRAPHS

WADSWORTH CANAL LEFT LEVEE


Photo 1. General view of the levee, looking upstream. Canal channel on the left.


Photo 2. General view of the levee, looking upstream.

Photo 3. Waterside levee slope and canal, looking north. High ground of Sutter Buttes in the background.

Photo 4. Landside levee slope, looking upstream.


Photo 5. Canal bank erosion, looking upstream.

SUTTER BYPASS LEFT LEVEE (DOWNSTREAM OF WADSWORTH CANAL)

Photo 1. General view of the levee, looking upstream. Bypass channel is on the left

Photo 2. Landside levee slope, looking downstream.


Photo 3 Ditch about 50 feet from the landside toe.


Photo 4. Typical waterside levee slope, looking upstream

Photo 5. Stability berm at the landside toe, looking upstream

Photo 6. Pump Station No. 2, looking downstream

FEATHER RIVER SOUTH – MAINTENANCE AREA 3


Photo 1. General view of the levee, looking upstream. Feather River channel to the right.


Photo 2. Landside levee slope, looking downstream. Note electrical tower near the toe.

Photo 3. Ditch near the landside toe

Photo 4. Waterside levee slope, looking upstream.

FEATHER RIVER SOUTH – LEVEE DISTRICT 1


Photo 1. General view of the levee, looking upstream. Feather River channel on the right.


Photo 2. Waterside levee slope, looking upstream.

Photo 3. Landside levee slope, looking upstream.

Photo 4. Drainage ditch and power poles near the landside levee toe.


Photo 5. Encroachment into the landside levee slope in Yuba City.


Photo 6. Star Bend relief wells and relief well drainage ditch.

FEATHER RIVER NORTH – LEVEE DISTRICT 9

Photo 1. General view of the levee.

Photo 2. Railroad embankment crossing the levee alignment at the LD1/LD9 boundary.

Photo 3. Landside levee slope, looking downstream.

Photo 4. Waterside levee slope, looking upstream.

Photo 5. Sutter Butte Canal adjacent to the levee toe.

Photo 6. Sutter Butte Canal adjacent to the landside levee toe at the Sunset Pump Plant.

FEATHER RIVER NORTH – MAINTENANCE AREA 16

Photo 1. General view of the levee, looking upstream. Feather River channel to the right.

Photo 2. Waterside levee slope, looking upstream.

Photo 3. House at the landside levee toe.

Photo 4. Sutter Butte Canal adjacent to the landside levee toe.

Photo 5. Erosion/sloughing of the landside levee slope into the Sutter Butte Canal.

Photo 6. Landside levee slope with powerpoles near the toe, looking upstream.

FEATHER RIVER NORTH – MAINTENANCE AREA 7

Photo 1. General view of the levee, looking upstream. Feather River channel to the right.

Photo 2. Waterside levee slope, looking upstream.

Photo 3. Landside slope erosion/sloughing into the adjacent Sutter Butte Canal; house on waterside of levee.

Photo 4. Sutter Butte Canal adjacent to the landside levee toe.


Photo 5. Waterside levee slope at the location of the former Gridley Bridge.


Photo 6. Weir structure in Sutter Butte Canal adjacent to the landside levee toe.

FEATHER RIVER NORTH – HAMILTON BEND

Photo 1. General view of the levee embankment, looking upstream. Feather River channel to the right.

Photo 2. Waterside levee slope, looking upstream.

Photo 3. Landside levee slope, looking downstream.

Photo 4. Sutter Butte Canal Headgate Structure. Note gate stems and hoist mechanisms have been removed, leaving the gates “stuck” in the closed position.

Photo 5. Dredge tailings on the waterside of the levee. Note tailings are higher than the levee crest in some locations.

ENCLOSURE B

SBLS PERFORMANCE HISTORY AND IMPROVEMENTS/MODIFICATIONS (FROM PERIODIC INSPECTION REPORT NO. 1)

Table M-1: Summary of Levee Performance Issues

Levee Mile or Location	Year	Problem/Issue	Document	Page
Hamilton West Levee - HAM1				
12.3	1955	Levee "broke in 4 places and potholes appeared"	P1GDR	2-42
Maintenance Area 07 - MA07				
0.1 to 0.9	N/A	Underseepage and high groundwater during irrigation season due to high water level in canal	P1GER	2-11
1 to 1.2	1999	Several large holes opened up during construction of slurry wall	P1GDR	2-40
2.7 to 2.9	1986	Seepage and boils	P1GER	2-12
9.8	1986	Seepage and boils	P1GER	2-12
N/A	N/A	Landside seepage and boils at various locations	P1GDR	2-41
Maintenance Area 16 - MA16				
N/A	N/A	Landside seepage and boils at various locations	P1GDR	2-41
Levee District 9 - LD9S				
0 to 1.6	1996	Seepage	P1GDR	2.56
0 to 1.6	1997	Seepage	MEM-97	3
0.1 to 0.42	1966	Seven sinkholes from old stumps left during levee construction	P1GER	2-9
2.17	1955	"Severe leaks"	P1GDR	2-42
2.3	N/A	Heavy saturation and levees turning to "mush"	P1GDR	2-42
2.5	N/A	Heavy saturation and levees turning to "mush"	P1GDR	2-42
2.56 to 2.65	1982	Erosion and raveling of rock revetments	P1GDR	2-44
2.8 to 3.1	1997	Seepage	P1GER	2-10
3 to 3.2	1997	Clear-water pinhole boils	P1GDR	2-42
3.5 to 3.6	N/A	Heavy saturation	P1GER	2-10
3.65	1955	"Severe leaks"	P1GDR	2-42
3.8	1986	Landside seepage and sand boils	P1GER	2-10
4.55-5.46	1997	900-feet of irrigation ditch slumping at levee toe	MEM-97	2
4.63	1957	Seepage at and beyond waterside toe during irrigation	P1GDR	2-42
4.93	1957	Seepage at and beyond waterside toe during irrigation	P1GDR	2-42
5.1	1986	Landside seepage and sand boils	P1GER	2-11
5.4 to 5.7	1997	Landside embankment distress	P1GER	2-11
N/A	N/A	Landside seepage and boils at various locations	P1GDR	2-41
Levee District 1 - LD1S				
1.5	1986	Crack formed in levee	P1GDR	2-25
2.75 to 5.25	1997	Boils at landside levee toe	LTR -97	Sheet 1
3.8	1986	2,600-ft stretch of boils, soft wet ground w/in 100 ft of toe	P1GDR	2-25
3.8	1997	Seepage	P1GDR	2-25
4.1 to 5	1995	Clear seepage at and beyond levee toe	P1GDR	2-25
4.1 to 5	1997	200-foot stretch of boils	P1GDR	2-25
6.1	1986	Erosion of small trees on waterside berm, holes on top of berm, berm slope eroded	P1GDR	2-25
6.3 to 7.01	1997	26 sink holes, probably rodent-related	MEM-97	2
7.3	N/A	Waterside bank erosion encroached on levee section	P1GDR	2-25

Table M-1: Summary of Levee Performance Issues

Levee Mile or Location	Year	Problem/Issue	Document	Page
Levee District 9 - LD9S (continued)				
8	1986	Waterside levee toe, portions of boat ramp parking lot and subgrade eroded	P1GDR	2-25
8.13	1997	Bank erosion at pump structure at waterside levee	LTR -97	Sheet 1
10.8	1986	Seepage	P1GDR	2-25
10.8	1995	High water, seepage, boil	P1GDR	2-25
11.0 to 12.4	1909	Levee broke	P1GDR	2-25
11.0 to 12.4	1911	Levee broke	P1GDR	2-25
11.0 to 12.4	1955	Levee broke	P1GDR	2-25
11.0 to 12.4	1986	Boils	P1GDR	2-26
11.0 to 12.8	1995	Sand boil noted, relief wells are damaged/not maintained	MEM-95	1
11.2	1997	Heavy seepage and boils	LTR -97	Sheet 1
12.1 to 12.3	1997	Heavy seepage and boils	LTR -97	Sheet 1
12.8 to 14.5	N/A	Seepage	P1GER	2-7
14 to 15.5	1955	Seepage	P1GDR	2-26
14 to 15.5	1986	Saturated, unstable, bulging landside slope, water pouring from parking lot pavement cracks & floor slab, waterside erosion at toe	P1GDR	2-26
14.27 to 14.57	1997	Bank and levee erosion	LTR -97	Sheet 1
14.74 to 14.81	1997	Bank erosion	LTR -97	Sheet 1
14.95 to 15.55	1997	Heavy seepage	LTR -97	Sheet 1
10th St Bridge to 1,000 ft north	1995	Seepage 100 ft landside of levee and Von Geldern Way	MEM-95	1
RM 19	1995	Seepage at toe of levee and beyond	MEM-95	2
Maintenance Area 03 - MA03				
2.3 to 3.3	N/A	Excessive seepage	P1GDR	2-24
3.7	1986	Boil in drainage ditch near levee toe	P1GDR	2-25
3.7	1997	Heavy seepage entering drainage ditch near levee toe, sloughing of the drainage ditch	P1GDR	2-25
Sutter Bypass, E Levee - SBP2				
4.4 to 5.4	1997	Heavy seepage, soil heaving	P1GDR	2-13
5.4 to 13 (McClatchy Road to Gilsizer Slough)	1958	Landside toe - ground heaving with mud flow, sand boils, berm movement and cracks	P1GDR	2-13
5.4 to 13 (Oswald Rd to Gilsizer Slough)	1958	Longitudinal cracks along landside slope	P1GDR	2-13
12.7 to 14.6	N/A	Seepage and boils	P1GER	2-4
16.6 to 16.78	N/A	Critical seepage area	P1GDR	2-14
17.6	1997	Seepage	P1GDR	2-14
20 to 22.37	1986	Waterside erosion at toe and lower slope	P1GDR	2-14
21.88 to 22.07	2001?	Seepage, boils, sinkhole on landside berm	P1GER	2-4

Table M-1: Summary of Levee Performance Issues

Levee Mile or Location	Year	Problem/Issue	Document	Page
Wadsworth Canal – Unit 1, left bank - WAD1				
1.0	1997	Heavy seepage	P1GDR	2-9
0.0 to 0.6	1997	Seepage, pin boils	P1GDR	2-3
0.0 to 0.6	1998	Seepage, pin boils	P1GDR	2-3
N/A	N/A	WS berm erosion & rodent holes for much of levee reach	P1GDR	2-10
Interceptor Canal – Unit 2, east canal - INT2				
N/A	N/A	N/A	N/A	N/A

Key to Document Abbreviations

Author	Date	Title	Type	Abbrv
USACE Soil Design Chief	09/15/95	Memorandum for Civil Projects Management Section, Subject: Contract 3 Levee Seepage Investigations	Memo	MEM-95
Levee District 1	02/17/97	Levee District 1 to Honorable Vic Fazio, Summary of Repairs and Improvements required for the Feather River Levee of Levee District No. 1 of Sutter County	Letter	LTR -97
USACE, Chief of Soil Design Section	03/03/97	Memorandum For Civil Projects Management Section, Subject: Levee Performance Letters from LD 1 and MHM regarding LD9	Memo	MEM-97
URS Corporation (for DWR)	03/17/08	Draft Phase 1 Preliminary Geotechnical Evaluation Report (P1GER) Sutter Study Area Urban Levee Geotechnical Evaluations Program	Report	P1GER
URS Corporation (for DWR)	11/14/08	Phase 1 Geotech Data Report (P1GDR) Sutter Study Area Urban Levee Geotechnical Evaluations Program Contract	Report	P1GDR

Table I-1: Sutter County Levee Alterations/Modifications

Levee Mile or Location	Year Complete	Alteration/Modification	Document	Page	USACE Drawing	USACE Spec
Hamilton West Levee (HAM1)						
N/A	1954	Levee reconstructed	152	2	--	--
HAM1 and/or MA07						
Unknown	1963	Levee enlargement	152	2A	4-4-534	2949
Unknown	1965	Bank protection	152	2A	50-4-4118	3283
Site Mile 51.1	N/A	Bank protection	152	2A	50-4-5798	8367
Maintenance Area 07 (MA07)						
N/A	1954	Levee reconstructed	152	2	--	--
1 to 1.2	1999	Bentonite/shredded tire slurry wall and nine monitoring wells installed	P1GDR	2-40	--	--
3 to 3.2	N/A	50-foot deep by 3 ft thick waterside slurry cutoff trench on WS levee slope constructed, covered with impervious blanket & embankment fill	P1GDR	2-52	--	--
4.25	1970	Bank protection	152	2A	50-4-4377	3426
4.7 to 5.7	N/A	Backfill landside irrigation ditch	P1GDR	2-52	--	--
10.2 to 10.5	N/A	2,640 feet of levee crest raised up to 2 feet	P1GDR	2-52	--	--
Near Gridley Bridge	1956	Emergency levee repairs	152	2A	4-4-384	2059
1.93 miles upstream from Gridley Bridge	1956	Emergency bank protection	152	2A	4-4-424	2228
Maintenance Area 16 (MA16)						
Sta. 952+00 to 1380+00	1934	Levee enlargement	148	2	4-4-135	--
MA16 and LD9S						
Fm SPRR (2 mi N of Yuba City) to 12 mi N	1945	Levee enlargement	148	2	50-4-2193 and 4-4-267	--
8 mi N of Yuba City	1948	Berm fill, drainage ditch, and road approaches	148	2	4-4-292	

Table I-1: Sutter County Levee Alterations/Modifications

Levee Mile or Location	Year Complete	Alteration/Modification	Document	Page	USACE Drawing	USACE Spec
Levee District 9 (LD9S)						
2.17	N/A	Replaced levee center section with impervious material, landside slope bolstered with dredge tailing blanket, repaired waterside slopes	P1GDR	2-54	--	--
2.3	N/A	Levee fortified with rock	P1GDR	2-42	--	--
2.5	N/A	Levee fortified with rock	P1GDR	2-42	--	--
3.5 to 3.6	N/A	Levee fortified with rock	P1GER	2-10	--	--
3.65	N/A	Replaced levee center section with impervious material, landside slope bolstered with dredge tailing blanket, repaired waterside slopes	P1GDR	2-54	--	--
4.55-5.46	1997	900-feet of irrigation ditch backfilled	#72	2	--	--
4.65 to 5.05	1960	USACE constructed 4-5 foot high, 60-foot wide 2,100-foot long waterside levee toe berm fill	P1GDR	2-54	--	--
N/A	1960	Levee stabilization	148	N/A	4-4-508	2655
Sta. 952+00 to 1380+00	1934	Levee enlargement	148	2	4-4-135	--
Site Mile 34.0	1974	Bank protection and selective clearing	148	N/A	4-4-599	4815
Site Mile 39.5&41.5	1968	Bank protection	148	N/A	50-4-4078	3288
LD9S and LD1S						
Unknown	1960	Levee stabilization in Levee Districts 1 & 9	148	N/A	4-4-508	2655
Levee District 1 (LD1S)						
Sta. 49+40 to 1380+00	1939	Levee enlargement	144	N/A	44-172-1	--
Shanghai to Starr Bend	1940	Levee enlargement	144	N/A	4-4-205-1	--
2 mi south of Starr Bend to Bear River	1940	Levee enlargement	144	N/A	4-4-205-1	--
Downstream from Yuba City	1956	Emergency repair and reconstruction of 8,000 feet of destroyed levee	144	N/A	4-13-405	--
Starr Bend	1956	Emergency levee repairs and bank paving	144	N/A	--	--
Shanghi Bend	1957	Drainage pump and sump	144	N/A	4-4-438	2295
N/A	1940	USACE reconstructs levee	P1GDR	2-22	--	--
N/A	1952	Section of levee crown "surfaced for patrol road purposes"	144	N/A	--	--

Table I-1: Sutter County Levee Alterations/Modifications

Levee Mile or Location	Year Complete	Alteration/Modification	Document	Page	USACE Drawing	USACE Spec
1.5	1998	Seepage/stability berm constructed	P1GER	2-6	--	--
3.8	1998	Levee raised 1 ft, pervious toe drain & seepage/stability berm installed	P1GDR	2-25	--	--
3.8 to 4.5	1997	Raised levee	P1GDR	2-36	--	--
4.1 to 5	1997	Relief wells installed	P1GDR	2-25	--	--
7.2 to 8.2	1997	Installed toe drain and berm	P1GDR	2-36	--	--
8	1986	Levee raised 1 ft, pervious toe drain & seepage/stability berm installed	P1GDR	2-25	--	--
10.8	1956	Emergency levee repairs	P1GDR	2-25	4-4-433	2252
10.8	1986	LD1 constructed seepage/stability berm	P1GDR	2-25	--	--
10.8	1997	LD1 construct new levee with 25-ft deep slurry wall & 6-ft wide clay core	P1GDR	2-25	--	--
11.0 to 12.8	1957	USACE emergency levee reconstruction outboard of previous location, installation of 42 relief wells	P1GDR	2-25, 26 & 53	4-13-405 & 4-4-435	2256
11.0 to 12.4	1990	Yuba City installed shallow seepage interceptor system along southern portion of site and additional relief wells	P1GDR	2-26	--	--
12.8 to 14.3	1997	4,610 feet of slurry wall, 1,690 feet of toe drain and berm, and 2,700 feet of slurry wall installed	P1GDR	2-35	--	--
12.8 to 14.5	1998?	Impermeable cutoff wall constructed	P1GDR/ P1GER	2-26/2-7	--	--
12.8 to 14.5	1990	Yuba City installed shallow seepage interceptor system along southern portion of site and additional relief wells	P1GER	2-7	--	--
N/A	1957	Emergency levee repairs, relief trench drain installed	P1GDR	2-26	4-4-437	2302
14 to 15.5	1956	Emergency levee repairs	P1GDR	2-26	4-4-431	2238
14 to 15.5	1960	Levee stabilization, 600 lineal feet of cutoff trench installed	P1GDR	2-26&53	4-4-508	2655
14 to 15.5	1986?	Yuba City constructed emergency landside toe berm	P1GDR	2-26	--	--
14 to 15.5	N/A	Emergency landside toe berm replaced with permanent seepage/stability berm from 5th St Bridge to 2,500 ft N of 10th Street Bridge	P1GDR	2-26	--	--
14 to 15.5	1998	Rock slope protection installed	P1GDR	2-26	--	--
From midway between 5th St & SR 20 to 4,000 ft north	N/A	Landside sloping drain and berm with toe trench	P1GDR	2-33	--	--
Mile 24.5	1966	Bank protection	144	N/A	50-4-4004	3154

Table I-1: Sutter County Levee Alterations/Modifications

Levee Mile or Location	Year Complete	Alteration/Modification	Document	Page	USACE Drawing	USACE Spec
Maintenance Area 03 (MA03)						
Sutter By-Pass to Nicolaus Bridge	1943	Enlarged levee	143	2	--	--
N/A	1952	Surfacing levee crown for patrol road purposes	143	2	--	--
N/A	1956	Emergency levee repairs	143	2A	4-4-417	2221
0 to 5.11	1963	Levee stabilization	P1GDR	2-24	4-4-531	2783
3.7	1997	Drainage ditch near levee toe converted to toe drain	P1GDR	2-25	--	--
3.7	1998	Seepage/stability berm constructed	P1GDR	2-25	--	--
Sutter Bypass, E Levee (SBP2)						
Sta. 0+66 to 16+50	1942	Levee enlarged to current dimensions	135	2 & 2A	50-4-1946-2	--
Sta. 1+62 to 484+00	1942	Levee enlarged to current dimensions	135	2	50-4-1945-2	--
Sta. 331+00 to 692+00	1942	Levee enlarged to current dimensions	135	3	50-4-2772	--
Sta. 800+00 to 1137+62	1942	Levee enlarged to current dimensions	135	2 & 2A	50-4-1946-2	--
Wadsworth Canal to Gilsizer Slough	1958	Emergency levee repair	135	3A	50-1-3531	2463
N/A	1968	Bank protection	135	3A	50-4-4393	3492
4.4 to 5.4	2001	12 foot wide pervious toe drain and seepage/stability berm	P1GDR	2-12	--	--
5.4 to 13	1957	Installation of exploration holes backfilled with sand (relief drains)	P1GDR	2-14	--	--
5.4 to 13	1958	Construction of 2 ft wide by 15 ft deep relief drain	P1GDR	2-14	--	--
7	1963	Levee repair, bank protection, channel rect.	135	3	4-4-536	
12.66	N/A	Pressure relief well at landside toe	P1GDR	2-14	--	--
12.7 to 14.6	1992	Construction of 10-ft high, 130-ft long rock relief toe drain and berm with 3-ft thick and 10-ft wide embankment berm	P1GDR	2-14	--	--
12.88	N/A	Pressure relief well at landside toe	P1GDR	2-14	--	--
16.6 to 16.78	N/A	Filled ditch on landside toe, constructed wide berm	P1GDR	2-14	--	--
17.6	2001	791-ft long, 2-ft wide, 5-ft deep relief toe trench/drain and overlying seepage/stability berm	P1GDR	2-14	--	--
19.42 to 19.5	N/A	Bow levee constructed around abandoned landside state pumping plant	P1GDR	2-15	--	--
21.88 to 22.07	2001?	1,000 foot long vertical drain in landside berm	P1GDR	2-14		
Near Nelson Bend	1971	Emergency repairs - 800 ft levee construction; 2,300 ft of bank protection	135	3A	4-4-588	3903

Table I-1: Sutter County Levee Alterations/Modifications

Levee Mile or Location	Year Complete	Alteration/Modification	Document	Page	USACE Drawing	USACE Spec
SBP2 and WAD1						
4.48 (SBP2) to 0.5 (WAD1)	2008	3,000 ft of soil-cement-bentonite cutoff wall to 40 to 61 foot depth, levee reconstruction near 0.4 (WAD1) due to slurry boil at waterside levee toe	135	3A and 3B	50-04-6235	1520
Wadsworth Canal – Unit 1, left bank (WAD1)						
N/A	1942	Enlarged levees to current configuration	135	3	50-4-1949-3	
1.0	2001	USACE constructed pervious toe drain	P1GDR	2-9		
Interceptor Canal – Unit 2, east canal (INT2)						
N/A	N/A	N/A	N/A	N/A	N/A	N/A

ENCLOSURE C

REPORT ON COMP STUDY VERTICAL DATUM CONVERSION

US Army Corps of Engineers, Sacramento District
American River Watershed Common Features, Control Survey
Contract No. W91238-07-D-0001, Task Order 0006 and 0007

Report for Task 5
July 01, 2010

Prepared by:

HJW GeoSpatial, Inc.
2001 Broadway, Third Floor
Oakland, CA 94612
510-834-2001

HJW geospatial
PACIFIC AERIAL SURVEYS

REPORT TITLE: Task 5 Report Summarizing Tasks 1 through 5 that have undergone ITR

SUBMITTED BY: HJW GeoSpatial, Inc, July 01, 2010

REPORT STATUS: For review and approval by USACE

PROJECT TITLE AND LOCATION: American River Watershed Common Features, Control Survey, Sacramento CA.

CONTRACT NO: W91238-07-D-0001 Task Order 0006 & 0007
The survey services performed for this project are described in the Statement of Work dated 5 June 2009 and revised 24 July 2009, together with the Supplemental Statement of Work dated 9 December 2009 and Revised 7 January 2010, issued as Task Orders 0006 & 0007 to USACE Contract No. W91238-07-D-0001

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1. Project Description

Background

During the late 1990's and early 2000's, the Sacramento District collected topographic and bathymetric survey data for the Sacramento River Bank Protection Project and Sacramento-San Joaquin River Basins Comprehensive Study. The data covered most of the Central Valley waterways. The mapping data is currently being used by multiple ongoing studies in the region. The primary submittals of the mapping data effort were Digital Terrain Model (DTM) surfaces and annotated contour and planimetric maps generated from the DTM surfaces. Cross sections were cut from the DTM surfaces for hydraulic models.

The comprehensive study mapping was performed by three vendors: Towill (Concord, CA), Andregg Geomatics (Auburn, CA) and Ayres (Sacramento, CA), under contract with the Corps of Engineers. Most of the comprehensive study mapping was collected to meet 1 inch = 200 feet, 2 foot contour interval map accuracy standards. The contract documents specified the mapping was to be based on NGVD29 vertical datum and NAD83 horizontal datum. Control surveys were tied to NGS benchmarks using static GPS network methods and Geoid 96. Most topographic data were collected using aerial photogrammetric methods. However, LiDAR was used for all of the Towill area and a portion of the Ayres/Andregg area. Bathymetric data for all areas were collected using post processed kinematic GPS for vertical and horizontal positioning of soundings. Bathymetric data were merged into the topographic data to create seamless data sets. Figure 1 illustrates the overall project area along with primary and secondary control locations.

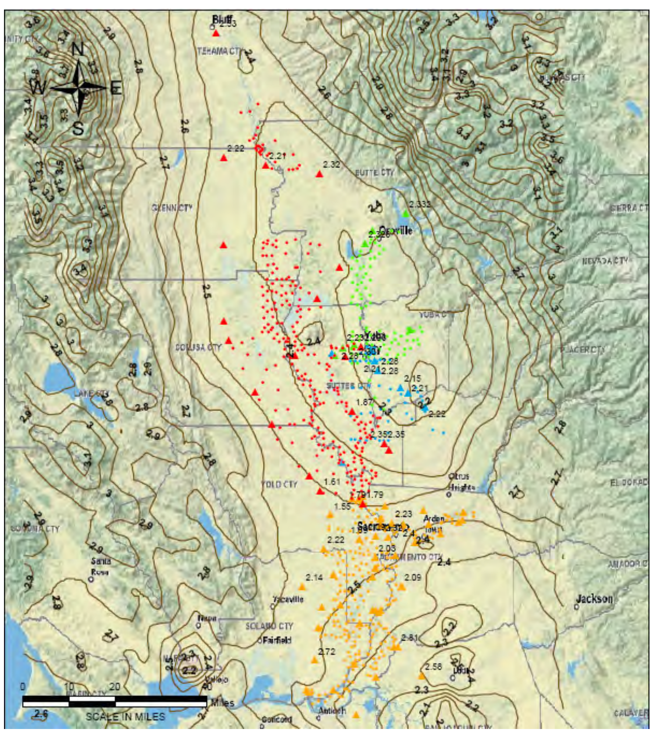

Figure 1- Map of project area and control network

Purpose

The NGS no longer provides elevations adjusted to the NGVD29 datum and recommends that NAVD88 be used for control. The U.S. Army Corps of Engineers, in cooperation with the National Geodetic Survey (NGS), has several engineering manuals including EM 1110-1-1005 Control and Topographic Surveying (Reference a) and EC 1110-2-6065 Comprehensive Evaluation of Project Datums (Reference b), that now mandate all such data and its many applications reference the North American Vertical Datum of 1988 (NAVD 1988). To make the comprehensive study mapping data compatible with recent data collected in the NAVD88 datum, the American River Common Features GRR Project has requested these data be accurately updated to the NAVD88 datum. The purpose of the five tasks described in this report is to develop a project-specific conversion surface to apply to the original mapping deliverables, to convert the datasets into NAVD88.

2. Task 1 Overview

Task Objective
Develop quality control plan and geospatial data management plan

Procedure
HJW developed a quality control plan which was signed and submitted by HJW on October 7th, 2009. This plan illustrates the following elements:

- HJW Quality Statement
- Management approach to quality control
- Project delivery team
- Quality control procedures
- Project milestone quality control timeline
- Independent technical review

On October 8th, 2009, HJW provided the geospatial data management plan, which integrates geospatial data management into project management business process, facilitating the implementation of enterprise data management.

Conclusion
HJW's quality control plan and geospatial data management plans were approved by the USACE, and the project team proceeded to Task 2.

3. Task 2 Overview

Task Objective
Review comprehensive study mapping control

Procedure
Task 2 as listed in the Statement of Work involved the review of the comprehensive study mapping control and GPS network files for the four separate networks. A subset of stations from the four networks was selected to be included in our survey, subject to recovery efforts, and criteria for selecting alternate stations were identified.

As part of Task 2, a technical memorandum was prepared describing the selection of stations for survey, the planned survey method and control. A draft report was prepared in accordance with the original Statement of Work which described a static GPS survey network to be surveyed according to NGS58/59 specifications. Interpretation of these specifications, particularly in regards to station spacing and minimum observation times, revealed that following the strictest interpretation would be unnecessarily prohibitive for this project given the accuracy requirements, and this led to discussion of alternate methods. Upon discussion with the USACE it was determined that network RTK would meet the project needs and the scope was modified by the issuing of a supplemental Statement of Work (dated December 9, 2009 and Revised January 7, 2010) before the control survey had commenced.

Conclusion
A final technical memorandum, dated March, 10 2010, describing the proposed survey was subsequently accepted by the USACE and is included in this submittal as Task 2/Appendix A. Filename: “Appendix A - 100310 Control Memo.pdf”.

4. Task 3 Overview

Task Objective
Control survey to achieve the following objectives:

1) Provide adequate NAVD88 vertical constraints to readjust four legacy static GPS networks to the NAVD88 datum. (Upper Feather River, Lower Feather River, Lower Sacramento & Upper Sacramento networks)
2) Resurvey a selection of points from all four legacy GPS networks to establish check points for validation of a conversion surface to be created from the readjusted legacy networks. These points will not be used in developing the conversion surface
3) Establish accurate NAVD88 benchmark elevations at approximately 30 State of California Department of Water Resources (DWR) stream gauging stations, and level elevations to additional marks and features at each site.
4) Establish accurate NAVD88 elevations on approximately 20 existing USED benchmarks
5) Establish accurate NAVD88 elevations on control points used in topographic mapping of five weirs specified by the USACE, to facilitate conversion of the mapping to the NAVD88 datum

Methodology for the control survey is described in the final technical memorandum, submitted as the Task 2 deliverable for the project.

4.1. Schedule

The three main components of the control survey were performed through the following dates:

1) **Control Station Recovery**: Most Stations were recovered between November 23, 2009 and December 17, 2009. Additional recovery efforts for USED Benchmark and stream gage reference marks took place in January through April of 2010.

2) **GPS Surveying:** GPS observations for control stations and Comprehensive Study Mapping control stations were performed between late January 2010 and early March 2010. Additional observations at USED benchmarks, stream gages, and reoccupation of stations with poor repeat precision were performed throughout May and April of 2010.

3) **Leveling:** Most leveling was performed during the second half of March 2010, with some additional leveling performed in April 2010

4.2. Control Station Recovery

Stations were searched for and recovered using handheld GPS receivers (Trimble Juno SB) and ArcPad 8.0 software. Recovered stations were photographed using the onboard camera, and basic recovery information (date, condition, party chief) was stored. An ESRI ArcGIS shapefile was created including all collected information and links to photographs. Photos taken using the Trimble Juno SB hardware were also geotagged if the GPS receiver had a positional fix at the time the photo was taken. Photos and GIS data are delivered with this submission in the folder: Task 3/Monument Photos and GIS Data.

An evenly distributed selection of stations from each of the comprehensive study GPS networks was identified as part of Task 2, and the field staff was instructed to search for these stations. If the selected station was not found or found disturbed, the crews searched for nearby stations until one in good condition was recovered. NAVD88 control stations were also recovered at this time.

4.3. RTK GPS Survey

Through coordination with the USACE it was determined that the best methodology to perform the survey within the time, budget & achieve the positional accuracy required would be to use RTK GPS in conjunction with a Real-Time Network (RTN). To maximize the accuracy of the RTK survey, all stations were observed for a minimum of 5 minutes (300 epochs), and all stations were observed at least twice, with observations a minimum of 3 hours apart in order to obtain significantly different satellite geometry. The two observed heights were then averaged to obtain a final result.

Statistical review of the data obtained in the first few weeks of surveying showed that for repeat observations, the 95% confidence interval for the height component was approximately +/- 0.20 feet between observations. This value was then used as a basis to determine which stations would need to be observed a third time. Any stations for which the two observed heights did not agree within 0.20' were observed a third time and the results were re-evaluated. If two of the heights were in relative agreement and the third was an obvious outlier, then the outlier was discarded and the average of the remaining two was taken. If the third observed height was midway between the first two observed heights, then an average of all three was taken.

Our assessment of the accuracy of the NAVD88 elevations obtained by RTK methods is based on a number of components. Evaluation of repeat observations showed the vertical accuracy of any given measurement to be +/- 0.2 feet at 95% confidence. Derived heights were based on a GPS site calibration to control stations with inherent uncertainty (typically height modernization stations published +/-0.10 foot vertical accuracy), and observations on the control stations have the same uncertainty as all other RTK observations. There is also inherent uncertainty in the Geoid model. We estimate that the overall vertical accuracy of surveyed stations relative to the National Spatial Reference System (NSRS) and NAVD88 will be better than +/-0.33 feet at 95% confidence.

The identical survey methodology was used for all stations observed as part of this survey regardless of purpose, including the comprehensive study mapping control, NAVD88 calibration points, USED benchmarks and stream gage benchmarks. All RTK survey data is included in the "RTK Survey Data" folder in the Task 3 submittal folder.

4.3.1. Equipment

Two of Bestor's RTK rovers were configured to work with either RTN, and two survey parties were dispatched to survey simultaneously. Equipment used consisted of the following:

Rover #1:
Trimble 5800 GPS Receiver / integrated GPS antenna
Trimble TSC-2 Data Collector with Survey Controller Software (v. 12.45)
Verizon MiFi 2200 mobile WiFi data link with Verizon Wireless 3G data service
Fixed-height GPS rover rod with clamp-on tripod.

Rover #2:
Trimble 5700 GPS Receiver
Trimble Zephyr Geodetic GPS Antenna
Trimble TSC-2 Data Collector with Survey Controller Software (v. 12.46)
Motorola Cellular phone with Verizon 3G data service & Bluetooth Dial-up Networking
Fixed-height GPS rover rod with clamp-on tripod.

4.3.2. Real-Time Networks

Two separate Real Time Networks (RTN's) were utilized to perform the RTK survey, maintained by two separate survey equipment vendors. Each network has unique characteristics. It was necessary to use both to networks achieve complete coverage of the project area.

Our primary deciding factor in deciding which to utilize was network coverage. Topcon California's network includes a higher density of stations near the southern end of the project, while the network operated by California Survey and Drafting Supply has coverage that extends further north.

Topcon California RTN:

Bestor is partnered with Topcon California to host a network station from their RTN on our building. This provides us with one user account to use the network, and it was used as the primary network for our survey. A second subscription was purchased in order to allow us to survey with two receivers simultaneously. Coverage for Topcon California's RTN does not extend to the northernmost extents of the project, however, so we used an alternate RTN to survey the northerly stations.

Topcon California's network includes options for network or single-base solutions. Network solutions were used exclusively for this project. While the RTN supports GLONASS observations, our rovers used only GPS for this project.

Real-time corrections for the Topcon California RTN were delivered via the RTCM broadcast format. This format did not allow positions to be store as vectors using our field equipment, therefore all observations were stored as positions.

California Survey and Drafting Supply (Trimble) RTN:

Corrections for the California Survey and Drafting Supply RTN are delivered via the CMR Broadcast format, and using this format we were able to choose to store our observations as vectors. All stored positions were based on a network solution based on the surrounding reference stations, however storing the observations as vectors from the nearest physical reference station can allow additional functionality using Trimble software. While the RTN supports GLONASS observations, our rovers used only GPS for this project.

4.3.3. GPS Site Calibrations

Three separate GPS site calibrations were performed, the results of which are include as GPS Site Calibrations Reports in Appendices B, C & D in the Task 3 submission folder.

As two different RTNs were used to cover different Geographic areas, each needed to include observations on a sufficient number and distribution of control stations to calibrate the given RTN to desired coordinate system and datum. Stations used in the calibrations are identified in the attached GPS Site Calibration Reports.

In addition to the two separate calibrations required by the use of two separate RTNs, the Lower Sacramento mapping area was calibrated to a different set of control stations than the rest of the project, which required a separate site calibration. All observations in the Lower Sacramento area used Topcon California's RTN, so there was no separate calibration of the CSDS RTN to the Lower Sacramento control.

Our survey was designed to include calibration points at approximately a 20 km spacing throughout the project, which we generally accomplished, however not all of the control stations we searched for were recovered which resulted in greater spacing in certain areas.

While performing the site calibrations, stations with high residuals were carefully reviewed, and in some cases obvious outliers were removed from the calibration.

GPS Site Calibrations were based on the following sets of control stations:

Lower Sacramento Mapping Area:

In the Lower Sacramento Mapping area, the Topcon California RTN was calibrated to stations from Delta Height Modernization survey, as published by the NGS. NGS published heights for these stations are derived using GEOID03, and are published with orthometric heights to the 0.1 foot. No surveying was performed in the Lower Sacramento mapping area using the California Survey and Drafting Supply RTN. Please refer to Appendix B in the Task 3 submission folder.

Upper Sacramento, Lower Feather & Upper Feather Mapping Areas:

In all other mapping areas, two separate site calibrations were performed to derived orthometric heights obtained by applying the GEOID09 separation to the unpublished Sacramento Valley Height Modernization Project (SVHMP) ellipsoid heights, as delivered by the USACE. Please refer to Appendix C and D in the Task 3 submission folder.

4.4. Leveling

Leveling was performed at various locations for specific purposes identified in the Statement of Work. Most of the leveling required was to establish elevations on various reference marks at stream gages.

Additional leveling was required for certain USED benchmarks. Where a USED benchmark was recovered but was not in a GPS-suitable location, a temporary benchmark was established nearby and leveling was performed from the temporary benchmark to the USED benchmark.

Equipment used for leveling consisted of Leica NA2 auto levels. All elevations determined through leveling were part of a closed loop.

All leveling was performed using closed loops, and all loop closures were less than 0.02 feet. Local accuracy between stations within one level loop is +/- 0.02 feet.

4.4.1. Stream Gages

The locations of stream gage reference marks were provided by the USACE, who obtained them from the operating agency (California Department of Water Resources or U.S. Geological Survey). At each gage site, our crews attempted to recover the reference marks indicated.

If a reference mark was found in a GPS-suitable location, an elevation was established by GPS methods (described above), and a leveling crew later returned to the gage site to level from the GPS-established benchmark elevation to additional reference marks and the water surface. If no recovered reference marks were GPS-suitable, then a temporary benchmark was established at the site using GPS methods for use as a local benchmark.

Our office prepared a custom form specific for leveling at the stream gage locations, and our leveling crews completed a leveling form for each stream gage. Scans of the completed stream gage leveling forms are attached as Appendix E in the Task 3 submission folder.

The GPS-established reference mark at each stream gage was determined through RTK GPS methods described in Section 4.3. Estimated accuracies for the reference marks with respect to the National Spatial Reference System (NSRS) and NAVD88 are +/-0.33 feet as described above. All leveling at each gage site was performed as part of a closed loop, and all loop closures were less than 0.02 feet. The local accuracy between leveled marks at each gage site is +/- 0.02 feet.

4.4.2. USED Benchmarks

Leveling was performed at certain USED benchmarks, in cases where the recovered benchmark was found in a location not suitable for GPS. The table below summarizes the results of our survey of USED benchmarks. Some of the elevations shown were established by leveling from a temporary benchmark, while others were surveyed directly with GPS methods.

In six locations, no benchmark was recovered after searching for all of the alternates identified on the spreadsheet provided by the USACE. In the general area identified as Tisdale Weir, none of the identified benchmarks were recovered; however we leveled to a USED Brass Disc that we found in the North End of Weir with the stamping illegible. Some additional "USED" stamped monuments not identified in the table below were also surveyed as reference marks at stream gage locations. Elevations for these monuments can be found on the stream gage leveling forms.

Area ID	General Area	Station ID	Benchmark NAVD88 Elevation (U.S. FT)	NAVD88 Survey Date	Notes
1	Collinsville	USGS BM 1906 5B	8.33	26-Jan-10	
2	Rio Vista	USC & GS BM D-585-JS1797	20.19	25-Jan-10	
3	Isleton	Bm # 15	22.59	27-Jan-10	
4	Walnut Grove	USED # 26 (marked #50)	26.82	27-Jan-10	
	Courtland	USED 61	25.17	2-Feb-10	
5	Courtland		27.14	28-Jan-10	No Requested Stations Found - Surveyed "USE64" in Same Vicinity
6	Freeport	USED 81	28.80	2-Feb-10	
7	Sacramento 1	Tidal 3 -JS2309	34.44	3-Feb-10	
	Sacramento 2	Weir # 1	44.64	3-Feb-10	AKA "SAI-179"
8	Sacramento 2	Weir # 2	44.50	3-Feb-10	AKA "SA-178"
9	Sacramento 3	USGS 21 B	23.68	15-Apr-10	
10	Knights Landing	USED 1001	43.65	3-Mar-10	
11	Tisdale Wier		44.22	3-Mar-10	No Requested Stations Found - Surveyed USED Brass Disc in North End of Weir, stamping illegible
	Middle of Upper Sac	USED 1259 Moulton Weir	84.41	16-Feb-10	
14	Middle of Upper Sac	USED 1257	84.43	16-Feb-10	
18	Yuba/Marysville	USED 3111	73.28	14-Apr-10	
12	Butte		No Benchmarks Recovered		
13	Colusa		No Benchmarks Recovered		
15	Upper/middle sac		No Benchmarks Recovered		
16	Upper Sac River		No Benchmarks Recovered		
17	Lower Feather		No Benchmarks Recovered		
19	Upper Feather		No Benchmarks Recovered		

Table 1: Summary of USED Benchmarks

4.4.3. Weir Surveys

A small amount of leveling was also performed at various weir survey locations for the purpose of establishing NAVD88 elevations on hard features identified in topographic survey drawings. These will be used as reference points to convert these drawings to the NAVD88 datum. Conversion will be done as part of Task 6.

The tables below show comparisons between the surveyed NAVD88 elevations and the NGVD29 elevation of certain benchmarks and features shown on CAD drawings provided by the USACE. At each weir site an average difference has been calculated for use in converting NGVD29 elevations shown on the drawing to NAVD88.

All elevations at any one weir site were determined through differential leveling from a single GPS-established reference mark, with the exception of the Fremont Weir, where a reference mark was established by GPS methods at each end of the weir. Estimated accuracies are +/- 0.02 feet within each closed level loop, and +/- 0.33 feet relative to the National Spatial Reference System (NSRS) and NAVD88.

Moulton Weir				
Point	**NGVD29 Elevation (m)**	**NGVD29 Elevation (USFT)**	**NAVD88 Elevation (USFT)**	**Difference (USFT)**
South Headwall / USED 1257	25.15	82.51	84.43	1.92
North Headwall / USED 1259	25.19	82.64	84.41	1.77
Weir Crest - South End	22.49	73.79	75.87	2.08
Weir Crest - Midpoint	22.55	73.98	75.92	1.94
Weir Crest - North End	22.56	74.02	75.99	1.97
			Average Difference:	**1.94**

Table 2- Elevations for Moulton Weir

Fremont Weir				
Point	**NGVD29 Elevation (m)**	**NGVD29 Elevation (USFT)**	**NAVD88 Elevation (USFT)**	**Difference (USFT)**
East Headwall / "Fremont"	11.84	38.85	41.11	2.26
Weir Crest - East End	9.25	30.35	32.67	2.32
Weir Crest - West End	9.21	30.22	32.42	2.20
		0.00	**Average Difference:**	**2.26**

Table 3- Elevations for Fremont Weir

Colusa Weir				
Point	NGVD29 Elevation (m)	NGVD29 Elevation (USFT)	NAVD88 Elevation (USFT)	Difference (USFT)
North End Weir @ Centerline	17.94	58.86	60.77	1.91
North End Weir @ West Edge	17.86	58.60	60.62	2.02
North End Weir @ East Edge	17.86	58.60	60.58	1.98
			Average Difference:	**1.97**

Table 4- Elevations for Colusa Weir

Tisdale Weir				
Point	NGVD29 Elevation (m)	NGVD29 Elevation (USFT)	NAVD88 Elevation (USFT)	Difference (USFT)
Weir Crest - North End / USED 1129	12.82	42.06	44.22	2.16
North Headwall / "RP#1"	14.69	48.20	50.33	2.13
			Average Difference:	**2.15**

Table 5- Elevations for Tisdale Weir

Sacramento Weir				
Point	NGVD29 Elevation (m)	NGVD29 Elevation (USFT)	NAVD88 Elevation (USFT)	Difference (USFT)
F9 01	11.90	39.04	41.28	2.24
SA1-229	12.44	40.81	43.09	2.28
Conc Wall NE Corner Weir / SA-178	12.89	42.29	44.50	2.21
Weir Crest - North End	6.28	20.60	22.87	2.27
Weir Crest - South End	6.39	20.96	22.91	1.95
			Average Difference:	**2.26**

Table 6- Elevations for Sacramento Weir

5. Task 4 Summary

Task Objective

Readjust Comprehensive Study GPS Network Files into NAVD88.

5.1. Lower Sacramento

The readjustment of the Lower Sacramento Comprehensive Study GPS Network was performed by a third party contracted directly by the USACE. The readjustment was based on stations surveyed by Bestor and provided to the USACE.

The original Lower Sacramento Comprehensive Study network adjustment was performed using a software package that our office was unable to work with (Geolab). Also, digital files containing the original GPS data or processed vectors were not available. Data available from the network consisted of only scanned printouts of the adjusted coordinate listing, and some of the adjustment results.

The USACE contacted the vendor of the Geolab software, Bitwise Solutions, and arranged to transpose the scanned adjustment reports and have Bitwise readjust the network using a current software release. A report documenting the readjustment was prepared by Bitwise Solutions and is included in this submittal in the Task 4 folder, in the "Bitwise Readjustment" subdirectory.

Bestor's involvement was limited to providing surveyed NAVD88 elevations for a sufficient number of stations to facilitate the readjustment, and reviewing the comparison of checkpoint elevations to elevations derived using a surface created from the readjusted elevations.

5.2. Lower Feather River

The comprehensive study GPS network files for the Lower Feather River mapping area were delivered to us by the USACE in Trimble Geomatics Office format, the format generally used by our office for GPS processing.

The project was opened and converted to our current software version (ver. 1.63) and all NGVD29 control elevations were discarded. Our resurveyed NAVD88 elevations were added as constraints, and a least-squares network adjustment was performed. GEOID09 was used in the readjustment.

A horizontal and vertical adjustment was performed, however only the readjusted heights were used in the development of the conversion surface.

A network adjustment report is included in the Task 4 submittal folder, as Appendix F.

5.3. Upper Feather River

HJW arranged to purchase the comprehensive study GPS network files for the Upper Feather River area from the original surveyor, Andregg Geomatics, which were delivered to Bestor on 28 September 2010.

Files were delivered in a Trimble .A1 archive format, which we were able to import into our Trimble Geomatis Office software ver. 1.63. All NGVD29 control point elevations were discarded, and a least-squares network adjustment was performed constraining the NAVD88 resurveyed elevations. GEOID09 was used in the readjustment.

A horizontal and vertical adjustment was performed, however only the readjusted heights were used in the development of the conversion surface.

A network adjustment report is included in the Task 4 submittal folder, as Appendix G, along with the adjustment data, in the “Trimble Geomatics Office” directory.

5.4. Upper Sacramento

The comprehensive study GPS network files for the USACE were provided to us by the USACE in an Ashtech format that is no longer supported. We were therefore not able to open the network files directly with software available to us, however we were able to import the processed vectors into Magellan’s GNSS Solutions software, version 3.10.10, which we obtained specifically to work with this data.
s
The Upper Sacramento network files consisted of five separate Ashtech projects, and were readjusted as five separate projects in GNSS Solutions, named “CHICO”, “DAVIS”, “SCID”, “YUBANRTH” & “YUBASTH”

Elevations and coordinates of resurveyed stations were added to the respective projects as constraints, and a least-squares adjustment was performed using the Fillnet component of the GNSS Solutions software. Since the purpose of the readjustment was to establish NAVD88 heights, no attempt was made to perform a precise horizontal readjustment, or to resolve horizontal misclosures.

Survey reports including adjustment reports generated using GNSS Solutions software are included as Appendices H through L, in the Task 4 submittal folder. The network adjustment data is also contained within the “GNSS Solutions” folder in the Task 4 submittal folder.

5.5 Checkpoint Evaluation of Readjusted Networks

For each of the four readjusted Comprehensive Study GPS Networks, the readjusted elevations of a selection of stations were compared to newly surveyed checkpoint elevations of the same stations. Checkpoint elevations were determined by the use of the network RTK methodology described in Section 4.3. The surveyed checkpoint elevations were for evaluation purposes only and were not used in the network readjustments. This comparison provides a measure of how well the readjusted elevations can be expected to represent resurveyed elevations. This is strictly an evaluation of the network readjustments, and is completely independent of the development of the conversion surface (Task 5). Tables and statistics for the checkpoint evaluation of each of the four readjusted networks are shown below. All units shown are U.S. Survey Feet.

Upper Sacramento Readjustment	
RMS Error	0.152
Mean	-0.063
Median	-0.056
Mode	#N/A
Standard Deviation	0.14208
Sample Variance	0.02019
Kurtosis	0.92899
Skewness	-0.61053
Range	0.594
Minimum	-0.404
Maximum	0.19
Sum	-1.131
Count	18

Table 7- Checkpoint analysis of Upper Sacramento readjusted network

One checkpoint, point # 388, was rejected and not used in the RMS calculation for the Upper Sacramento region. The point was a statistical outlier for the dataset, outside the 95% confidence interval (1.96 sigma). The large observed difference between the readjusted elevation and surveyed elevation is likely indicative that the monument has been disturbed since the time of the original Comprehensive Study survey.

Upper Feather Readjustment	
RMS Error	0.049
Mean	-0.015
Median	-0.017
Mode	#N/A
Standard Deviation	0.04922466
Sample Variance	0.00242307
Kurtosis	-1.1690365
Skewness	-0.0034134
Range	0.146
Minimum	-0.087
Maximum	0.059
Sum	-0.152
Count	10

Table 8- Checkpoint analysis of Upper Feather readjusted network

Lower Feather Readjustment	
RMS Error	0.066
Mean	-0.035
Median	-0.026
Mode	#N/A
Standard Deviation	0.059122
Sample Variance	0.003495
Kurtosis	1.771227
Skewness	-0.86275
Range	0.198
Minimum	-0.158
Maximum	0.04
Sum	-0.314
Count	9

Table 9- Checkpoint analysis of Lower Feather readjusted network

Lower Sacramento Readjustment	
RMS Error	0.196
Mean	-0.095
Median	-0.091
Mode	#N/A
Standard Deviation	0.185508
Sample Variance	0.034413
Kurtosis	1.979844
Skewness	1.232794
Range	0.556
Minimum	-0.292
Maximum	0.264
Sum	-0.662
Count	7

Table 10- Checkpoint analysis of the Lower Sacramento readjusted network

One checkpoint, Point # 108, was rejected and not used in the RMS calculation for the Lower Sacramento region. The point was a statistical outlier for the dataset, outside the 95% confidence interval (1.96 sigma). The large observed difference between the readjusted elevation and surveyed elevation is likely indicative that the monument has been disturbed since the time of the original Comprehensive Study survey. Additionally two points, #86 and # 159, were surveyed for checkpoint purposes; however no readjusted elevations were provided for these stations so no comparison could be made.

Conclusion

RMS vertical errors for the four network readjustments range between 0.05' and 0.20' based on our checkpoint evaluation. This is consistent with our expected vertical accuracy of our survey methods used to resurvey the control stations and checkpoints, demonstrating that all four readjustments were successful. The readjusted coordinates represent accurate NAVD88 elevations, and are suitable for use in development of the conversion surface.

6. Task 5 Summary

Task Objective
Develop conversion surface which will be used to convert Comprehensive Study DTM data from original datum into NAVD88.

6.1 Methodology

The Comprehensive Study DTM data was tied into multiple unreliable control networks referencing NGVD29. Because of this lack of reliability, Vertcon alone does not provide an adequate conversion to NAVD88. Therefore, in addition to the Vertcon correction, we also model the "Vertcon error", which we define as the difference between the Vertcon correction and the measured correction at known points. The "Vertcon error" for this network of points acts as the conversion surface; an enhancement to the transformation from NGVD29 to NAVD88 that is applicable only to this dataset- the Comprehensive Study DTM. For clarity, it is important to note that our use of the term "Vertcon error" does not imply that there is error in Vertcon, but rather it describes the error that remains for this particular dataset after applying the Vertcon transformation.

The following steps were taken to develop the conversion surface:

1. For each surveyed point in the control network, we determined the difference between NGVD29 and NAVD88 using Vertcon.
2. We subtracted the Vertcon-derived value of NAVD88 from the newly-surveyed, adjusted value
3. A TIN was created of these differences, which we call the "Vertcon error"
4. The TIN was converted into a DEM at 1,000' point posting
5. CORPSCON was used to determine Vertcon correction values, based upon the horizontal position of each DEM point
6. At each DEM point, the "Vertcon error" is added to the Vertcon value. These are the final correction values that can be applied to the Comprehensive Study DTM in order to convert into NAVD88.
7. The DEM was then represented as a TIN for subsequent contour generation (at 0.1' interval) and utilization as a conversion surface.

Input data used in the development of the conversion surface was provided by Bestor Engineers, as a result of their completion of Task 4. All primary and secondary control points that were surveyed had the following information provided as input to the conversion surface computation:

a. Station Name
b. Horizontal position (Northing and Easting in SPCS NAD 83 Zone II – US Foot)
c. Height in NGVD29
d. Height in NAVD88 – resulting from field survey (primary points), or as a result of network adjustment (secondary points)
e. Vertical differences between NAVD88 and NGVD29 at these locations.

Initially, the team had planned to generate each of the four conversion surfaces independently of each other; that is to say that there will be one conversion surface for Upper Feather, Lower Feather, Upper Sacramento and Lower Sacramento, and each surface will utilize only control points (primary / secondary) that were part of each region's original survey. This approach was modified after determining that the control networks as planned did not provide suitable geometry for circumscribing the respective mapping areas for interpolation. The solution to this was to create two (rather than four) conversion surfaces.

1. Conversion surface 1 ("CSurface 1"): Lower Feather River Area
2. Conversion surface 2 ("CSurface 2): Lower Sacramento, Upper Sacramento, Upper Feather Areas.

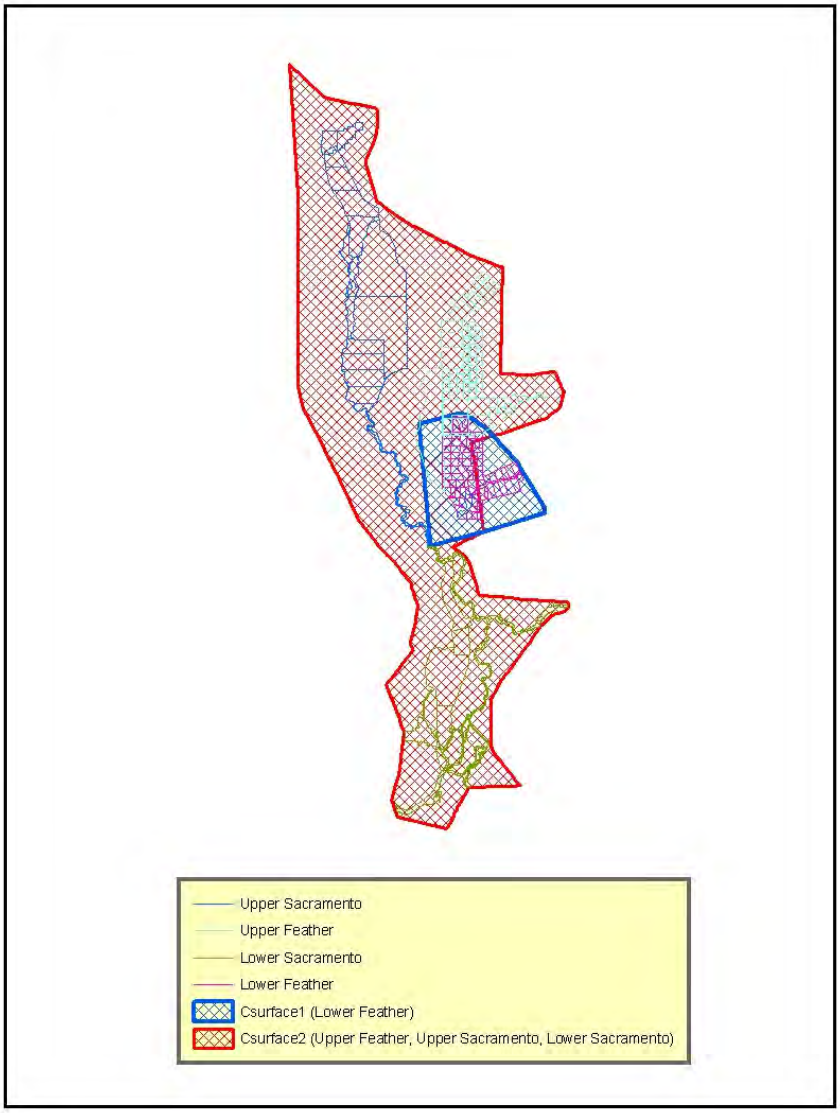

Figure 2: Conversion surface 1 in blue, conversion surface 2 in red

6.1.1 Conversion Surface 1- Lower Feather River Area

For the Lower Feather River area, it was determined that due to the control distribution, another control point external to the planned network was necessary for proper interpolation of the conversion surface. The USACE provided an additional point- NGS station Q213 (see table below) to use for this purpose. All data in the following table is with respect to State Plane Coordinates, Zone 2, NAD83, USFT.

PT	N	E	EL29	EL88	EL_DIFF	VERTCON	VCON_ERROR
Q 213	2229142.12	6816654.03	1814.763	1817.221	2.458	2.451	0.007

PT = Station Name
N = Northing
E = Easting
EL29 = Elevation in NGVD 29
EL88 = Elevation in NAVD 88
EL_DIFF = Elevation difference EL88 – EL29
VERTCON = Vertcon value
VCON_ERROR = Vertcon error values

Table 11- NGS point provided by USACE to complete conversion surface 1 TIN

Conversion Surface 1 covers 403 square miles and utilizes 57 surveyed points. One point was omitted from the surface computation after it was determined to be in error, as it was exhibiting a spike in the surface. Nine checkpoints were tested for this conversion surface.

6.1.2 Conversion Surface 2- Lower Sacramento, Upper Sacramento and Upper Feather River Areas

In Conversion Surface 2, there were four points common to the Upper Sacramento and Lower Sacramento areas that Bestor had surveyed and which were also computed independently by adjustment of an existing dataset. In the conversion surface, we used only the surveyed points in this case. The four points and surveyed coordinates are listed in the table below.

PT	N	E	EL29	EL88	EL_DIFF	VERTCON	VCON_ERROR
AYRES 494	2013742.95	6654790.27	47.621	49.474	1.852	2.661	0.809
AYRES 500	2014208.17	6663022.65	38.520	40.318	1.798	2.694	0.896
DUFOUR RM 2	2039908.19	6606054.47	64.370	66.054	1.684	2.772	1.088
Q858	2069993.26	6697206.30	44.560	46.228	1.667	2.516	0.849

Table 12- Surveyed points used in Upper Sacramento area conversion surface

It was necessary to add three more control points in order to have proper TIN interpolation of the conversion surface. The USACE provided HJW with three additional NGS points, as listed in the table below.

PT	N	E	EL29	EL88	EL_DIFF	VERTCON	VCON_ERROR
S 138	2585481.44	6736182.87	4850.200	4853.616	3.416	3.428	-0.012
Q 213	2229142.12	6816654.03	1814.763	1817.221	2.458	2.451	0.007
G 900	2341925.07	6669223.60	320.925	323.306	2.381	2.375	0.006

Table 13- Additional NGS points provided by USACE for Conversion Surface 2

Conversion Surface 2 covers 3,567 square miles and utilizes 612 surveyed points. Three points were classified as blunders and omitted from the surface computation as they were showing spikes in the conversion surface. 36 checkpoints were used in this conversion surface. Section 6.3 lists the supporting data files and deliverables applicable to this conversion surface.

6.2 Checkpoint Comparison & Conversion Surface QA/QC

Based on the readjusted elevations from the Comprehensive Study GPS networks, two NGVD29 to NAVD88 conversion surfaces were developed for use in converting the mapping to the NAVD88 datum. Methodology for the development of the conversion surfaces was determined by coordination between HJW and the USACE, with minimal involvement from Bestor.

Bestor's role in the QA/QC of the conversion surfaces was primarily the comparison of the elevations of surveyed checkpoints to elevations derived using the conversion surface.

Some outlier checkpoints with excessively high residuals were rejected, and review of the locations of the surveyed monuments found that most of these were in locations that were likely to have been disturbed. In all cases any rejected checkpoints were statistical outliers that were outside the 95% confidence interval (1.96 sigma) for the data set.

A comparison of nine checkpoints to elevations derived using the conversion surface yields the following statistics for the residuals (units are U.S. Survey Feet):

Lower Feather	
RMSE	0.077
Mean	0.005
Median	-0.006
Standard Deviation	0.081
Sample Variance	0.007
Range	0.313
Minimum	-0.124
Maximum	0.189
Sum	0.048
Count	9

Table 14- Checkpoints results for conversion surface 1- Lower Feather River Area

A similar comparison using 36 checkpoints from the remaining three areas (and rejecting points # 388 & 504) gives the following:

Lower / Upper Sacto & Upper Feather	
RMSE	0.133
Mean	-0.032
Median	-0.044
Standard Deviation	0.131
Sample Variance	0.017
Range	0.573
Minimum	-0.296
Maximum	0.277
Sum	-1.144
Count	36

Table 15- Checkpoint results for conversion surface 2- Lower and Upper Sacramento, Upper Feather River Areas

The calculated RMSEz for the two sets of checkpoints indicate that the conversion surfaces should be effective in accurately converting the comprehensive study mapping files to the NAVD88 datum, as the accuracy of surveyed checkpoints will exceed most all map accuracy standards used for aerial mapping that may have been used in the development of the comprehensive study mapping.

6.3 Conclusion

Two conversion surfaces were generated for the four mapping areas (upper and lower Feather, and upper and lower Sacramento River areas) by incorporating measurements of height differences between NGVD29 and NAVD88 at known primary survey points and secondary points determined through network readjustments. This allows for the determination of errors that remain after the Vertcon transformation between datums. Checkpoint analysis of the conversion surface indicates accuracies that are well within mapping specifications for the Comprehensive Study DTM.

Conversion Surface 1

- Applicable area: Lower Feather River mapping area
- Based upon checkpoint comparisons, the RMSE of 0.077' indicates that tested points are well within the requirements of converting the Comprehensive Study DTM data.

Conversion Surface 2

- Applicable areas: Lower Sacramento, Upper Sacramento, Upper Feather River mapping areas.
- Based upon checkpoint comparisons, the RMSE of 0.133' indicates that tested points are well within the requirements for converting the Comprehensive Study DTM data.

Instructions for use of the Conversion Surfaces
Both conversion surfaces area applicable only to the Comprehensive Study DTM data, which was collected in the late 1990's and early 2000's, by three vendors, as illustrated in the introduction to this report. The conversion surface is used to convert elevations from the instance of NGVD29 used in the Comprehensive Study project, into NAVD88. Task 6 is dedicated to the application of this conversion surface to the DTM files.

6.4 Listing of Task 5 Deliverables and Data Directory Index

1. Report summarizing Tasks 1-5, after having undergone independent technical review, including assessment of uncertainty based upon values at checkpoints. Final report in digital and three paper hardcopies.

2. ESRI TIN of conversion surface
    a. Conversion Surface/CSurface1/ESRI/TIN/
        i. CSurf1_vconer
        ii. CSurface1
    b. Conversion Surface/CSurface2/ESRI/TIN/
        i. CSurf2_vconer
        ii. CSurface2

3. Contour map of conversion surface in shapefile format
    a. Conversion Surface\CSurface1\ESRI\Shp\Csurface1_contourlines.shp
    b. Conversion Surface\CSurface2\ESRI\Shp\Csurface2_contourlines.shp

4. Shapefile of final control points- NGVD29 and new values
    a. Conversion Surface/CSurface1/ESRI/Shp/
        i. Csurface1_points_primary.shp
        ii. Csurface1_points_secondary.shp
    b. Conversion Surface/CSurface2/ESRI/Shp/
        i. Csurface2_points_primary.shp
        ii. Csurface2_points_secondary.shp

5. Approval of conversion surface by PLS
    a. Attached: "100526 Conversion Surface Certification.pdf"

6. Map of checkpoints and uncertainties
    a. Conversion Surface\CSurface1\PDF\ CSurface1_CheckpointandUncertaintiesMap.pdf
    b. Conversion Surface\CSurface2\PDF\ CSurface2_CheckpointandUncertaintiesMap.pdf

7. Conversion surface in shapefile format
    a. See item 4- shapefile of final control points. This contains the conversion surface points as well.

8. Conversion surface in Arc Generate ASCII format
    a. Conversion Surface\CSurface1\ESRI\ArcGenerate
    b. Conversion Surface\CSurface2\ESRI\ArcGenerate

9. Conversion surface in InRoads V8 format
    a. Conversion Surface\CSurface1\Bentley\InRoads
    b. Conversion Surface\CSurface2\Bentley\InRoads

10. Conversion surface and contours in DGN format
    a. Conversion Surface\CSurface1\Bentley\Microstation
    b. Conversion Surface\CSurface2\Bentley\Microstation

11. Conversion surface and contours in Microstation V8 ASCII InRoads format
    a. Conversion Surface\CSurface1\Bentley\InRoads
        i. CSurface1_r.dat
        ii. CSurface1_e.dat
    b. Conversion Surface\CSurface2\Bentley\InRoads
        i. CSurface2_r.dat
        ii. CSurface2_e.dat

12. Paper map of conversion surface (22" X 34")
    a. Conversion Surface\CSurface1\PDF\CSurface1_ConversionSurfaceMap.pdf
    b. Conversion Surface\CSurface2\PDF\CSurface2_ConversionSurfaceMap.pdf

13. Quality control certification
    a. Considered complete per PLS approval document "100526 Conversion Surface Certification.pdf"

14. Raw GPS files
    a. All GPS data that was collected for this project is within the RTK Survey Data\ directory.

15. Trimble Geomatics office project
    a. See item 17 "Machine-readable unadjusted and adjusted network files"

16. GPS log sheets and field notes (digital version)
    a. Field notes: Stream Gage Level Data\Stream Gage Benchmark Forms.pdf
    b. USED Benchmarks\Surveyed USED Benchmarks.xls
    c. RTK Survey Data\
        i. 671305 - Lower Sac RTK
        ii. 671305 - Upper Sac RTK – CSDS
        iii. 671305 - Upper Sac RTK – Topnet
    d. Monument Photos & GIS Data\

17. Machine-readable unadjusted and adjusted network files
    a. Network Readjustment Files\
        i. Bitwise (Lower Sacramento Area)
        ii. GNSS Solutions
        iii. Trimble Geomatics Office
            1. Andregg Upper Feather
            2. Towill Feather River

18. Survey results at stream gages
    a. Field notes: Stream Gage Level Data\Stream Gage Benchmark Forms.pdf

Converted DTM Data- Two tiles in the “Converted Tiles” directory.

1. Microstation DGN V8, contours [planimetrics and DTM not converted for these two tiles; not present in input data]
2. Microstation DTM files
3. Microstation DTM files in ASCII format
4. Metadata
5. ESRI DTM files
6. ESRI shapefiles of contours, breaklines, planimetrics, exterior boundaries, etc.
7. ESRI DTM ArcGenerate ASCII files (interior, exterior, etc.)
8. DWG files contours [planimetrics and DTM not converted for these two tiles; not present in input data].

ENCLOSURE D

GEOMORPHOLOGY ASSESSMENTS

WILLIAM LETTIS & ASSOCIATES, INC.
1777 Botelho Drive, Suite 262, Walnut Creek, California 94596
tel (925) 256-6070 fax (925) 256-6076

September 8, 2009

Mr. Juan Vargas
URS Corporation
2870 Gateway Oaks Drive, Suite 150
Sacramento, CA 95833

RE: Surficial geologic mapping and geomorphic assessment, California Department of Water Resources Urban Levees, Wadsworth Canal, Sutter County, California

Dear Mr. Vargas:

This memorandum presents the surficial geologic mapping and preliminary geomorphic assessment of the Wadsworth Canal area, for the California Department of Water Resources (DWR) Urban Project Levees geotechnical characterization. The goal of this mapping and geomorphic assessment is to provide information on the type and distribution of surface and shallow subsurface deposits that likely underlie the project levees along the canal, with respect to potential levee underseepage. This letter presents the technical approach, surficial geologic map, conceptual geomorphic model, and initial results based on map analysis and preliminary review of Phase 1 geotechnical data.

We appreciated the opportunity to provide these geomorphic and geologic data and preliminary interpretations of the shallow stratigraphic conditions in the Wadsworth Canal study area. Please do not hesitate to call either of the undersigned if there are any questions or comments.

Respectfully,

WILLIAM LETTIS & ASSOCIATES, INC.

Justin Pearce, C.E.G. 2421
Senior Project Geologist
(925) 395-2035

Keith Kelson, C.E.G. 1610
Principal Geologist
(925) 395-2032

1.0 Approach

The approach to developing a surficial geologic map of the Wadsworth Canal area (Figure 1, Plate 1) consisted of analysis of the following data: Aerial photography (black and white stereo-pairs taken in 1937, ~1:20,000-scale); early topographic maps (USGS, 1911); published surficial geologic maps (Helley and Harwood, 1985); early and modern soil survey maps (Strahorn et al., 1911; Lytle, et al., 1988); field reconnaissance visit on June, 22, 2007, and other maps and documents (i.e., Chambers, 2002). Synthesis of these data allow for the development of a detailed surficial geologic map that provides an initial understanding of primary geomorphic processes that have acted in the study area during recent geologic and historical time. Through this mapping, primary geomorphic features and associated surficial geologic deposits are identified, such as abandoned paleochannels, marsh and basin deposits, and other features commonly associated with flood basins adjacent to large, active river systems.

The surficial geologic map was developed at the nominal scale of the aerial photography (1:20,000). This scale establishes the resolution of the map (Plate 1). The map unit contacts shown on the surficial geologic map should be considered approximate, and accurate to no more than about 30 feet on either side of the line shown on the map. The 1937 aerial photographs are the primary data set for interpreting the surficial geologic deposits because: (1) they are the oldest high-quality images available and pre-date much of the cultivation and landscape alteration within present-day Sutter County (Figure 2); and, (2) because these data represent a close approximation to the surficial deposits that were likely present at the ground surface prior to construction of the levees. The 1937 photographs generally were taken in later summer or early autumn (i.e., August). By 1937, the area had experienced moderate cultivation that locally obscures geomorphic conditions. However, integration of data from the 1937 photographs, old and recent topography, existing geologic maps, existing soil surveys and historical documents provides sufficient information to delineate many of the pre-historical and historical surficial deposits in detail. Taken together, these data provide key insights to the geomorphic processes and resulting deposits that may affect levee underseepage.

Additional floodplain deposition may have occurred after 1937, due to flood overflows, levee overtopping, or localized levee failure. A time series analysis that interprets successive aerial photographs taken after major flood events (i.e., 1955) or known local levee failures (i.e., 1986) may reveal additional information on surficial deposits in the Wadsworth Canal area. However, such analyses are beyond the scope of this project. The data and interpretations presented herein address the primary goal of characterizing the type and distribution of deposits likely present directly beneath the project levees.

1.1 Report Preparation Quality Control

The surficial geologic map data and geomorphic interpretations presented in this memorandum were subject to quality control and quality assurance procedures as required by the Levee Geotechnical Evaluation Project Management Plan (PMP). The surficial geologic map data developed by this study were reviewed for accuracy and completeness through an internal review and an independent technical review by Dr. Janet Sowers of WLA. Results of QA/QC review were documented using PMP Exhibit 2.2-3 (Independent Technical Review Report) and are kept on file according filing control plan. Subsurface data shown on diagrams were provided as draft information, and were not verified for accuracy or completeness by this study.

2.0 Geologic Setting

The Wadsworth Canal (WC) study area is southeast of the Sutter Buttes, a presently in-active and dissected rhyolitic and andesitic volcanic neck, and between the Sacrametno River to the west and the Feather River to the east (Figure 1). The WC levee addressed in this study borders the southeastern side of Wadsworth Canal from just north of Butte House Road to the eastern Sutter Bypass levee. The WC levee trends northeast-southwest, and ties in to the eastern Sutter Bypass levee (Figure 1).

The WC levee lies northeast of Sutter Basin, a low-lying area east of the Sacramento River where overflow and floodwaters produce a seasonally marshy area. Except for the Sutter Buttes, the land regional surface is nearly flat, and along the WC area gently slopes southwest at an elevation of about 40 to 50 feet. Construction of the WC levee was completed by 1924, and was subsequently enlarged in 1942 (DWR, 1976). Prior to cultural modification, surface water runoff in the WC area was delivered to the Sutter Basin via intermittent, meandering creeks and sloughs from the northern Central Valley, including: Snake River, Snake Slough, Little Blue Creek, and ephemeral channels emanating from the eastern side of Sutter Buttes. Presently, many of the natural drainages and channels have been replaced by linear ditches, agricultural drains, and canals (Figure 2).

3.0 Surficial Geologic Mapping

Published surficial geologic maps within the WC study area generalized the surficial deposits primarily as Quaternary basin deposits, with localized units of Quaternary alluvium (map unit Qa) and Quaternary Modesto Formation (lower member, map unit Qml) (Helley and Harwood, 1985). These map units were delineated by Helley and Harwood (1985) at a regional scale (i.e., 1:62,500). The current analysis of the WC uses this existing geologic framework as a basis for more detailed mapping of late Holocene alluvium and geomorphic features (Plate 1). The surficial geologic map units in the Wadsworth Canal study area are described below, in order from oldest to youngest.

The oldest map unit exposed in the study area is the Pliocene-Pleistocene tuff breccia (map unit QTm). This rock primarily comprises a peripheral topographic ring around the relatively high relief Sutter Buttes, and consists of consolidated coarse material derived from the volcanic rocks of the Buttes. This bedrock is exposed in the northwest corner of the WC map area (plate 1).

The Quaternary Riverbank Formation (lower and upper members) is exposed at the ground surface adjacent to the tuff breccia (map unit Qrl and Qru, Plate 1). This map unit does not directly underlie the project levees in this study area, but is present in the shallow subsurface as alluvial-fan deposits derived from the Sutter Buttes during the middle Pleistocene (about 400,000 to 200,000 years ago). The Riverbank Formation is semi-consolidated, and the top of the formation is marked by a hardpan (or, duripan) layer that is a product of soil-forming processes over substantial geologic time. This hardpan layer reflects an ancient land surface that is now buried by younger deposits. In WC area, the upper Riverbank formation is associated with the Sutter clay (Strahorn, et al., 1911), and Marcum clay loam with "siltstone" hardpan (Lytle, 1988).

The late Pleistocene Modesto Formation is exposed at the surface as alluvial-fan deposits emanating from southwestern Sutter Buttes, and is younger than, and inset into, the Riverbank Formation (Plate 1). This unit is divided into two members, a lower (older) unit that is about 42,000 to 29,000 years old (map unit Qml), and an upper member that is about 24,000 to 12,000 years old (map unit Qmu) (Helley and Harwood, 1985). The upper member in the map area is associated with sub-linear low ridges to the east of the WC that have not been completely covered by basin deposits. The Modesto Formation is locally associated with the Sutter sandy loam (Strahorn, et al., 1911), and the Olashes sandy loam (Lytle, et al., 1988); the sand consisting of volcanic lithologies indicating derivation from Sutter Buttes parent material. The latest Pleistocene Modesto Formation, in general, consists of unconsolidated sand, silt, and clay, and is associated with a moderate amount of secondary (pedogenic) clay accumulation that may form laterally continuous zones of low hydraulic conductivity.

Holocene deposits (less than 11,000 years old) in the WC map area consist of basin and alluvial deposits (Qb of Helley and Harwood [1985]; map unit Qn, Plate 1). These widespread basin deposits, about 4 to 8 feet thick, overlie the Modesto Formation. The soils developed on the basin deposits are generally the Gridley clay loam and Oswald clay (Stahorn et al., 1911; Lytle, et al., 1988), immature soils with fine-grained textures. Undifferentiated Quaternary alluvium (map unit Qa) is present near the western margin of the map area, deposited by pre-historic Butte Creek. Holocene alluvium is mapped at the surface as alluvial-fan deposits emanating from southwestern Sutter Buttes, and is younger than, and locally overlies the upper Modesto Formation. These deposits likely consist of poorly sorted mixtures of fine gravel, sand, and silt derived from the volcanic rocks of the Buttes. The Quaternary marsh deposits (map unit Qs, Plate 1) are present between the levees of the Sutter Bypass, and consist of fine grained deposits that are differentiated from basin deposits by generally being underwater or having standing water at the time when the 1937 photographs were taken.

Holocene alluvial channels (map unit Hch, Plate 1) are mapped as a network of moderately sinuous channels with southwesterly orientations. These channels appear to be mostly filled in with sediment on the 1937 photographs, and are not expressed as strong topographic lows in the ground surface. Many of these channels extend beyond, and therefore cross beneath, the eastern Sutter Bypass levee and WC levee (Plate 1). The infilling material in the basal portions of the channel consists of relatively loose, coarse material (i.e., sand), which fines upward into fine-grained, silt and clay. The channel deposits are tentatively associated with the Liveoak series, sandy clay loam soil (Lytle, et al., 1988).

Localized deposits related to the Holocene alluvial channels are in-stream bars (map unit Hb) that typically occur in the medial portions of the channels, and distributary fans (map unit Hdf) that occur where the channel morphology tapers out and the channel has deposited sediment on the basin floor. These two types of deposits are uncommon in the study area, and have been mapped only distant from the WC levee.

Historical alluvial channels (map unit Rch, Plate 1) also are mapped as a network of moderately sinuous channels that have southwesterly orientations toward Sutter Basin. The term "historical" is applied to deposits that are estimated to be less than 150 years old. The historical channels are differentiated from the slightly older Holocene channels on the basis of cross-cutting relationships, relative degree of geomorphic expression, and correlation with mapped creek positions on the 1911 USGS topographic map. The Wadsworth Canal levee overlies the former locations of these alluvial channels in several locations throughout its length (Plate 1).

4.0 Conceptual Geomorphic Model

Based on synthesis of surficial geologic mapping, early topographic maps, soil surveys, geologic maps, and review of readily available subsurface geotechnical information, this section presents a preliminary conceptual model describing general relationships among surface and subsurface deposits in the Wadsworth Canal area. This conceptual model provides a consistent basis for understanding the type and distribution of surficial geologic deposits, primary geomorphic processes, and shallow subsurface stratigraphy in the area.

The geologic deposits present at the surface and in the shallow subsurface are derived from three general source areas: (1) material eroded from the Sutter Buttes and transported to the adjacent low-lying basin floor forming modest alluvial fans (i.e. Riverbank and Modesto Formations); (2) material deposited on the basin floor as fine silt and clay settled from standing or slow moving floodwaters of large rivers (i.e., basin deposits); and, (3) material transported to the basin by the ephemeral creeks and sloughs that traversed the valley floor prior to present day modification (i.e., channel fill).

The WC project levee trends southwest, and is primarily underlain by clayey basin deposits with some silt and sand (Plate 1, Figure 3). The basin deposits rest directly on the upper Modesto Formation, the upper boundary of which is characterized by a clay hardpan horizon associated with a buried soil. The hard pan layer is generally observed as a very stiff to hard, lean to fat clay, 10YR ¾ colors (Munsell color notation) associated with locally increased density (i.e., blow counts, CPT tip resistance), and likely very low permeability. Thus, the upper Modesto Formation mapped in northwest potion of the map area extends below ground, and dips southeasterly beneath the project levee in the shallow subsurface.

Fine-grained basin deposits overlie the upper Modesto Formation near the WC levees (Figure 3). These deposits accumulated on the valley floor over geologic time resulting from flooding of the major rivers (i.e., Sacramento and/or Feather Rivers), tributaries draining Sutter Buttes, and sheetwash from the generally flat valley floor. This resulted in inundation of the basin with standing water, and subsequent settlement of silt and clay from suspension. The thickness of the basin deposits is about 4 to 8 feet, but locally may be thicker. Review of available Phase 1 and other existing geotechnical data (i.e., Chambers 2002) indicate medium stiff to very stiff relative density of the basin deposits. However, there is a substantial lateral and vertical variability in the hardness properties of the basin deposits.

Laterally cross-cutting, and vertically inset into the basin deposits, are the Holocene and Historical channel deposits (map units Hch and Rch, Plate 1). These southwest-trending alluvial channel deposits locally underlie the WC levee, and thus result in local differences in material textures beneath the levee (Figure 3). Field reconnaissance on June 22, 2007 reveals that the topographic expression of these channels has been obliterated by cultivation. However, sub-linear to curvilinear differences in ground color (i.e., darker strips) were observed in the cultivated fields in areas that potentially correlate with mapped channels, suggesting a contrast in materials in the shallow subsurface. Review of subsurface geotechnical data indicate that the channel fill deposits include a lower channel fill consisting of relatively loose, coarser material (i.e., sand), fining upward and grading into fine-grained silt and clay. Many of these channels extend across, and therefore continue beneath, the WC levees (Plate 1, Figure 3).

Figure 3 illustrates the relationships between the surficial channels, basin deposits, and shallow stratigraphy that underlie the WC project levee, wherein dense, semi-consolidated Pleistocene deposits are overlain by a layer of fine-grained basin deposits, locally cut by alluvial channel deposits.

5.0 Applications to the Urban Levee Project

Based on synthesis of the surficial geologic map with preliminary Phase 1 boring and cone penetrometer (CPT) data, and historical geotechnical subsurface exploration data (i.e., Chambers, 2002), the WC levee is underlain by relatively young fine-grained clay and sandy clay deposits that are laterally interrupted by local coarser channel fill deposits (Figure 3). Mud rotary borehole WSEWWC-002B penetrates a mapped surficial channel unit (Figure 3, Plate 1), and indicates the channel fill is silty sand that grades upward into clay, with an uncorrected SPT blow count of 5 blows per foot. This suggests locally loose and unconsolidated, and therefore likely permeable, material in the shallow subsurface. Initial review of subsurface boring profiles completed along the eastern landside of the Wadsworth Canal near the tie-in to the Sutter Bypass levee (Chambers, 2002) also shows relatively loose and soft sandy deposits (i.e., blow counts of 0 to 5) that are overlain by a layer of medium stiff clay-rich material.

Synthesis of the surficial mapping and geotechnical data indicate that subsurface stratigraphy the WC area locally may be conducive to levee underseepage. Shallow strata typically include denser and probably semi-cemented material (i.e., Modesto Formation) that likely contains a low-permeability hardpan horizon. The hardpan may or may not be laterally continuous, depending on post-depositional soil formation and erosional processes. The Modesto formation is overlain by about 4 to 6 feet of medium stiff to stiff clay (i.e., basin deposits). Surficial mapping indicates that the basin materials locally are cross-cut by relatively loose, sandy channel deposits; subsurface geotechnical data show lateral and vertical variations in texture and density that are probably related to buried channel deposits. Therefore, this shallow subsurface stratigraphy may promote levee underseepage along certain areas of the WC project levees where geologically young, loose, sandy channel material lies between the dense Pleistocene deposits and relatively stiff, low-permeability clay-rich surface “blanket” layer.

Lateral and vertical variability in the shallow subsurface deposits has resulted from past geomorphic processes. The conceptual subsurface stratigraphic framework suggests that stratigraphic relationships may promote localized levee underseepage, given certain hydraulic conditions. Further spatial analyses of the surficial geologic mapping and subsurface geotechnical exploration data are needed to better constrain and characterize areas that are most susceptible to underseepage in the study area.

6.0 Limitations

This geomorphic assessment and associated data interpretation have been performed in accordance with the standard of care commonly used as the state-of-practice in the geologic engineering profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this geographic area performing the same services under similar circumstances during the same time period.

Discussions of surface and subsurface conditions summarized in this technical memorandum are based on geologic interpretations of subsurface soil data at limited exploration locations available to this assessment through July of 2007. Variations in subsurface conditions may exist between exploration locations, and the project team may not be able to identify all adverse conditions in the levee and its foundation. This memorandum is for the use and benefit of DWR. Use by any other party is at their own discretion and risk.

7.0 References

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Strahorn, A.T., Mackie, W.W., Holmes, L.C., Westover, H.L., Van Duyne, C., 1911, Soil Survey of the Marysville Area, California; 1:62,500, USDA Bureau of Soils

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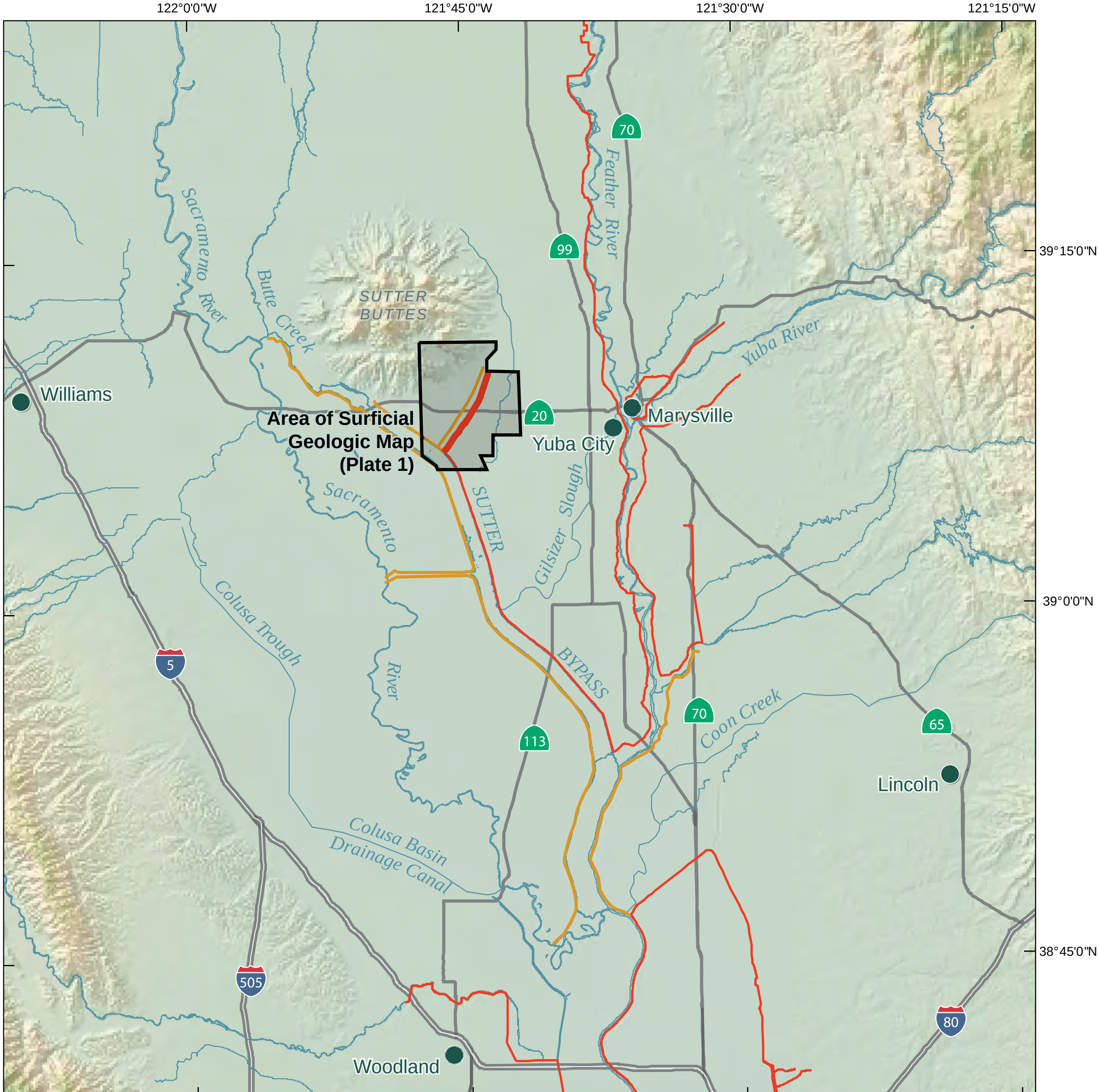


Sources: ESRI, 2006

Projection: UTM Zone 10, NAD83

Explanation

- Wadsworth Canal levee
- Urban project levee
- Other levee

N

0 5 10 mi

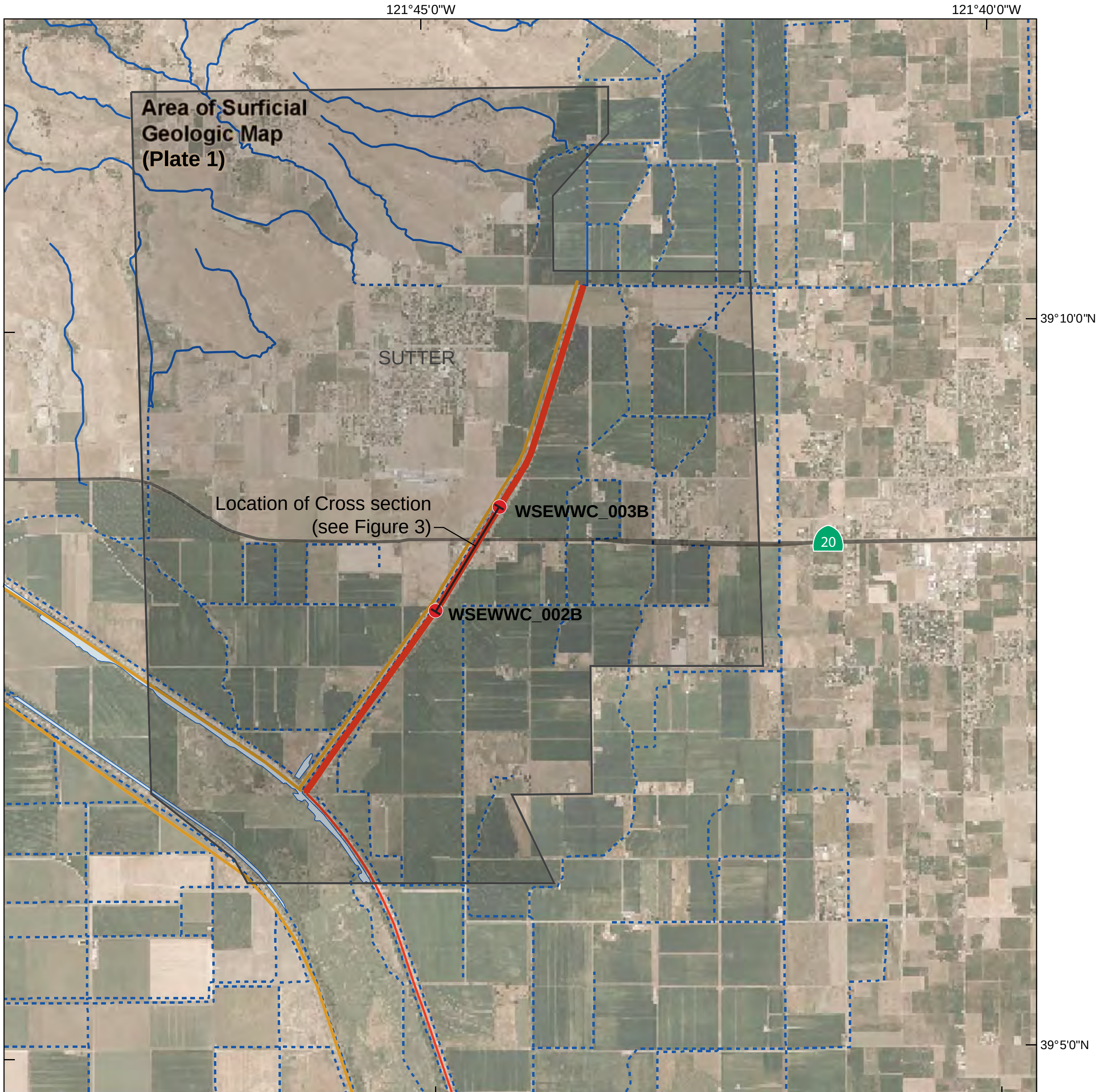
0 5 10 km

WLA

Map of Northern Central Valley, California

DWR URBAN LEVEE PROJECT

Figure 1



Sources: ESRI, 2006

Projection: UTM Zone 10, NAD83

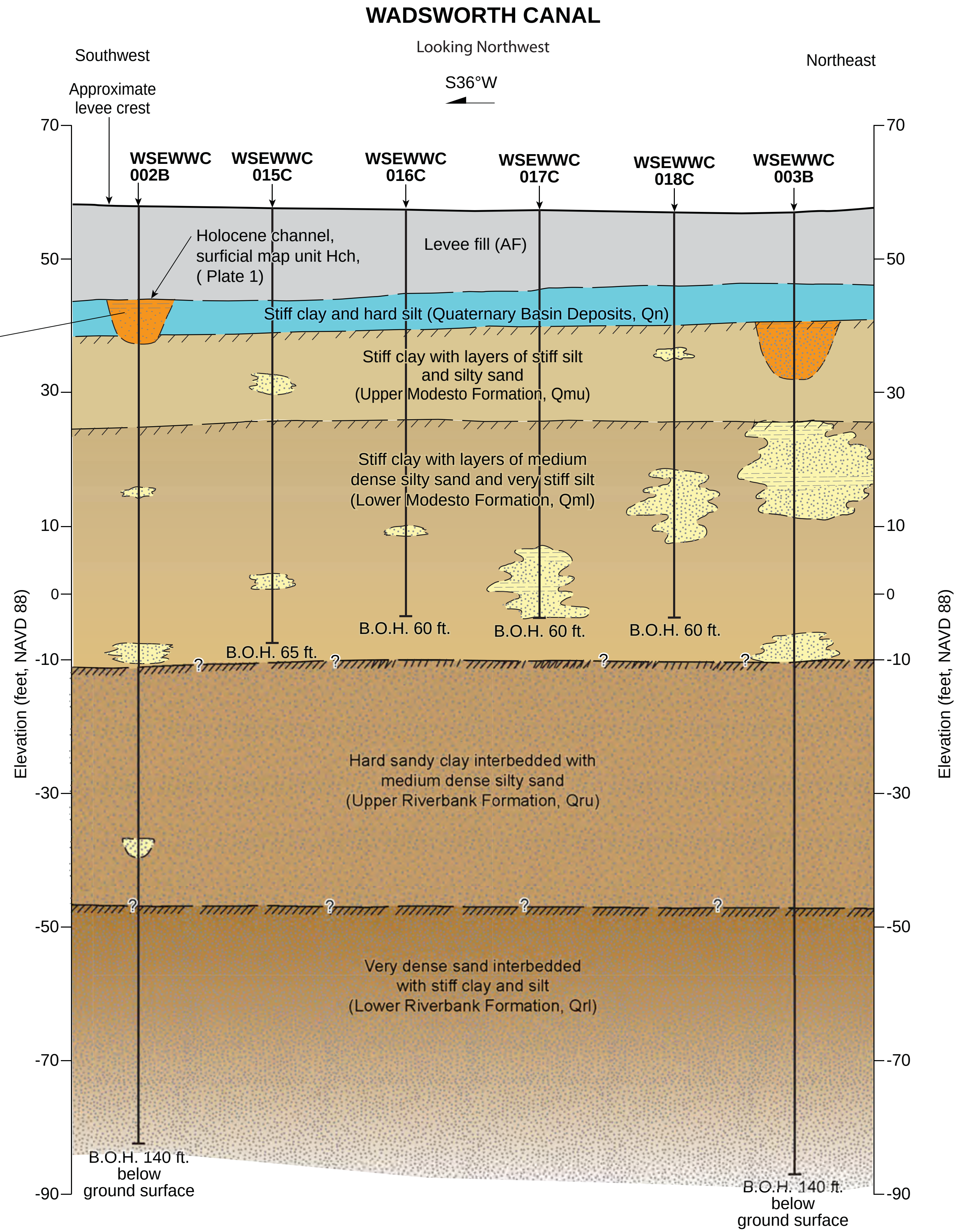
Explanation

- Creek, slough
- Canal, drain
- Wadsworth Canal Urban Project levee
- Other Urban Project levee
- Other levee

N

0 1 2 mi

0 1 2 km



Log description from borehole WSEWWC 002B

44.0 to 42.5 feet. Lean CLAY (CL) stiff; dark brown (10YR 3/3); moist; 90% medium plasticity; 10% fine sand

42.5 to 41.0 feet. SILT with SAND (ML) loose, dark yellowish brown (10YR 3/4); moist; 15% fine sand

41.0 to 39.0 feet. SILTY SAND (SM) medium dense; dark yellowish brown (10YR 3/4); moist; 75% fine to medium sand; 25% fines

42.5 to 41.5 SPT = 5 blows/foot

Explanation

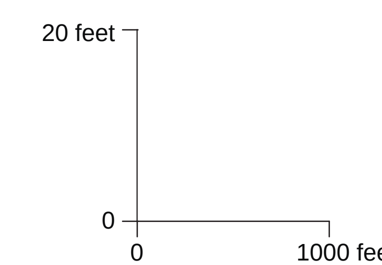
Channel identified on surficial geologic map or as fining upward sequence of sediments in boreholes

Localized sand and gravel; possible channel; interpreted from suburface borehole data

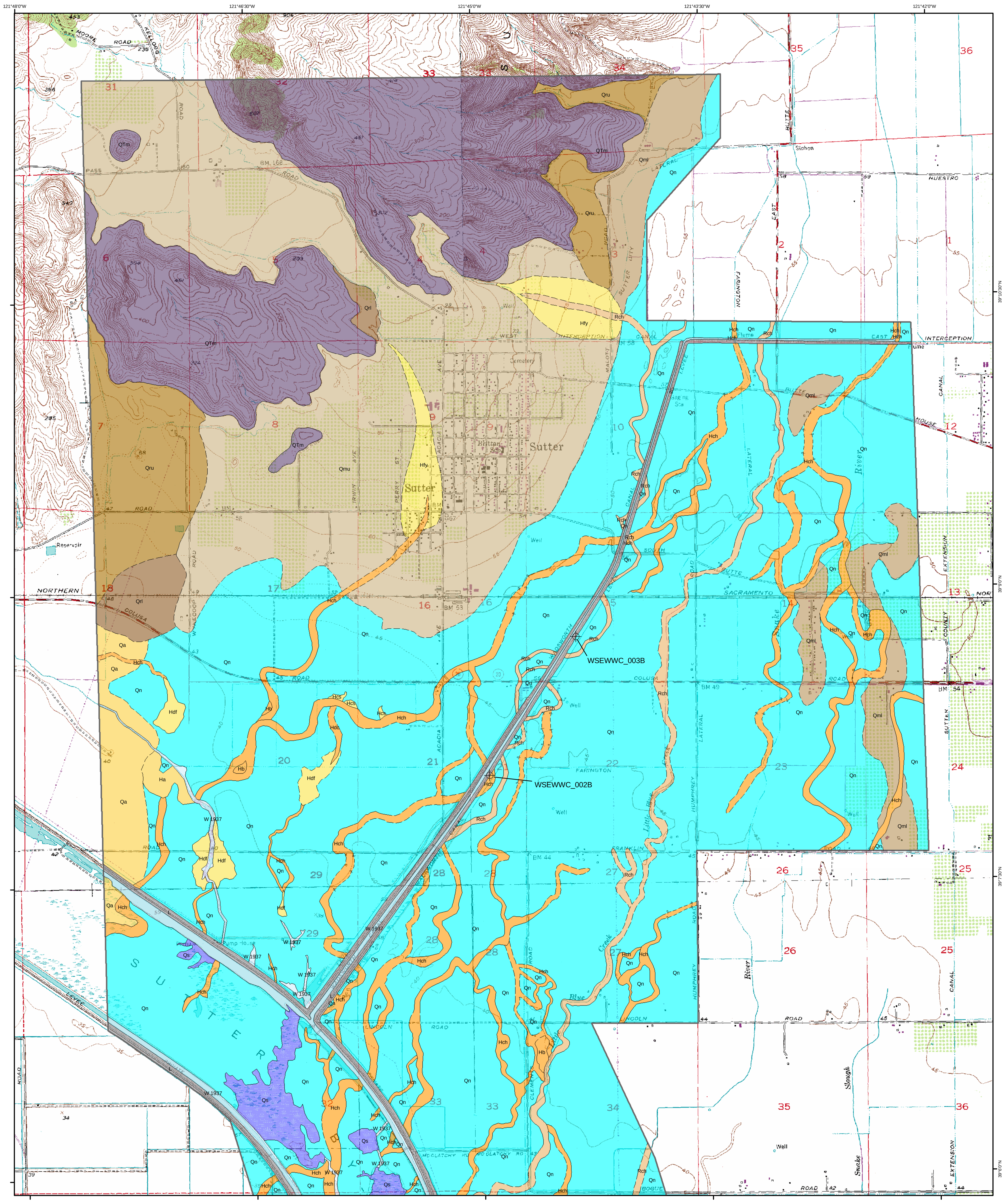
Moderate paleosol (hardpan)

Strong paleosol (hardpan)

Notes:
1. Borehole ground elevation values from Engeo, Inc. draft borehole logs as estimated from map (NAVD 88). Absolute elevations of geologic contacts could change if reported ground elevations of boreholes are revised.
2. CPT borehole surface elevations are approximate, placed on projected ground surface between continuous boreholes WSEWWC-002B and 003B.
3. All depths (vertical axis) shown as elevation values (NAVD88), as shown on Engeo, Inc. borehole logs.
4. Bottom of hole (B.O.H.) values shown as total depth below ground surface.
5. Borehole names and and horizontal distance shown above (from Engeo logs and location maps). Geologic relations could change if borehole locations are revised.
6. Drilling method indicated as last letter in borehole name:
   B = Mud Rotary borehole with SPT
   C = Cone Penetrometer Test
7. See Figure 2 for location of cross section.



Scale
Vertical exaggeration 50X



Explanation

Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain; solid contacts have a resolution no better than about 30'.

WSEWWC_003B — Geotechnical borehole, approximately located.

W — Water visible on 1937 aerial photography.

Geologic Units

HISTORICAL

- AF — Artificial fill; visible on 1937 aerial photography.
- Rch — Channel deposits; sorted sands and silty sand; fining upward.

HOLOCENE

- Hcs — Crevasse splay deposits; fine to coarse sand, silt and clay.
- Hdc — Distributary channels, sand, silt, and clay.
- Hdf — Distributary fan deposits; sand, silt and clay.
- Ha — Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
- Hch — Channel deposits; sorted sands and silts; fining upward.
- Hb — Channel bar deposits; fine sand, and silt deposited in or along channel lateral margins.
- Hfy — Alluvial fan deposits, well graded gravel, sand, silt and clay; volcanic lithologies.
- Qa — Alluvial deposits, undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
- Qn — Basin deposits; fine sand, silt and clay, dark yellow to dark yellowish brown, under cultivation in 1937.
- Qs — Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.

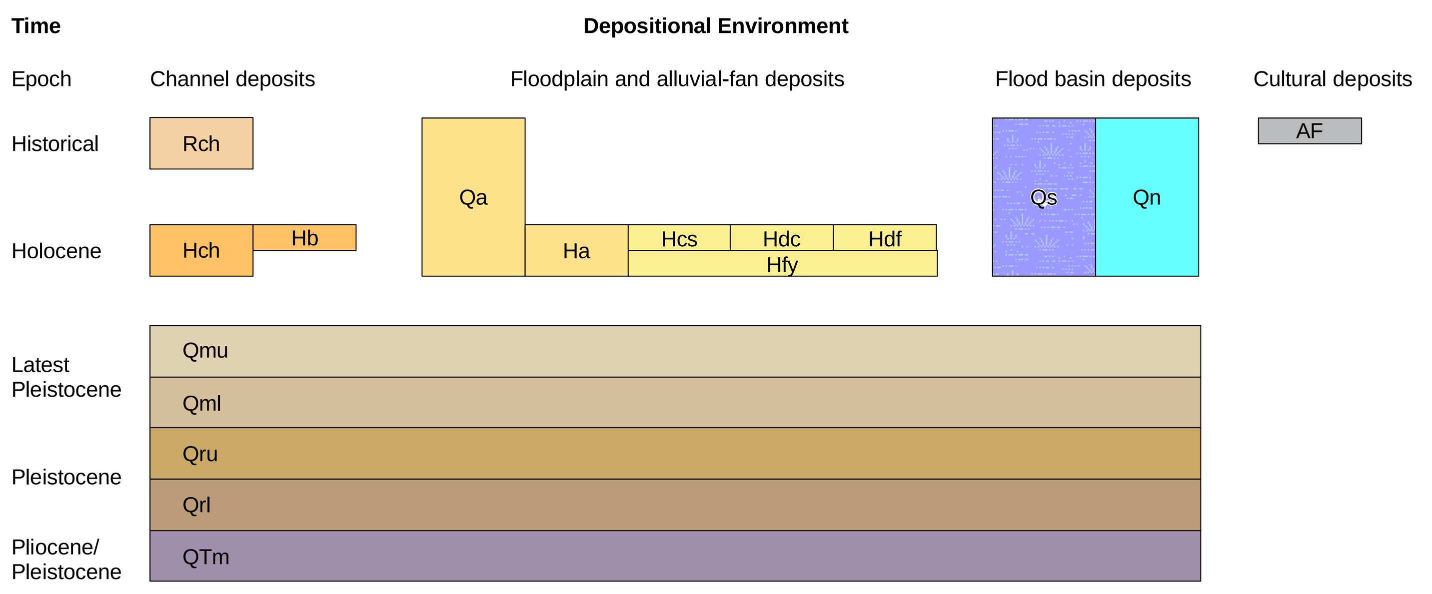
PLEISTOCENE

- Qmu — Modesto Formation; upper member; unconsolidated to semi-consolidated gravel, sand, silt, and clay.
- Qml — Modesto Formation; lower member; unconsolidated gravel, sand, silt and clay.
- Qru — Riverbank Formation; upper member, semi-consolidated to consolidated gravel, sand, silt and minor clay.
- Qrl — Riverbank Formation; lower member; consolidated gravel, sand, silt, and clay, generally associated with strong duripan horizon.

PLIOCENE / PLEISTOCENE

- QTm — Tuff Breccia; Volcanic tuff breccia (andesitic and rhyolitic) from Sutter Buttes, latest Pliocene.

Stratigraphic Correlation Chart



1:20,000

N

0 — 0.5 — 1 Miles

0 — 0.5 — 1 Kilometers

Map projection: UTM NAD83 Zone 10N
Topo base USGS quadrangles.

Plate 1 - Surficial Geologic Map of the Wadsworth Canal Area

WLA — WILLIAM LETTIS & ASSOCIATES, INC.
Walnut Creek, CA

Sutter_MGT_Plate1_Wadsworth.mxd — 09/04/2009

WLA
WILLIAM LETTIS & ASSOCIATES, INC.
1777 Botelho Drive, Suite 262, Walnut Creek, California 94596
tel (925) 256-6070 fax (925) 256-6076

September 8, 2009

Mr. Juan Vargas
URS Corporation
2870 Gateway Oaks Drive, Suite 150
Sacramento, CA 95833

RE: Surficial geologic mapping and geomorphic assessment, California Department of Water Resources Urban Levees Project, Sutter Bypass, Sutter County, California

Dear Mr. Vargas:

This memorandum presents the surficial geologic mapping and preliminary geomorphic assessment of the eastern Sutter Bypass area, for the California Department of Water Resources (DWR) Urban Levees Project geotechnical characterization. The goal of this mapping and geomorphic assessment is to provide information on the type and distribution of surface and shallow subsurface deposits that likely underlie the project levees along the eastern part of the bypass. The purpose of this study is to develop spatially-continuous geologic data and a conceptual model that allows reasonable stratigraphic interpretations between widely-spaced subsurface explorations, with respect to potential levee underseepage (i.e., Llopis et al., 2007). This letter presents the technical approach, surficial geologic map, conceptual geomorphic model, and initial results based on map analysis and preliminary review of available Phase 1 geotechnical data.

We appreciate the opportunity to provide these geomorphic and geologic data and preliminary interpretations of the shallow stratigraphic conditions in the Sutter Bypass study area. Please do not hesitate to call either of the undersigned if there are any questions or comments.

Respectfully,

WILLIAM LETTIS & ASSOCIATES, INC.

Justin Pearce, C.E.G. 2421
Senior Geologist
(925) 256-6070

Keith Kelson, C.E.G. 1610
Principal Geologist
(925) 256-6070

1.0 Approach

The approach to developing a surficial geologic map of the Sutter Bypass area (Figure 1, Plate 1) consisted of analysis of the following data: Aerial photography (black and white stereo-pairs taken in 1937, ~1:20,000-scale); early USGS topographic maps (i.e., 1911); published surficial geologic maps (Helley and Harwood, 1985); early and modern soil survey maps (Strahorn et al., 1911; Lytle, et al., 1988); field reconnaissance visit on June 22, 2007; and other maps and documents. Synthesis of these data allow for the development of a detailed surficial geologic map that provides an initial understanding of primary geomorphic processes that have acted in the study area during recent and historical geologic time. Through this mapping, we identify primary geomorphic features and associated surficial geologic deposits, such as abandoned paleochannels, marsh and basin deposits, flood-basin deposits, and other features commonly associated with flood-basins adjacent to large, active river systems. Knowledge of fluvial processes and the ability to recognize depositional environments in the geologic record are key to identifying locations along levees where underseepage is most likely to occur (Llopis, 2007).

The surficial geologic map was developed at the nominal scale of the aerial photography (1:20,000). This scale establishes the resolution of the map (Plate 1). The map unit contacts shown on the surficial geologic map should be considered approximate, and accurate to no more than about 30 feet on either side of the line shown on the map. The 1937 aerial photographs are the primary data set for interpreting the surficial geologic deposits because: (1) they are the oldest high-quality images that pre-date much of the urbanization and landscape alteration within present-day Sutter County (Figure 2); and, (2) these data represent a close approximation to the surficial deposits that were likely present at the ground surface prior to the construction of the levees. The 1937 photographs generally were taken in late summer or early autumn (i.e., August). By 1937, the area had experienced moderate cultivation that locally obscures geomorphic conditions. However, integration of data from the 1937 photography, old and recent topographic maps, geologic maps, soil surveys and historical documents provides sufficient information to delineate many of the pre-historic and historic surficial deposits in detail. Taken together, these data provide key insights to the geomorphic processes and resulting deposits that may affect levee underseepage.

Additional flood-basin or floodplain deposition may have occurred after 1937, due to flood overflows, levee overtopping, or localized levee failure. A time series analysis that interprets successive aerial photographs taken after major flood events (i.e., 1955) or known local levee failures (i.e., 1986) may reveal additional information on surficial deposits in the Sutter Bypass area. Such analyses are beyond the scope of this study. The data and interpretations presented herein address the primary goal of characterizing the type and distribution of deposits likely present directly beneath the project levees.

1.1 Report Preparation Quality Control

The surficial geologic map data and geomorphic interpretations presented in this memorandum were subject to quality control and quality assurance procedures as required by the Levee

Geotechnical Evaluation Project Management Plan (PMP). The surficial geologic map data developed by this study were reviewed for accuracy and completeness through an internal review and an independent technical review by Dr. Janet Sowers of WLA. Results of QA/QC review were documented using PMP Exhibit 2.2-3 (Independent Technical Review Report) and are kept on file according filing control plan. Subsurface data shown on diagrams were provided as draft information, and were not verified for accuracy or completeness by this study.

2.0 Geologic Setting

The Sutter Bypass (Bypass) study area lies southeast of the volcanic Sutter Buttes, between the Sacramento and Feather Rivers. The project levee addressed in this study borders the eastern side of the Sutter Bypass, extending from the Wadsworth Canal southeast to the Feather River (Figure 1). The Bypass levee generally trends northwest-southeast, and ties in to the Feather River west bank levee.

The Bypass levee lies northeast of Sutter Basin, a low-lying area east of the Sacramento River and west of the Feather River, where overflow and floodwaters from Butte Basin (located northwest of the Sutter Buttes), the Sacramento River, and the Feather River produced a seasonally marshy area. Except for the Sutter Buttes area, the regional land surface is nearly flat, and along the Bypass area gently slopes southwest at an elevation of about 30 to 40 feet. Construction of the Sutter Bypass was completed in 1924 to serve as an overflow for Sacramento River floods in the winter, and a source of irrigation in the summer (DWR, 1976). The eastern levee was enlarged in 1942 (Corps of Engineers, 1953). Prior to cultural modification, surface water runoff in the Bypass area was delivered to the Sutter Basin via intermittent, meandering creeks and sloughs from the northern Central Valley, including: Snake River, Snake Slough, Gilsizer Slough, Nelson Slough, and flood overflow channels emanating from the western side of the Feather River. The construction of the Bypass levee blocks water from the east that normally drains to the Sutter Basin and Sacramento River (DWR, 1976). Presently, many of the natural drainages and channels have been replaced by linear ditches, agricultural drains, and canals (Figure 2).

3.0 Surficial Geologic Mapping

Published surficial geologic maps of the Sutter Bypass study area generalized the surficial deposits primarily as late Quaternary basin (map unit Qb) deposits, with localized units of late Quaternary alluvium, Quaternary Modesto Formation (lower member), and Quaternary Riverbank Formation (lower member) (Helley and Harwood, 1985). These map units were delineated at a regional scale (i.e., 1:62,500). The current analysis of the Bypass uses this geologic framework as a basis for more detailed mapping of late Holocene alluvium and geomorphic features (Plate 1). The surficial geologic map units within the Sutter Bypass study area are described below, in order from oldest to youngest.

The oldest map unit exposed in the study area is the late Quaternary Riverbank Formation (lower member), and is mapped in the south portion of the study area east of Nelson Slough, where it likely directly underlies the project levee near the latitude of Laurel Avenue (Plate 1). This formation (map unit Qrl) is present in the shallow subsurface beneath much of the bypass area, and consists of alluvial-fan deposits derived from the Sierra Nevada during the middle Pleistocene (about 400,000 to 200,000 years ago). The Riverbank Formation is semi-consolidated, and is associated with the presence of a well-developed hardpan (or, duripan) layer that is a product of soil-forming processes over substantial geologic time. This hardpan layer reflects an ancient land surface that locally is buried by younger deposits. Soils developed on the Riverbank Formation in the Bypass area include the San Joaquin loam of Strahorn et al. (1911) and the Yuvas loam (Lytle et al., 1988), both of which document a strongly cemented hardpan at depths of about 1.5 to 3 feet below ground surface.

The late Pleistocene Modesto Formation is younger than the Riverbank Formation and is present in the map area primarily along the margin of Gilsizer Slough and directly east of Highway 113 (Plate 1). This unit is divided into two members, a lower (older) unit that is about about 42,000 to 29,000 years old (Qml), and an upper member that is about 24,000 to 12,000 years old (Qmu) (Helley and Harwood, 1985). The Modesto Formation, in general, consists of unconsolidated sand, silt, and clay, and is associated with a moderate amount of secondary (pedogenic) clay accumulation. This clay-rich horizon may form laterally continuous zones of low hydraulic conductivity. These soil horizons may extend across boundaries between coarse and fine-grained strata within the latest Pleistocene alluvium, and may form relatively continuous zones of low vertical hydraulic conductivity within the Modesto Formation. Soils developed on the Modesto Formation include the Gridley loam of Strahorn et al. (1911) and the Marcum clay loam with “siltstone” hardpan (Lytle, 1988).

Younger surficial deposits overlying the Riverbank and Modesto Formation include late Quaternary marsh, basin, and alluvial deposits (map units Qs, Qn, and Qa, respectively), which are considered Holocene age (i.e., less than 11,000 years old). The widespread basin deposits are about 4 to 8 feet thick and bury the gently southwest dipping Modesto Formation (Figure 3). The thickness of the basin deposits increases to the southwest, in the direction of Sutter Basin (Figure 3). The soils developed on the basin deposits generally are associated with the Stockton clay adobe and Marcuse clay of Strahorn et al. (1911) and the Oswald clay (Lytle et al., 1988), and thus represent immature soils with overall fine-grained textures. Undifferentiated alluvial deposits (map Qa, Plate 1) are present along Gilsizer Slough, and are inset (i.e., topographically lower) into the adjacent Modesto Formation. The Quaternary marsh deposits (map unit Qs, Plate 1) are present between the Sutter Bypass levees northwest of Gilsizer Slough, and are also fine-grained deposits that are differentiated from basin deposits by usually being underwater or having standing water at the time when the 1937 photographs were taken (usually late summer to early autumn).

Inset into the units described above are deposits of Holocene alluvial channels (map unit Hch, Plate 1), which are a network of moderately sinuous channels with southwesterly orientations. These channels appear to be mostly filled with sediment by the time of 1937 photographs, and are expressed only locally as subtle topographic lows in the ground surface. Many of these channels extend west of, and therefore cross beneath, the eastern Sutter Bypass levee (Plate 1). The alluvial channels west of Gilsizer Slough start on the alluvial plain as intermittent creeks, and are not directly connected to the Feather River (USGS Tisdale Weir quadrangle, 1911).

The channel deposits are tentatively associated with the Liveoak series, sandy clay loam soil (Lytle et al., 1988), and consist of a lower, sandy unit that fines-upward into an upper, silt and clay layer.

Subdivisions of the Holocene channels include sloughs (map unit Hsl, Plate 1), distributary channels (map unit Hdc), and overflow channels (map unit Hofc). These deposits, in general, also consist of a fining-upward sequence of sand, silt, and clay. The sloughs are present primarily east of Highway 113 (Plate 1) and have southwesterly orientations. The sloughs are ephemeral channels that drain the alluvial plain between Gilsizer Slough and the Feather River. The term "slough" in this study does not mean tidally-influenced channels, but instead channels that likely conveyed relatively slow-moving water from direct precipitation and sheet-flow runoff. The overflow channels convey flood flows that overtop the banks of the Feather River onto the floodplain, and are interpreted as higher-energy channel systems relative to the sloughs. The distributary channels route flow from and sediment onto the floodplain, and end at distributary-fan deposits. The overflow and distributary channel deposits are present in the southeastern portion of the Bypass area, south of the latitude of Laurel Avenue (Plate 1).

Localized deposits related to the Holocene alluvial channels are bars (map unit Hb) that typically occur in the medial and lateral portions of the channels, and distributary fan deposits (map unit Hdf) that occur where the channel becomes unconfined and has deposited sediment on the basin floor. Channel bars are relatively uncommon in the Sutter Bypass study area. Distributary fans are common in the southeast portion of the Bypass area, south of the latitude of Sacramento Avenue (Plate 1). The distributary-fan deposits likely consist of unconsolidated fine sand and silt (i.e., Strahorn et al., 1911).

Historical geologic deposits are present along the length of the Bypass study area (i.e., map unit Rch, map unit Rdf). The term "historical" is applied to deposits that are estimated to be less than 150 years old. These deposits share the same genetic origin as the Holocene deposits described above. The historical channel deposits are differentiated from the Holocene channel deposits on the basis of cross-cutting relationships with other map units, relative degree of geomorphic expression and/or dissection, and correlation with land surface expression on the early and modern topographic maps. The Bypass eastern levee overlies the former locations of Holocene and historical alluvial channels in several locations throughout its length (Plate 1).

Undifferentiated Holocene and historical alluvium (map units Ha and Ra) is mapped in the southeastern Bypass area, near the confluence of the Sutter Bypass and the Feather River, generally east of Sawtelle Road (Plate 1). The undifferentiated map unit is delineated where the morphology of these deposits is indistinguishable on 1937 photographs as a result of cultural modifications (i.e., agriculture). The soils developed on the undifferentiated historical alluvium generally correspond with the Sacramento series fine sandy loam and silt loam of Strahorn et al. (1911) and the Shanghai silt loam (Lytle et al., 1988). There is no hardpan layer associated with these soils, supporting the interpretation of geologically young deposits.

4.0 Conceptual Geomorphic Model

Based on synthesis of surficial geologic mapping, topographic maps, soil surveys, geologic maps, and review of readily available subsurface geotechnical information, we present a preliminary conceptual geomorphic model describing general relationships among surface and subsurface deposits along the Sutter Bypass study area. This conceptual model provides a consistent basis for understanding the type and distribution of surficial geologic deposits, primary geomorphic processes, and shallow subsurface stratigraphy in the area. Identification of subsurface stratigraphic formations is challenging, primarily because of a lack of distinctive and laterally extensive stratigraphic marker beds within late Quaternary deposits of the northern Central Valley (i.e., Page, 1986), and because there is little apparent difference in lithology between the late Quaternary formations (i.e., Helley and Harwood, 1985). This study relies heavily on the identification and local correlation of hardpan horizons and deposit color and density changes to delineate subsurface formations.

In a general sense, the Sutter Bypass levees traverse across the distal portions of ancient alluvial-fan deposits that were derived from the Sierra Nevada, and prograded westward onto the valley floor (i.e., Riverbank and Modesto Formations). These Pleistocene deposits are exposed at the ground surface northeast of the Bypass study area (Helley and Harwood, 1985; Page, 1986), dip to the southwest and are mantled by younger fine-grained basin deposits (Figure 3). In contrast, the Modesto Formation is exposed at the ground surface along Gilsizer Slough and directly east of Highway 113 (Plate 1). The surficial map pattern of the Modesto deposits in these locations suggests depositional lobes from an ancestral Gilsizer Slough. These deposits may have been related to discharges and sediment loads that were higher than present-day conditions. These deposits may, perhaps, represent an ancestral Feather River channel location that occupied the present-day Gilsizer Slough during the latest Pleistocene and was subsequently abandoned.

The surficial geologic mapping (Plate 1) shows differences in deposit type and distribution from northwest to southeast along the Bypass, which are associated with changes in watershed production of water and sediment, related geomorphic processes, soil profile development, and the underlying subsurface hardpan layer. These differences illustrate the diversity of past geomorphic processes along and near the Bypass and, as a consequence, the type of geologic deposits at and near the ground surface. The surficial geologic map allows the interpretation of “reaches” along the Bypass within which geomorphic processes and their associated deposits are likely to be relatively consistent. The Bypass study area consists of four general reaches, from northwest to southeast, each having characteristic deposit types and distributions (Plate 1).

The westernmost reach of the Bypass study area extends from the junction with the Wadsworth Canal to directly south of the Tisdale Weir (“Reach I”, Plate 1). The levee along this reach, about 8.1 miles long, primarily overlies fine grained basin deposits accumulated on the valley floor over geologic time. This deposition resulted from flooding of the Sacramento and Feather Rivers, tributaries draining Sutter Buttes, and sheet flow from the generally flat valley floor. Holocene and historical channel deposits (map units Hch and Rch, Plate 1) are inset into the basin deposits. These southwest-trending alluvial channel deposits locally underlie the Bypass levee, and result in local differences in material textures beneath the levee (Figure 4). About 27 abandoned channels traverse the levee along this reach (approximately 3 channels per levee mile). The channels are about 250 feet wide, but range from about 100 to 300 feet wide (Plate 1). In this area, the channels are about 6 to 8 feet deep, and are typically filled with sand, silt, and clay in a fining-upward sequence, i.e., coarser-grained sand overlain by about one to two feet of silt and clay. This sedimentary sequence may be conducive to seepage where relatively more-permeable channel sands are overlain by a relatively thin, fine-grained “blanket” layer.

The second reach along the Bypass, about 1.1 miles long, extends across Gilsizer Slough (“Reach II”, Plate 1), where Modesto Formation deposits are present at the ground surface. Undifferentiated alluvium (map unit Qa, Plate 1) is present along the historically-active Gilsizer channel floor, and is inset to the Modesto Formation (Figure 5). The Gilsizer Slough alluvium extends beneath the eastern and western Bypass levee, and thus represents the progradation of younger deposits with respect to the Modesto Formation. Along this reach, the Bypass levee is underlain by younger Gilsizer Slough alluvium flanked by the relatively denser, semi-consolidated late Pleistocene Modesto deposits (Figure 5). Areas where the levee directly overlies the Modesto Formation may be relatively less conducive to underseepage, as the associated hardpan layer may form locally continuous zones of low hydraulic conductivity.

The third reach along the Bypass extends from the Gilsizer Slough to the latitude directly south of Laurel Avenue, and is about 6.6 miles long (“Reach III”, Plate 1). This reach is generally similar to Reach I, except Reach III has Pleistocene deposits (i.e., lower Modesto and Riverbank Formations) exposed at or very near the ground surface, and has a sparser channel density (about 2 channels per levee mile) compared to Reach I. About 14 southerly-oriented sloughs are mapped across this reach and locally underlie the Bypass levee (Plate 1). The sloughs originate from the Feather River, near Star Bend and Shanghai Bend, extending southward toward the Bypass. The sloughs along Reach III are about 250 feet wide, but range from about 100 to 300 feet wide, similar to Reach I (Plate 1). In this area, the channels are also probably about 6 to 8 feet deep, and probably filled with sand fining-upward to silt and clay. These channel deposits may be conducive to underseepage because of the deposit stratigraphy that has coarser-grained sand overlain by about one to two feet of silt and clay. Late Quaternary Riverbank Formation is at the ground surface along the southwestern end of Reach III (Plate 1), and likely is not conducive to seepage due to the dense and strongly-developed hardpan clay layer that is usually at about 1.5 to 4 feet depth below ground surface.

The fourth reach along the Bypass extends from directly south of the latitude of Laurel Avenue to the confluence with the Feather River west bank levee (“Reach IV”, Plate 1). Reach IV, about 1.9 miles long, has Holocene and historical alluvium at the ground surface along this reach of the Bypass, primarily because of the proximity to the Feather and Bear Rivers (Plate 1). About 8 distributary channels, usually 90 feet wide but ranging from 45 to 190 feet wide, cross the floodplain in southwesterly orientations, leading to geologically young distributary-fan sediments. These sediments, primarily consisting of fine to coarse sand and silt, probably were deposited as a result of increased sediment and water input contributed to the Feather River from the Bear River; the confluence located directly upstream from this reach of the Bypass (Figure 1). Historically, the Feather River and the Bear River have aggraded from substantial mining debris input, thus reducing channel cross sectional area (i.e., James, 1999).

This reduction of cross section area, coupled with the trajectory of floodflow from the Bear River watershed, resulted in water overtopping the Feather River channel banks, and depositing sediment onto the western floodplain where the Bypass levee is located (Plate 1).

5.0 Applications to the Urban Levee Project

Based on synthesis of the surficial geologic map with preliminary Phase 1 borehole and cone penetrometer (CPT) data, the Bypass levee generally is underlain by relatively young fine-grained clay and sandy clay deposits that are laterally interrupted by local coarser channel fill deposits (i.e., Figures 3, 4, 5, and 6).

The northernmost reach of the Bypass levee ("Reach I") is predominantly underlain in the shallow subsurface by relatively young fine-grained clay and sandy clay deposits. These basin deposits are laterally interrupted by coarser-grained deposits filling abandoned channels that are about 250 wide (Plate 1, Figure 4). Mud rotary borehole WSESBP_011B, which penetrated channel unit Rch norths of Gilsizer Slough (Plate 1), indicates the channel deposit is about four-feet thick, consisting of about 60% fine to coarse sand (medium dense) with clayey sand. The clayey sand grades upward into clay, of about 45% sand fraction. This suggests locally coarse and unconsolidated, and therefore likely permeable, material in the channel fill. Based on review of adjacent borehole data, the basin deposits (Figure 4) generally consist of stiff clay, with less than 10% fine sand. It is likely that most or all of the small channels mapped herein as unit Rch are similar in textural characteristics and depths, because of similar genetic origin and geomorphic process of channel development and infilling. These deposits underlie Reach I in at least 27 places between Wadsworth Canal and Gilsizer Slough (Plate 1).

Reach II crosses late Pleistocene and Holocene geologic deposits associated with Gilsizer Slough (Plate 1). Review of subsurface borehole and CPT data indicate that the basin deposits north of the slough consist of medium stiff to stiff clays (Figure 5). The channel fill deposits within Gilsizer Slough (map unit Qa, Plate 1) consist of alternating beds of sandy gravel and clay. These channel deposits are inset into the lower Modesto Formation which, in this area, consists of very stiff sandy clay interbedded with silty sand and localized dense sand. Directly south of Gilsizer Slough, the lower Modesto Formation is at the ground surface (Plate 1). Subsurface data suggest that a hardpan horizon is encountered at about 3 to 4 feet below the ground surface. The uppermost deposit above the hardpan consists of sand and silty sand, and probably is weathered and/or culturally re-worked materials of the lower Modesto Formation. Thus, north of Gilsizer Slough, potentially low-permeability basin materials blanket the Modesto, and are locally cut by channel deposits, whereas at and south of Gilsizer Slough the local channel deposits are inset directly into the dense Modesto Formation. Where the Bypass levee rests on the unconsolidated Qa deposits within Gilsizer Slough, these coarse deposits may be associated with higher probabilities of levee underseepage. In constrast, the sections of the levee underlain directly by the Modesto Formation containing consolidated (hardpan) horizons are much less likely to experience underseepage.

Reach III is similar in geomorphic nature to Reach I, except it has a lower frequency of channels as compared to Reach I (Plate 1). It is probable that the composition of these deposits generally will be consistent with those along Reach I (i.e., coarse-grained channel fill with upper fine-grained layers). These channels are more likely to promote seepage beneath the levee compared to the basin deposits. Additionally, the Pleistocene materials that likely directly underlie the project levees along this reach (Plate 1) are relatively dense and the associated hardpan layer may form a relatively continuous zone of lower hydraulic conductivity. Where the levee directly overlies Modesto formation (NW ¼, Section 20; southeast of the Sutter Causeway), there is a lower likelihood of underseepage. There is also a lower likelihood of underseepage where the levee rests on the Riverbank Formation in lower length of Reach III (SW ½, Section 34).

Along Reach IV, geologically young Holocene and historical alluvium is beneath the Bypass levee (Plate 1). This uppermost layer, about five-feet thick, is locally cross-cut by channel deposits that also consist of silt and sand (Figure 6). Quaternary basin deposits do not directly underlie the Bypass levee along this reach. Review of Phase 1 subsurface geotechnical data indicates that these alluvial deposits consist of silty sand and sandy silt textures. Based on review of Phase 1 data in other Project areas (i.e., Marysville), the uppermost alluvium generally has low densities (i.e. loose to medium dense), and consequently relatively high permeability. The surficial mapping indicates that essentially all of this reach of the levee (about1.9 miles) is underlain by loose, unconsolidated sandy alluvium, which may be susceptible to substantial underseepage. The local recent channels (map units Ra and Rdc; Plate 1) may contain coarser deposits and may be more susceptible to underseepage.

Synthesis of the surficial mapping and geotechnical data indicate that subsurface stratigraphy along the Sutter Bypass area locally may be conducive to levee underseepage. Shallow strata typically include denser and probably semi-consolidated material (i.e., Modesto Formation) that likely contains a moderately developed low-permeability hardpan horizon. The hardpan may or may not be laterally continuous, depending on post-depositional soil formation and erosional processes. Along Reach I and III, the Modesto formation is overlain by about 4 to 6 feet of medium stiff to stiff clay (i.e., basin deposits). The basin materials locally are cross-cut by relatively loose, sandy channel deposits that have a thin fine-grained upper “blanket” layer. Therefore, this shallow subsurface stratigraphy may promote levee underseepage along certain areas of the Bypass project levees that overlie geologically young, loose, sandy channel material lies between the dense Pleistocene deposits and relatively thin, low-permeability clay-rich “blanket” layer. Along Reach IV, a layer Holocene and historical alluvium from the Feather River mantles the Modesto Formation, and also may promote levee underseepage.

Lateral and vertical variability in the shallow subsurface deposits has resulted from past geomorphic processes. The conceptual subsurface stratigraphic framework suggests that stratigraphic relationships may promote localized levee underseepage, given certain hydraulic conditions. Further spatial analyses of the surficial geologic mapping and subsurface geotechnical exploration data are needed to better constrain and characterize areas that are most susceptible to underseepage in the study area.

6.0 Limitations

This geomorphic assessment and associated data interpretation have been performed in accordance with the standard of care commonly used as the state-of-practice in the geologic engineering profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this geographic area performing the same services under similar circumstances during the same time period.

Discussions of surface and subsurface conditions summarized in this technical memorandum are based on geologic interpretations of subsurface soil data at limited exploration locations available to this assessment through August of 2007. Variations in subsurface conditions may exist between exploration locations, and the project team may not be able to identify all adverse conditions in the levee and its foundation. This memorandum is for the use and benefit of DWR. Use by any other party is at their own discretion and risk.

7.0 References

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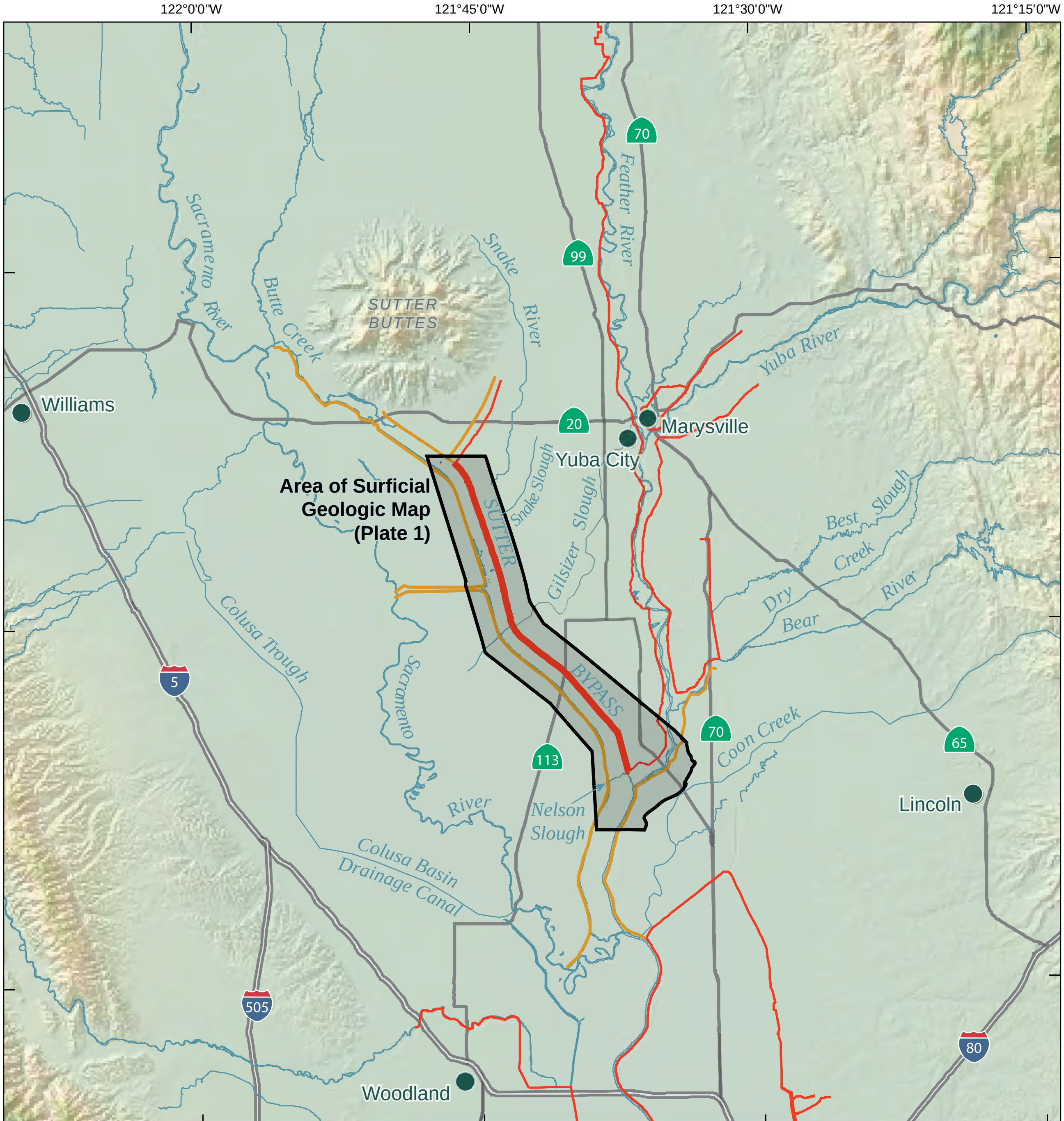
USGS, Gilsizer Slough topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.

USGS, Marcuse topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.

USGS, Nicolaus topographic quadrangle, surveyed 1908, published 1910; map scale 1:31,680, five foot contour interval.

USGS, Tisdale Weir topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.



Sources: NAIP, 2006

Projection: UTM Zone 10, NAD83

Explanation

- Sutter Bypass project levee
- Urban project levee
- Other levee

N

0 5 10 mi

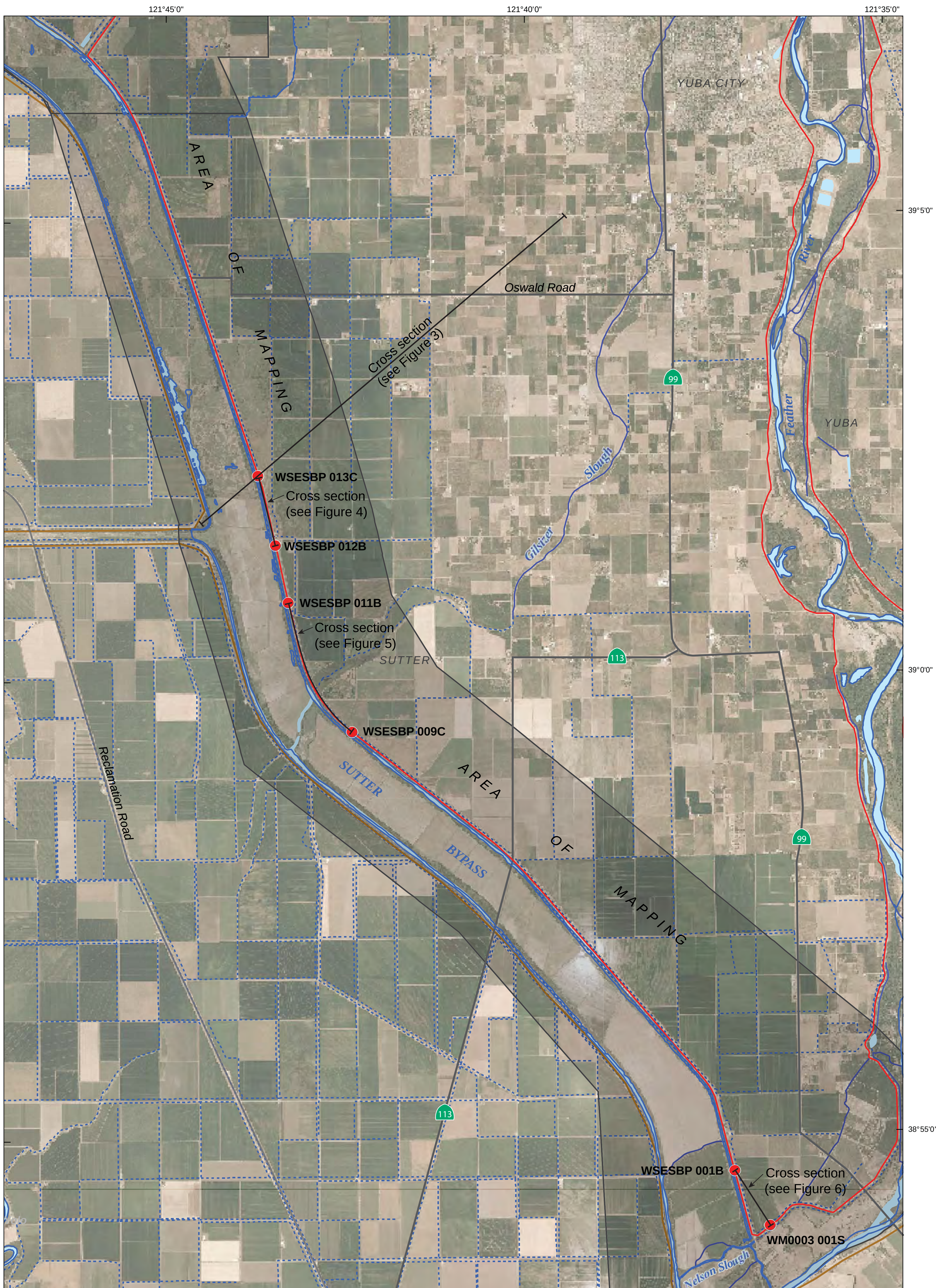
0 5 10 km

WLA

Map of Northern Central Valley, California

DWR URBAN LEVEE PROJECT

Figure 1



Sources: NAIP, 2006

Projection: UTM Zone 10, NAD83

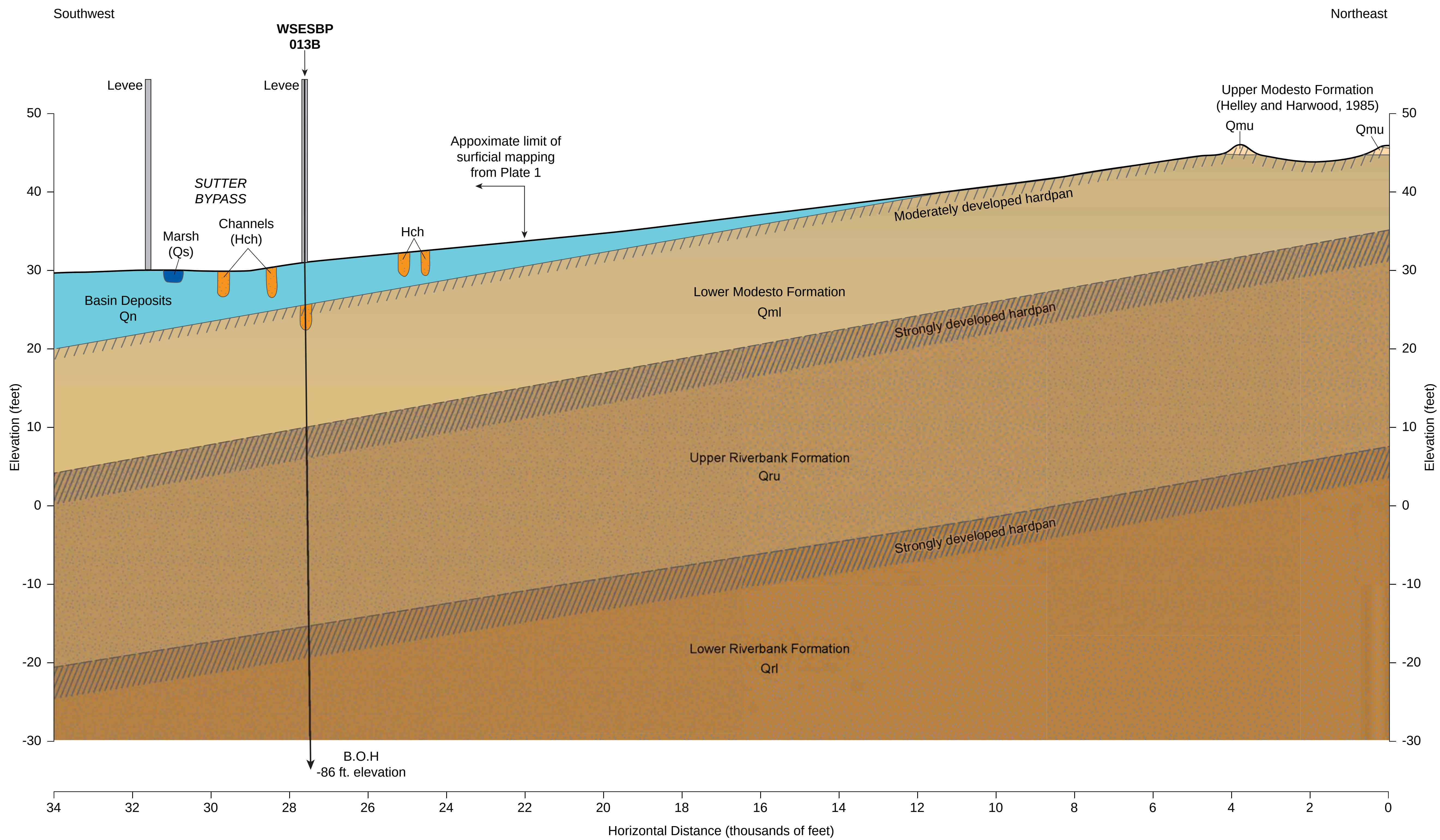
Explanation

Creek, slough	
Canal, drain	
Urban Project levee	
Other levee	
WSESBP 001B ●	Location of subsurface geotechnical exploration

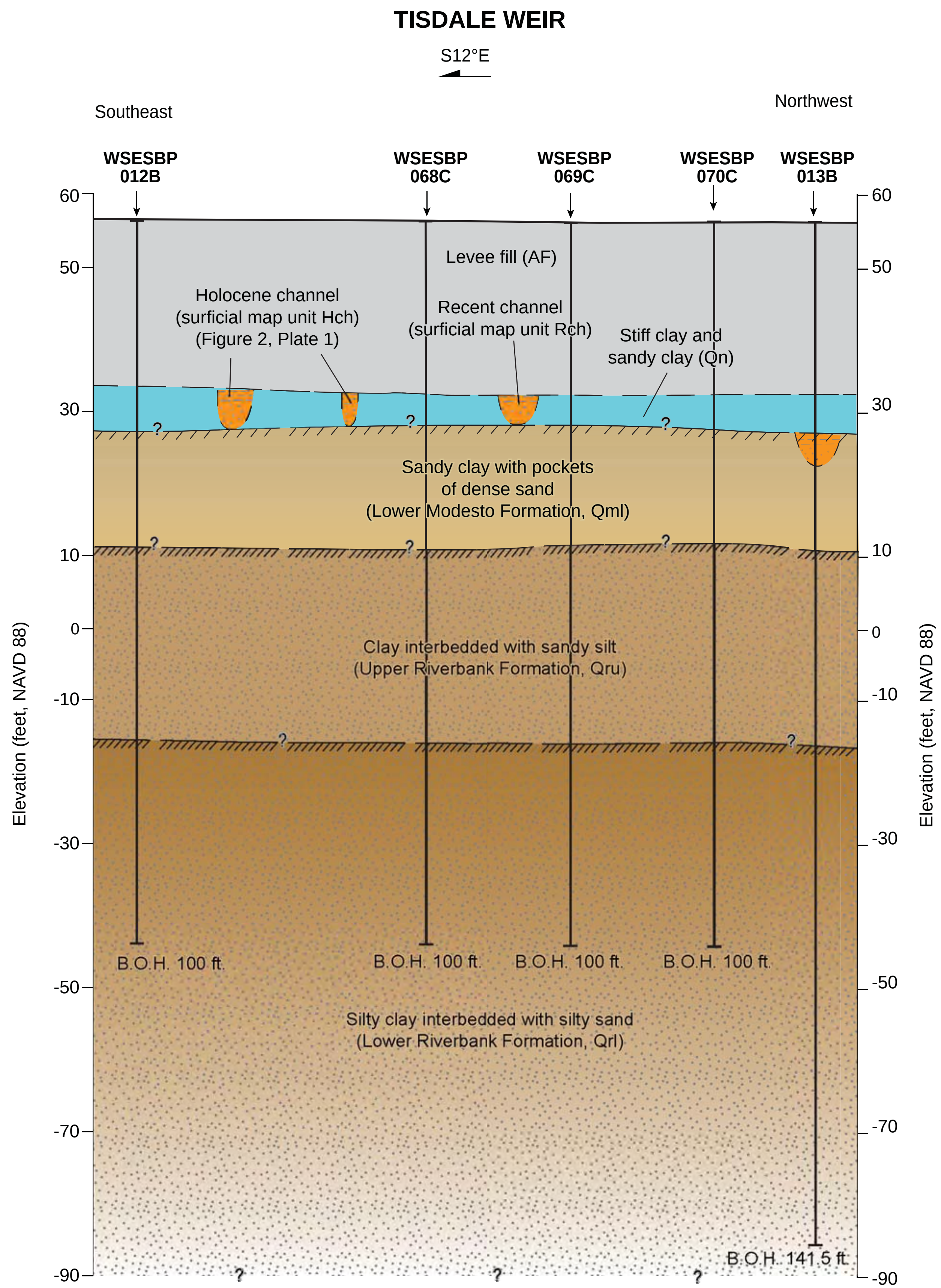
N

0 1 2 mi

0 1 2 km



Notes: 1. See Figure 2 for location of cross section.
2. Surficial geologic units and contacts from this study and Helley and Harwood (1985).
3. Ground surface elevation from USGS Gilsizer Slough topographic map, 5-foot contour interval.



Explanation

Channel identified on surficial geologic map or as fining upward sequence of sediments in boreholes

Moderate paleosol (hardpan)

Strong paleosol (hardpan)

Notes:
1. Borehole ground elevation values from URS Corp., and reported in the Boring Location Survey, DWR task #10. (NAVD 88).
2. CPT borehole surface elevations are approximate, placed on projected ground surface between boreholes WSESBP_012B and WSESBP_013B.
3. Bottom of hole (B.O.H.) values shown as total depth below ground surface.
4. Borehole names and horizontal distance shown above from draft URS logs and location maps. Geologic relations could change if borehole locations are revised.
5. Drilling method indicated as last letter in borehole names.
   B = Mud Rotary unit with SPT
   C = Cone Penetrometer Test.

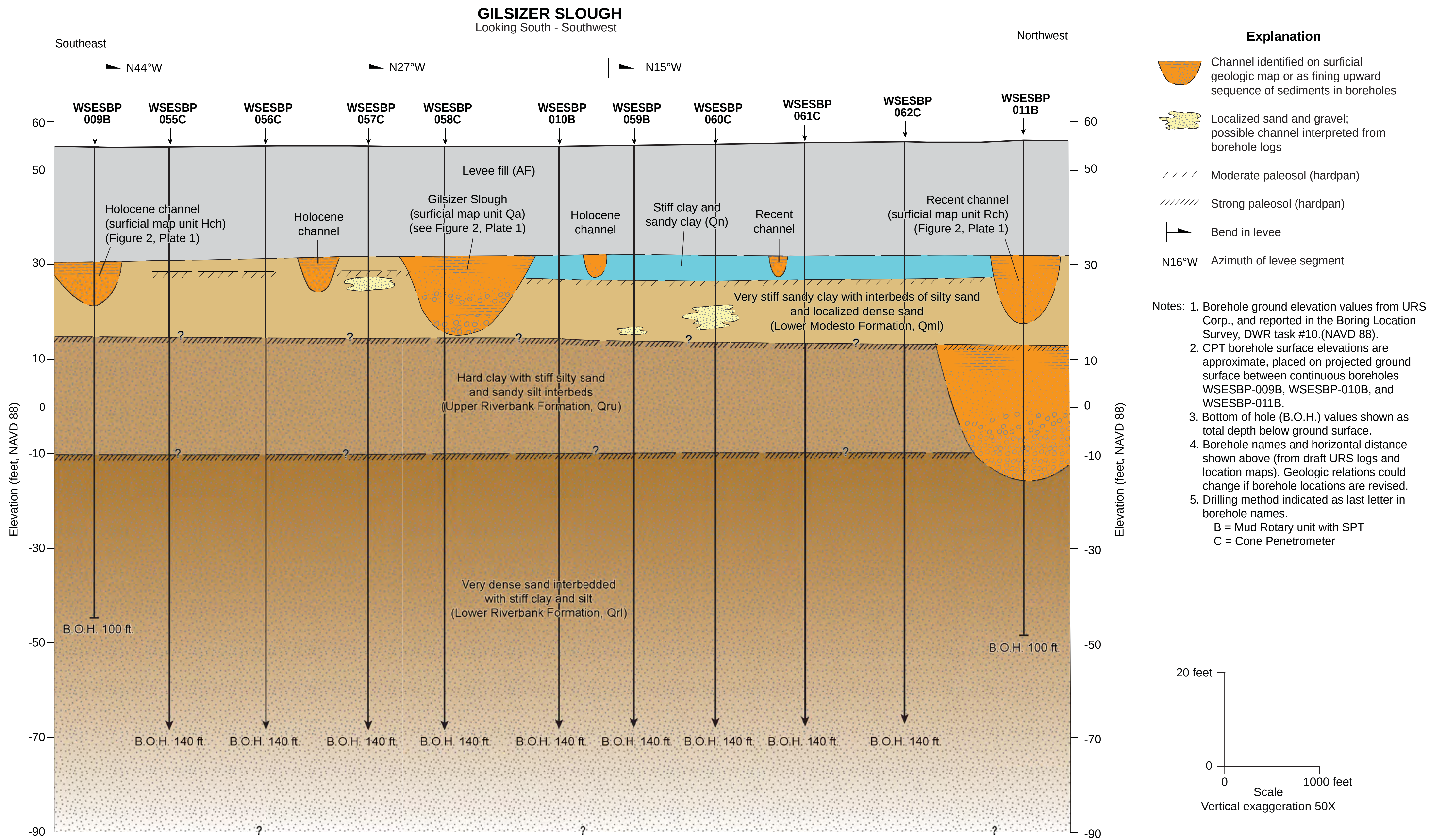
20 feet
0
0 1000 feet
Scale
Vertical exaggeration 50X

WLA

Conceptual Subsurface Diagram across Eastern Levee Crest of Sutter Bypass at Tisdale Weir

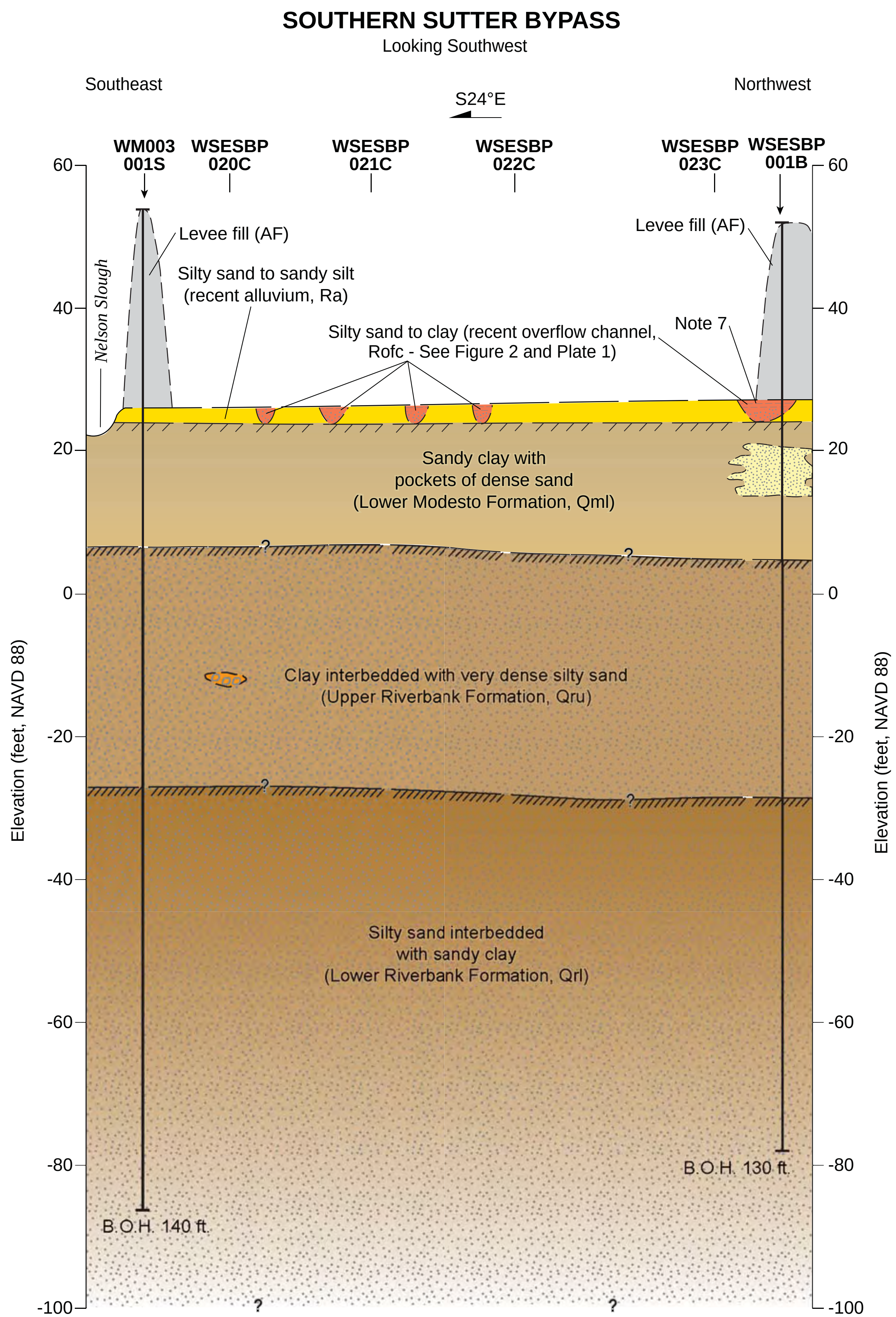
DWR URBAN LEVEE PROJECT
Figure 4

Modified 09.03.09

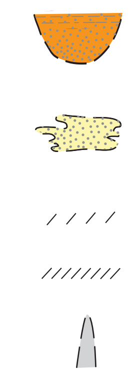


Notes:
1. Borehole ground elevation values from URS Corp., and reported in the Boring Location Survey, DWR task #10.(NAVD 88).
2. CPT borehole surface elevations are approximate, placed on projected ground surface between continuous boreholes WSESBP-009B, WSESBP-010B, and WSESBP-011B.
3. Bottom of hole (B.O.H.) values shown as total depth below ground surface.
4. Borehole names and horizontal distance shown above (from draft URS logs and location maps). Geologic relations could change if borehole locations are revised.
5. Drilling method indicated as last letter in borehole names.
   B = Mud Rotary unit with SPT
   C = Cone Penetrometer

Conceptual Subsurface Diagram across Eastern Levee Crest at Gilsizer Slough



Explanation



Channel identified on surficial geologic map or as fining upward sequence of sediments in boreholes

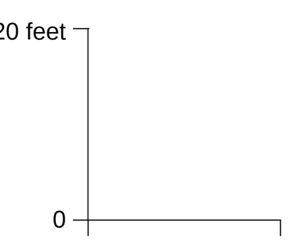
Localized sand and gravel; possible channel interpreted from borehole logs

Moderate paleosol (hardpan)

Strong paleosol (hardpan)

Levee widths and side slopes are schematically shown

Notes:
1. Borehole ground elevation values from URS Corp., and reported in the Boring Location Survey, DWR task #10. (NAVD 88).
2. CPT borehole surface elevations are approximate, placed on projected ground surface between boreholes WM00_001S and WSESBP_001B.
3. Bottom of hole (B.O.H.) values shown as total depth below ground surface.
4. Borehole names and horizontal distance shown above from draft URS logs and location maps. Geologic relations could change if borehole locations are revised.
5. Drilling method indicated as last letter in borehole names.
   - B = Mud Rotary unit with SPT
   - S = Sonic vibracore
   - C = Cone Penetrometer Test.
6. Cone penetrometer borehole locations projected to the trend of this cross section.
7. Recent over flow channel shown beneath the northwestern levee intersects the levee at a sub-orthogonal angle. This conceptual cross section intersects the levee and the over flow channel at an oblique angle, as shown in the channel asymmetry.



Scale

Vertical exaggeration 50X

WLA

Conceptual Subsurface Diagram across the Eastern Levee of the Sutter Bypass Confluence with the Feather River

DWR URBAN LEVEE PROJECT

Figure 6


Explanation
Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain; solid contacts have a resolution no better than about 30'.
Geomorphic Reaches Discussed in Text
Project Levee
WSESBP_001B
Geotechnical borehole, approximately located.
W
Water visible on 1937 aerial photography.
Geologic Units
HISTORICAL
AF Artificial fill; visible on 1937 aerial photography.
Rob Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
Rcs Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
Rdf Distributary fan deposits; sand, silt and clay.
Rch Channel deposits; well sorted sands and fine gravels.
Rb Channel bar deposits; fine gravel, sand, and silt deposited in or along channel lateral margins.
Rdc Distributary channel deposits, trace gravel, sand, silt, and clay; channelized flow conducting sediment to floodplain.
Rofc Overflow channels; vertically stratified sand, silt, and clay in floodplain channels occupied primarily when high-stage water overtops channel banks.
Rsl Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Ra Alluvial deposits, undifferentiated; sand, silt, and minor lenses of fine gravel.
HOLOCENE
Hdf Distributary fan deposits; sand, silt and clay.
Hch Channel deposits; sorted sands and silts; fining upward.
Hb Channel bar deposits; fine sand, and silt deposited in or along channel lateral margins.
Hdc Distributary channel deposits, sand, silt, and clay.
Hsl Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Ha Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
Qa Alluvial deposits, undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
Qn Basin deposits; fine sand, silt and clay, dark yellow to dark yellowish brown, under cultivation in 1937.
Qs Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.
PLEISTOCENE
Qmu Modesto Formation; upper member; unconsolidated gravel, sand, silt, and clay.
Qml Modesto Formation; lower member; unconsolidated to semi-consolidated gravel, sand, silt and clay.
Qru Riverbank Formation; upper member, semi-consolidated to consolidated gravel, sand, silt and minor clay.
Qrl Riverbank Formation; lower member; consolidated gravel, sand, silt, and clay, generally associated with strong duripan horizon.
Stratigraphic Correlation Chart
Time
Depositional Environment
Epoch
Channel deposits
Floodplain and alluvial-fan deposits
Flood basin deposits
Cultural deposits
Historical
Holocene
Latest Pleistocene
Pleistocene
Rch Rdc Rsl Rofc
Ra Rcs Rdf Rob
Qa Ha Hdf
Hch Hdc Hb Hsl
Qs Qn
AF
Qmu
Qml
Qru
Qrl
1:20,000
0 0.5 1 Miles
0 0.5 1 Kilometers
N
Map projection: UTM NAD83 Zone 10N
Topo base USGS quadrangles
Plate 1 - Surficial Geologic Map of the Sutter Bypass Area
WLA
WILLIAM LETTIS & ASSOCIATES, INC.
Walnut Creek, CA
Sutter_Bypass_MGT.mxd
09/04/2009
Geologic Mapping by J. Pearce and A. Streig
Digital Cartography by M. Ticci

WILLIAM LETTIS & ASSOCIATES, INC.
1777 Botelho Drive, Suite 262, Walnut Creek, California 94596
tel (925) 256-6070 fax (925) 256-6076

September 8, 2009

Mr. Juan Vargas
URS Corporation
2870 Gateway Oaks Drive, Suite 150
Sacramento, CA 95833

RE: Surficial geologic mapping and geomorphic assessment, California Department of Water Resources Urban Levees Project, Southern Feather River, Sutter County, California

Dear Mr. Vargas:

This memorandum presents the surficial geologic mapping and preliminary geomorphic assessment of the southern Feather River study area, for the California Department of Water Resources (DWR) Urban Levees Project geotechnical characterization. The goal of this mapping and geomorphic assessment is to provide information on the type and distribution of surface and shallow subsurface deposits that likely underlie the project levees along the western bank of the Feather River. The purpose of this study is to develop spatially-continuous geologic data and a conceptual model that provides a framework for stratigraphic interpretations between widely-spaced subsurface explorations. A primary goal is to provide a geologic framework for the geotechnical assessment of potential levee underseepage. This memo presents the technical approach, surficial geologic map, conceptual geomorphic model, and initial results based on map analysis and preliminary review of Phase 1 geotechnical data.

We appreciated the opportunity to provide these geomorphic and geologic data and preliminary interpretations of the shallow stratigraphic conditions in the southern Feather River study area. Please do not hesitate to call either of the undersigned if there are any questions or comments.

Respectfully,

WILLIAM LETTIS & ASSOCIATES, INC.

Justin Pearce, C.E.G. 2421
Senior Geologist

Ashley Streig
Senior Staff Geologist

Keith Kelson, C.E.G. 1610
Principal Geologist

1.0 Approach

The approach to developing a surficial geologic map of the southern Feather River project area (Figure 1, Plate 1) consisted of analysis of the following data: Aerial photography (black and white stereo-pairs taken in 1937, ~1:20,000-scale); early USGS topographic maps (i.e., 1911); published surficial geologic maps (Helley and Harwood, 1985); early and modern soil survey maps (Strahorn et al., 1909; Lytle, et al., 1988); and other maps and documents (Busacca et al., 1989). Synthesis of these data allow for the development of a detailed surficial geologic map that provides an initial understanding of primary geomorphic processes that have acted in the study area during recent and historical geologic time. Through this mapping, primary geomorphic features and associated surficial geologic deposits are identified, such as abandoned paleochannels, channel deposits, floodplain deposits, basin deposits and other features commonly associated with surficial deposits with large active river systems. Knowledge of fluvial processes and the ability to recognize depositional environments in the geologic record are key to identifying locations along levees where underseepage is most likely to occur (Llopis et al., 2007).

The surficial geologic map was developed at the nominal scale of the aerial photography (1:20,000). This scale establishes the resolution of the map (Plate 1), such that analysis of the map data at a more detailed scale than 1:20,000 may introduce uncertainties beyond the original resolution of the data. The map unit boundaries shown on the surficial geologic map should be considered approximate, and accurate within 30 feet on either side of the line shown on the map. The 1937 aerial photographs are the primary data set for interpreting the surficial geologic deposits because: (1) they are the oldest high-quality images that pre-date much of the urbanization and landscape alteration within present-day Sutter County (i.e. Figure 2); and, (2) these data represent a close approximation to the surficial deposits that were likely present at the ground surface prior to the construction of the levees. The 1937 photographs generally were taken in late summer or early autumn (i.e., August). By 1937, the area had experienced moderate cultivation that locally obscures geomorphic conditions. However, integration of data from the 1937 photography, old and recent topographic maps, geologic maps, soil surveys and historical documents provides sufficient information to delineate many of the pre-historical and historical surficial deposits in detail. Taken together, these data provide key insights to the characteristics of shallow deposits beneath the levees, as well as the geomorphic processes responsible for their distribution.

Additional floodplain deposition may have occurred after 1937, due to flood overflows, levee overtopping, or localized levee failure. A time series analysis that interprets successive aerial photographs taken after major flood events (i.e., USDA, black and white stereo-pairs taken in 1958, ~1:20,000-scale) or known local levee failures (i.e., 1986) may reveal additional information on surficial deposits in the southern Feather River area. Such analyses are beyond the scope of this study. The data and interpretations presented herein address the primary goal of characterizing the type and distribution of deposits likely present directly beneath the project levees.

1.1 Report Preparation Quality Control

The surficial geologic map data and geomorphic interpretations presented in this memorandum were subject to quality control and quality assurance procedures as required by the Levee Geotechnical Evaluation Project Management Plan (PMP). The surficial geologic map data developed by this study were reviewed for accuracy and completeness through an internal review and an independent technical review by Dr. Janet Sowers of WLA. Results of QA/QC review were documented using PMP Exhibit 2.2-3 (Independent Technical Review Report) and are kept on file according filing control plan. Subsurface data shown on diagrams were provided as draft information, and were not verified for accuracy or completeness by this study.

2.0 Geologic Setting

The southern Feather River study area lies in the Central Sacramento Valley, between the Coast Ranges to the west and the Sierra Nevada foothills to the east. Feather River drains the western slope of the Sierra Nevada, and emerges from the mountains south of the Thermalito Afterbay (Figure 1). The river flows southward from the Thermalito Afterbay, over middle-to late Pleistocene dissected alluvium derived from the Sierra Nevada. The regional land surface is nearly flat, with a gentle west-southwest slope that flattens out south of the Sutter Buttes, in Sutter Basin. The Feather River is entrenched into middle to late Pleistocene semi-consolidated sediments. Holocene alluvium deposited by the Feather River is present between the present-day levees, inset to the older formations, as well as on the western floodplain as subdued natural levees. The river trends roughly south until its confluence with the Bear River, where it curves

to the southwest (Figure 1). The Feather River lies east of, and is a tributary to the Sacramento River, converging near the town of Nicolaus (Figure 1). A primary influence on the historic processes in the river system was the hydraulic mining that began in the 1850's. Mining occurred through the early 1900's in the Feather, Yuba and Bear River watersheds, and abruptly introduced large quantities of sediment, drastically changing the geomorphic characteristics of these river systems (DWR, 2004; Ellis, 1939). Aggradation within the stream channel was a primary response to the introduction of substantial mining debris (James, 1999), consequently young alluvial deposits are common throughout the study area.

3.0 Surficial Geologic Mapping

Previous geologic mapping in the study area along the Feather River and surrounding areas generalize the surficial deposits as: Quaternary Alluvium (Qa) and Quaternary stream channel deposits (Qsc) within and proximal to the modern Feather River channel, (Helley and Harwood, 1985). These map units are considered Holocene age (less than 11,000 years old). Late Quaternary Modesto Formation (Qmu, Qml) is mapped along the western margin of the floodplain. These map units were delineated by Helley and Harwood (1985) at a regional scale (i.e., 1:62,500). The current analysis of the Feather River uses this geologic framework as a basis for more detailed mapping of late Holocene alluvium and geomorphic features (Plate 1).

The surficial geologic map units within the southern Feather River study area are described below, in order from oldest to youngest. Surficial geologic mapping for this study subdivides these map units and delineates individual deposits based on relative age and depositional process or environment (Plate 1). The map units depicted on Plate 1 are based primarily on analysis of 1937-vintage photography, and thus the map essentially is a “snapshot” of geologic conditions at this time.

The oldest unit exposed along the Feather River is the lower member of the Riverbank Formation (Qrl) of Helley and Harwood (1985). This unit is a highly dissected alluvial surface with textures of weathered gravel, sand and silt with strong soil-profile development. The Riverbank Formation is semi-consolidated, and is associated with the presence of a well-developed hardpan (or, duripan) layer that is a product of soil-forming processes over substantial geologic time. This hardpan layer reflects an ancient land surface that locally is buried by younger deposits. The Riverbank Formation is late to middle Pleistocene in age, and is estimated to be 130,000 to 450,000 yrs old (Helley and Harwood, 1985). The upper member is unconsolidated dark brown to red alluvium consisting of gravel, sand, silt and minor clay (Busacca et al., 1989, Helley and Harwood, 1985).

The Modesto Formation is divided into two members, a lower (older) unit that is latest Pleistocene in age (about 29,000 to 49,000 years old), and consists of unconsolidated slightly weathered gravel, sand, silt and clay. The upper member, a younger unit, is latest Pleistocene age (circa 12,000 to 26,000 years old) (Helley and Harwood, 1985). This unit (Qmu) is composed of sand, silt, and some gravel, comprising river channel and floodplain deposits, and is associated with a moderate amount of secondary (pedogenic) clay accumulation. This clay-rich horizon may form laterally continuous zones of low hydraulic conductivity, and may extend across boundaries between coarse and fine-grained strata within the latest Pleistocene alluvium. Soils on the Modesto Formation deposits include the Gridley loam of Strahorn et al. (1909) and the Conejo complex of Lytle et al. (1988).

Latest Holocene deposits overlie or are inset into the Modesto Formation, and are categorized as channel, floodplain, and basin deposits (Plate 1). Channel deposits include Holocene channels (Hch), distributary channels (Hdc), overflow channels (Hofc), sloughs (Hsl), in-stream or lateral bars (Hb), and meander scrolls (Hms). These deposits likely consist of fine to coarse sand, silty sand, and clayey sand, with trace fine gravel. Holocene channel deposits (Hch), which are present along Gilsizer Slough and the western floodplain as secondary channels, contain fining-upward sequences of sand, silt and clay. Overflow channels (Hofc) transport water across the land surface during high flow stages toward Sutter Basin. Networks of sloughs wander across the distal floodplain, and are likely filled with a fining-upward sequence of silt and clay (map unit Hsl). These deposits are associated with former channels, and generally are present landside (outboard) of the present-day human-made levees.

Holocene floodplain deposits include crevasse splays (Hcs), distributary fans (Hdf), and overbank deposits (Hob). Crevasse splays (Hcs) are sandy deposits that form from breaching of river banks or natural levees. Distributary fan deposits (Hdf) occur when water and velocity within a distributary channel decreases, can no longer transport its sediment load, and sediment is laid down on the floodplain. Overbank sediments are formed by localized overtopping of river banks or natural levees, subsequent deposition from shallow sheet flow or standing water. Basin deposits occur on the distal floodplain and include undifferentiated basin deposits (Qn), and marsh deposits (Qs). Basin and marsh deposits are present in the topographically low areas west of the present-day natural levees along the Feather River. These deposits consist of fine sand, silt, and clay laid down in a relatively low-energy depositional environment. Soils developed on these deposits are the Sacramento series silt loam, fine sandy loam, clay, Alamo clay loam adobe and Stockton clay adobe. Marsh deposits are generally saturated and are often underwater in the present-day environment. Undifferentiated Holocene and Quaternary alluvium (Ha and Qa, respectively) usually are proximal to the river channel, and this map unit is used in areas where geomorphic features are obscured or obliterated by historical (1937-era) agriculture or cultivation. The deposits within these agriculturally modified areas are assigned a relative age (Ha or Qa) based on overlapping and cross cutting relationships with the surrounding deposits as follows: Ha if the agriculture-modified area is mapped within or shown overlying Holocene deposits; or Qa where it is difficult to evaluate the surface age based on the nearby deposits. Soils associated with these, undifferentiated units (Qa) are the Sacramento silt loam and Sacramento fine sandy loam, (Strahorn et al., 1909), and the Columbia fine sandy loam of Lyle et al. (1988), which are weakly developed soils commonly developed on relatively young deposits.

Historical deposits mapped in the area include stream channel and floodplain deposits, as well as artificial fill deposits (L and SP) (Plate 1). Historical deposits are estimated to be less than 150 years old, dating from approximately 1800 to 1937. Historical stream channels (Rch), distributary channels (Rdc), and overflow channels (Rofc) within the floodplain are recently abandoned channels or reflect active channels with low water flow. Lateral bar deposits (Rb) and meander scrolls (Rms) are located adjacent to the present-day Feather River, and are generally present inboard (waterside) of the present-day Feather River levees. When the river overtops its banks, distributary channels (Rdc) and recent overflow channels (Rofc) transport water and sediment across the floodplain. These channel deposits likely consist of silt and sand with traces of gravel. The upper few feet of these deposits probably are filled with debris from upstream hydraulic mining activities. Historical sloughs transport low velocity water flow derived from distributary channels proximal to the Feather River onto the distal floodplain and into the Sutter Basin. Slough deposits (Rsl) likely consist of fining-upward silt and clay.

Historical flood plain deposits include crevasse splay (Rcs), distributary fan (Rdf), and overbank (Rob) deposits, which generally consist of a fining upward or episodic fining upward sequence of sand, silt, and clay. Historical overbank (Rob) deposits are slightly finer grained sand, silt, and clay deposited via sheet flow when the river is at flood-stage and overtops natural and artificial levees. These historical deposits are differentiated based on cross-cutting and superposition relationships relative to existing cultural deposits visible on the 1937 photographs. Historical alluvial deposits (Ra), generally located within the Feather River channel, consist of undifferentiated sand, silt, and minor lenses of gravel. Historical artificial fills (map units L and SP) are culturally-emplaced heterogeneous deposits, with varying amounts of clay, silt, sand, and gravel from local sources. These deposits include levee structures and canal levee systems (L), and some undifferentiated soil piles (SP), and are shown on the surficial geologic map where present and identifiable on the 1937 photography.

Mapping of historical and Holocene deposits shown on Plate 1 generally is consistent with early, less-detailed soil survey mapping along the western banks of the Feather River as areas of Gridley loam, Sacramento Series fine sand, sandy loam and silt loam soils (Strahorn et al., 1909). The Gridley loam occurs along the northern Feather River from Thermalito south to the confluence with the Bear River, and closely corresponds to the Modesto Formation of Helley and Harwood (1985). The relationship between the mapped surficial geologic units and the potential for underseepage is summarized below.

4.0 Geomorphic Conceptual Model

The preliminary conceptual model described here is based on general relationships among surface and subsurface geologic deposits along the Feather River, as described above and shown on Plate 1. This conceptual model provides a consistent basis for understanding the type and stratigraphy in the area.

Published geologic maps of the project area identify a complex series of westward aggrading alluvial fans and terraces derived from the Sierra Nevada, identified as the Riverbank and Modesto formations. The Riverbank Formation and Modesto Formation are semi-consolidated to unconsolidated deposits characterized by intraformational paleochannels and lateral and vertical stratigraphic complexity related to past fluvial processes and buried paleo-topography. The Riverbank Formation unconformably overlies the Laguna Formation, which is a deeply dissected alluvial surface (Busacca et al., 1989).

Subsurface deposits about 150 feet beneath the ground surface rest on a resistant volcanic tuff capped by interbedded alluvial gravel, sand and silt, interpreted as Pliocene-Pleistocene age Laguna Formation that represents a period of relatively stable landscape conditions (Helley and Harwood, 1985). The Laguna Formation is overlain by the Pleistocene Riverbank Formation, (very dense gravel deposits) that are, in turn, overlain by a medium dense sand and gravelly sand package of the latest Pleistocene Modesto Formation (Busacca et al., 1989). The upper member of the Modesto Formation is exposed at the ground surface adjacent to the western bank of the Feather River south of Marysville and Yuba City. The Modesto Formation is mantled by unconsolidated deposits of Holocene age that comprise most of the surficial geologic deposits along the western side of the Feather River (Plate 1).

Geomorphic evidence suggests that the Feather River system south of Yuba City may have been located west of its present course (Figure 3). The present-day Gilsizer Slough diverges from the modern Feather River directly north of Yuba City and trends southwestward toward the Sacramento River. Alluvial deposits of Gilsizer Slough are inset (i.e. incised) into the Modesto Formation from Yuba City southward. The ancestral Gilsizer Slough perhaps extended to as far as the Sacramento River (Figure 3), based on surficial mapping not included in this report, and inspection of topographic maps. The ancestral Gilsizer Slough deposits are related to discharges and sediment loads that were higher than present-day conditions, and perhaps is an ancestral course of the Feather River.

Surficial geologic deposits near the Yuba City airport indicate the Feather River occupied an intermediate position between ancestral and present locations. The river occupied an abandoned channel arm north of Shanghai Bend, located between Gilsizer Slough and the modern Feather River (Figure 3). From this point the river continued southward in nearly its present location. This paleochannel had a sharp, more exaggerated bend than the present-day channel at Shanghai Bend (Figure 2). The channel subsequently moved eastward, laterally backfilling and abandoning the meander above Shanghai Bend, and moved to the rivers' present location closer to Marysville. Today, Gilsizer Slough is a natural bypass for high water flow stages on the Feather River, in the area between Marysville and Yuba City (Ellis, 1939).

Surficial geologic mapping (Plate 1) shows differences in deposit type and distribution from north to south along the Feather River, which is associated with changes in watershed production of water and sediment, related geomorphic processes, soil profile development, and the underlying subsurface hardpan layer. These differences illustrate the diversity of past geomorphic processes along the river and, as a consequence, the type of geologic deposits at and near the ground surface. The surficial geologic map allows the delineation of reaches along the river within which geomorphic processes and their associated deposits appear to be relatively consistent.

Between Yuba City on the north to the confluence with the Sutter Bypass on the south, the southern Feather River consists of four major reaches, each having characteristic deposit types and distributions. The river reaches are numbered Southern Feather one through four (SF-I through SF-IV), sequentially from north to south (Plate 1, Figure 3). This report describes the surficial geologic characteristics of Reach SF-I, SF-II, SF-III and SF-IV of the southern part of the Feather River, extending from Yuba City, south to the confluence with the Sutter Bypass.

Reach SF-I, extends from the north end of Yuba City to the Yuba City airport, and is about 1.15 miles long (Plate 1, Figure 3). The Project levee along Reach SF-I trends roughly north-south, and overlies alluvial sediments deposited by the Feather River. In Yuba City the levee rests on Holocene deposits associated with Gilsizer Slough that are inset into the upper member of the Modesto Formation. The active Feather River channel is east of, and inset to these Holocene channel deposits (Figure 4).

The second reach of south Feather River project area, SF-II, extends from the Yuba City airport south to Shanghai Bend, and is about 2.9 miles long. Near the Yuba City airport, and south of the confluence of the Feather and Yuba Rivers, young channel deposits are inset against the Gilsizer Slough channel deposits (Plate 1). From the Yuba City airport, south to Epley Drive (about 1.5 miles), the levees overlie historical alluvium of mining debris origin, map unit Ra. From Epley Drive south to Shanghai Bend Road the levees (about 1.4 miles) overlie historical meander scrolls, map unit Rms, (Figure 2, Plate 1). The levee along this reach, SF-II, primarily overlies Holocene channel fill, historical alluvium and overbank deposits. These channels are likely filled with a fining-upward sequence of gravel, sand and silt, the upper few feet of these features are probably covered by a veneer of sediment derived from upstream hydraulic mining activities (Figure 4).

River Reach SF-III extends from Shanghai Bend on the north to just south of the confluence with Bear River, and is approximately 12 miles long (Plate 1). Along Reach SF-III, the active river floodplain is inset into the upper member of the Modesto Formation. Over geologic time, flooding has lead to the vertical accretion of overbank and crevasse splay deposits onto the Modesto Formation west of the Feather River. Overflow channels and related deposits (Rofc) are common along this reach of the river. Beginning at Shanghai Bend and continuing southward are seven overflow channels that range from approximately 100 to 200 feet wide. The Project levees overlie these channels in the area around Messick Road (Plate 1). A few overflow channels conduct water flow immediately landside of the levees, across a short distance between Shanghai Bend and Oswald Avenue, then converge with the Feather River. The overflow channels are slightly inset to the Modesto Formation, and based on borehole data from locations where these channels cross the Sutter Bypass, are probably 6 to 15 feet deep. These channels are likely filled with episodic fining upward sequences of silt, sand and gravel, representing multiple flood events on the Feather River. The upper few feet of these channels are probably filled with sediment from upstream historic hydraulic mining activities. The river channel widens considerably between Country Club Road (0.5 mile width) and Obanion Road (1 mile width), (Plate 1). Feather River meanders along the eastern edge of Abbott Lake, swings sharply southward into Star Bend, where the river is deflected eastward by a resistant knob of Modesto Formation (which forms Star Bend). Historical crevasse splay and overbank deposits overlie the Modesto Formation from Abbott Road to Star Bend Road, along the western edge of Abbot Lake (Figure 5). These crevasse splay deposits are likely filled with a fining-upward sequence of fine gravel, sand and silt, The upper few feet of these features are probably covered by a veneer of hydraulic mining sediment.

The southernmost reach, Reach SF-IV, extends from the area south of the confluence with the Bear River to the confluence of the Feather River and Sutter Bypass, and is roughly 4 miles long (Plate 1). The sediments underlying the levee along this reach are geomorphically complex, resulting from depositional convergence between the Feather River and Bear River. The Bear River channel deposits large amounts of sediment across the ground surface adjacent to the confluence. The Modesto and Riverbank Formations are exposed at the ground surface adjacent to natural levees immediately north of the Bear River confluence, and north of this reach (Plate 1). These formations are covered by historical alluvium, sourced from the Feather and Bear Rivers. Much of the historical activity along this reach is located near the levee at Laurel Avenue. Here, consisting eight distributary channels (Rdc), typically 90 feet wide but ranging from 45 to 190 feet wide, cross the floodplain in southwesterly orientations, terminating in geologically young distributary-fan sediments. These sediments, primarily consisting of fine to coarse sand and silt, probably were deposited as a result of increased sediment and water input contributed to the Feather River from the Bear River. Historically, the Feather River and the Bear River have aggraded from substantial mining debris input, thus reducing channel cross sectional area (i.e., James, 1999). This reduction of cross section area, coupled with the trajectory of flood flow from the Bear River watershed, resulted in water overtopping the Feather River channel banks, and depositing sediment onto the floodplain between the confluence of the Feather River and Sutter Bypass (Plate1).

5.0 Applications to the Urban Levee Project

Based on an initial analysis of surface geologic and geomorphic data, the levees bordering the western side of the Feather River from Yuba City to the Sutter Bypass, (Reaches SF-I, SF-II, SFIII and SF-IV) probably are underlain by a veneer of near-surface sandy deposits, or by buried channels that are inset into the Modesto Formation. The preliminary conceptual surface and subsurface geologic relationships as they relate to levee structures and potential underseepage along each reach of the river are described below. This study does not account for any existing seepage mitigation structures, i.e. slurry wall or cutoff wall, which may be present.

Reach SF-I contains the Gilsizer paleochannel deposits, this channel intersects the levees roughly 660 feet south of Lynn Way to Colusa Avenue (Plate 1). Along this length the levees are underlain by coarse channel deposits. These coarse grain deposits are likely laterally continuous and poorly consolidated and relatively highly permeable, and likely are susceptible to underseepage.

Levees along the reach SF-II are underlain by a Holocene paleochannel and historical meander scroll deposits (Figure 2, Plate 1). These deposits are coarse grained, laterally continuous and poorly consolidated, and likely are susceptible to underseepage. The presence of this paleochannel deposit suggests locally permeable material (channel fill) directly underlying the levees. Historical alluvium most likely of mining debris origin, blankets the Yuba City airport paleochannel and meander scroll deposits. The levees along this reach are underlain by a thick sequence of young, permeable alluvium of meander scroll deposits that are highly susceptible to seepage (Glynn and Kuszmaul, 2004).

Reach SF-III consists of coarse-grained avulsion deposits (overbank, crevasse splay and overflow channel deposits) overlying the Modesto Formation. Overflow channels (Rofc) are common along this reach, are relatively thin, slightly inset to the Modesto Formation and are filled with poorly consolidated sediments that may provide local pathways for underseepage. Individual shallow coarse deposits may be laterally discontinuous and may be separated by clayey interbeds (i.e. thin blankets). Local coarse deposits may be associated with higher likelihoods of levee underseepage. Deeper deposits probably consist of consolidated Modesto Formation with occasional small, but unconsolidated, overflow channel deposits incised into resistant strata.

Along Reach SF-IV the levee is underlain by laterally-continuous sandy deposits formed by distributary overbank fans and by the south flowing ancestral Feather River (Gilsizer Slough). These coarse-grained deposits likely are permeable and susceptible to underseepage. Near Laurel Avenue distributary channel deposits underlie the levees and may be relatively coarser than the surrounding alluvium.

6.0 Summary

Lateral and vertical variability in the shallow subsurface deposits has resulted from past geomorphic processes. Surficial geologic mapping along the south Feather River allows reach classifications within which conditions may be relatively consistent. The conceptual subsurface stratigraphic framework suggests that stratigraphic relationships may promote localized levee underseepage, given certain hydraulic conditions, particularly along reach SF-I and II. Further spatial analyses of the surficial geologic mapping and subsurface geotechnical exploration data are needed to better constrain and characterize areas that are most susceptible to underseepage in the study area.

7.0 Limitations

This geomorphic assessment and associated data interpretation have been performed in accordance with the standard of care commonly used as the state-of-practice in the geologic engineering profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this geographic area performing the same services under similar circumstances during the same time period.

Discussions of surface and subsurface conditions summarized in this technical memorandum are based on geologic interpretations of subsurface soil data at limited exploration locations available to this assessment through August of 2007. Variations in subsurface conditions may exist between exploration locations, and the project team may not be able to identify all adverse conditions in the levee and its foundation. This memorandum is for the use and benefit of DWR. Use by any other party is at their own discretion and risk.

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USGS, Marcuse topographic quadrangle, surveyed 1906, published 1910; map scale 1:31,680, five foot contour interval.

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USGS, Ostrom topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.

USGS, Sutter topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.

USGS, Yuba City topographic quadrangle, surveyed 1909, published 1911; map scale 1:31,680, five foot contour interval.

USGS, Gilsizer Slough topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.

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USGS, Sutter topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.

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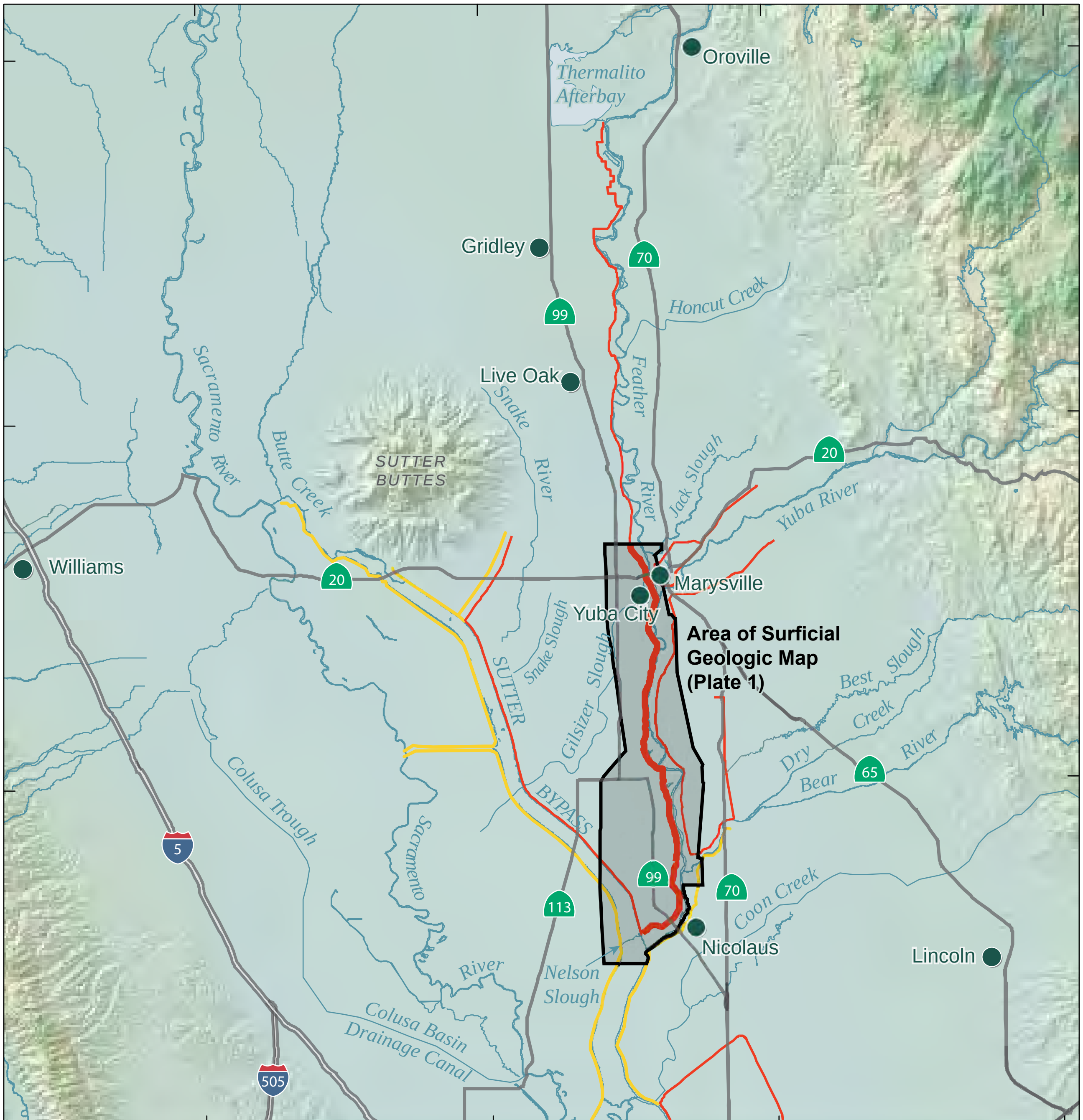
USGS, Yuba City topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.



Sources: NAIP, 2006

Projection: UTM Zone 10, NAD83

Explanation

Southern Feather River project levee

Urban project levee

Other levee

N

0 5 10 mi

0 5 10 km

WLA

Map of Central Valley near Sutter Buttes, California

DWR URBAN LEVEE PROJECT

Figure 1

A. 1937 USDA Air Photo

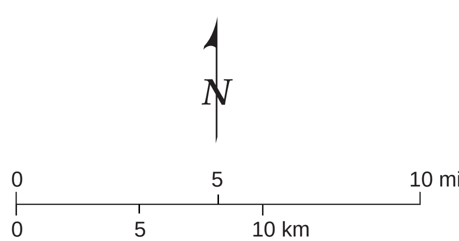
B. 2006 NAIP Ortho Imagery

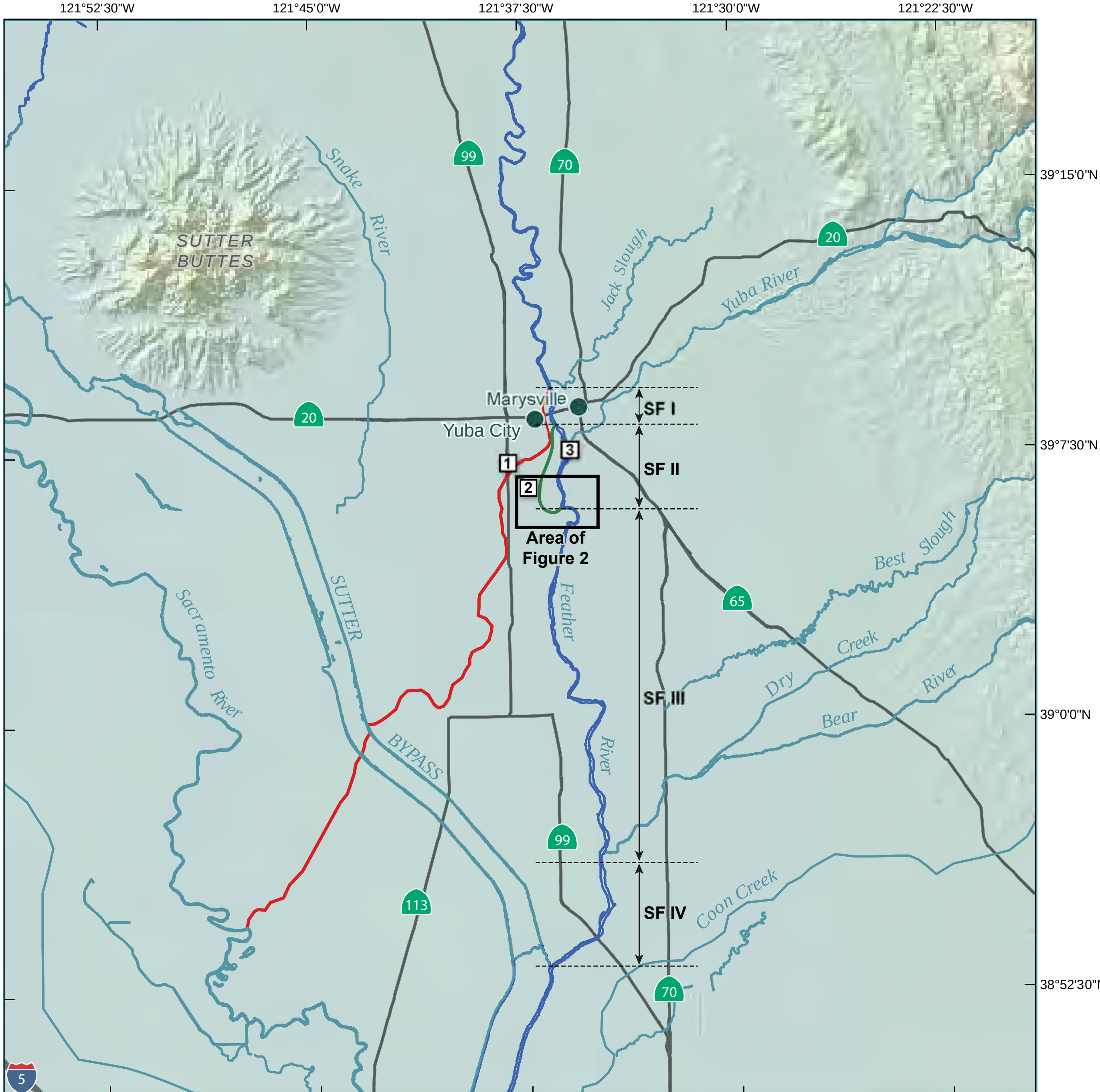
Explanation

Southern Feather River project levee

Urban project levee

Major road





Source: ESRI, 2006

Projection: UTM Zone 10, NAD83

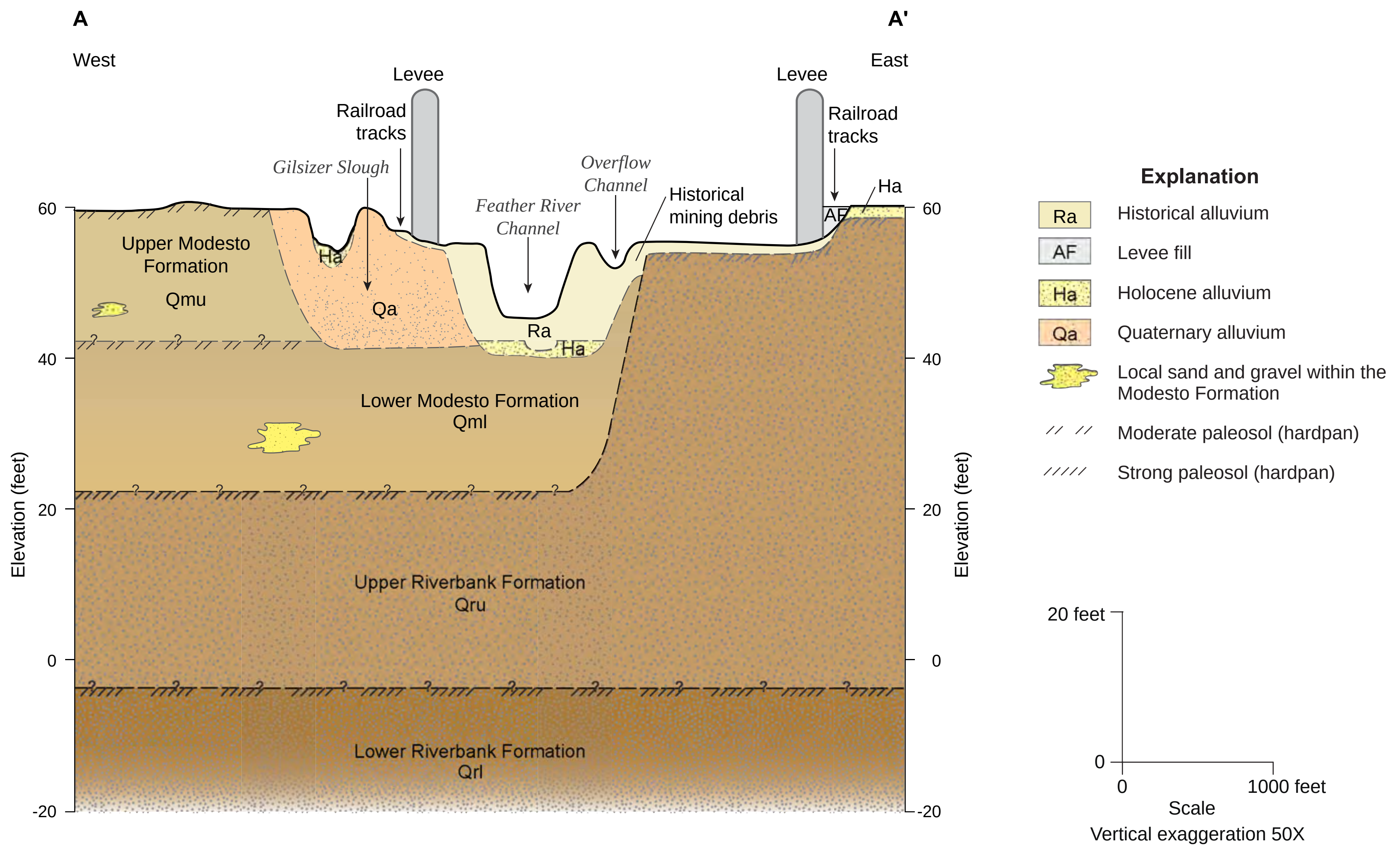
Explanation

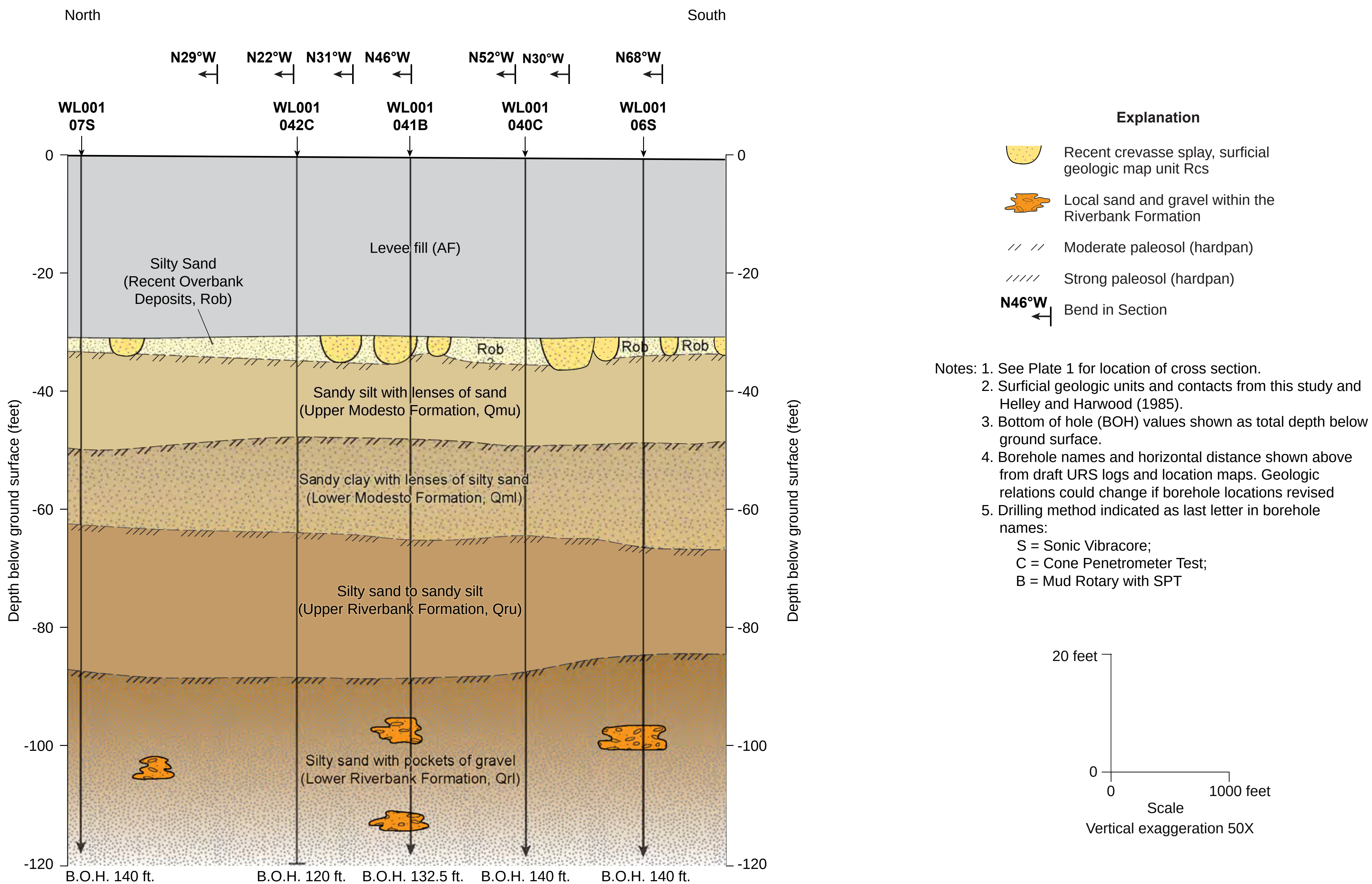
- 3 Modern Feather River Channel
- 2 Abandoned arm of the Feather River
- 1 Ancestral Feather River Channel (Gilsizer Slough)
- **SF I** Reach of Southern Feather River Study Area

N

0 5 mi

0 5 km


A
A'
West
East
Levee
Levee
Railroad tracks
Railroad tracks
Gilsizer Slough
Feather River Channel
Overflow Channel
Historical mining debris
Ha
AF
Upper Modesto Formation
Qmu
Qa
Ha
Ra
Ha
Lower Modesto Formation
Qml
Upper Riverbank Formation
Qru
Lower Riverbank Formation
Qrl
Elevation (feet)
60
40
20
0
-20
Explanation
Ra Historical alluvium
AF Levee fill
Ha Holocene alluvium
Qa Quaternary alluvium
Local sand and gravel within the Modesto Formation
Moderate paleosol (hardpan)
Strong paleosol (hardpan)
20 feet
0
0
1000 feet
Scale
Vertical exaggeration 50X



Notes:
1. See Plate 1 for location of cross section.
2. Surficial geologic units and contacts from this study and Helley and Harwood (1985).
3. Bottom of hole (BOH) values shown as total depth below ground surface.
4. Borehole names and horizontal distance shown above from draft URS logs and location maps. Geologic relations could change if borehole locations revised
5. Drilling method indicated as last letter in borehole names:
   - S = Sonic Vibracore;
   - C = Cone Penetrometer Test;
   - B = Mud Rotary with SPT


Plate 1 - Surficial Geologic Map of the Feather River, Southern Section
This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on modern U.S. Geological Survey topographic base map prepared in 1952 and photo revised in 1973. See accompanying report for complete descriptions of map units, process descriptions and methodology.
Explanation
Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain; solid contacts within 30' of line shown on map.
SF-II
Geomorphic Reaches Discussed in Text
Project Levees
A A'
Cross Section
WSL001_006S
Borehole locations from URS, Corp; locations from Sutter P2 Boring Locations, Region 1 Northern Area map.
W 1937
Water bodies, circa 1937.
BP
Borrow pits present in 1937.
Geologic Units
L Levees (artificial fill), circa 1937.
SP Spoils pile (artificial fill), circa 1937.
HISTORICAL
Rob Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
Rcs Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
Rdf Distributary fan deposits; sand, silt and clay.
Rch Channel deposits; well sorted sands and fine gravels.
Rms Channel meander scroll deposits; sand, silt and clay from lateral channel migration.
Rb Channel bar deposits; fine gravel, sand, and silt deposited in or along channel lateral margins.
Rcu Cut off channel (chute); occurs on insides of meander bends within the river channel; on flood plain – entrenchment of overflow channels into hydraulic mining debris.
Rdc Distributary channel deposits, trace gravel, sand, silt, and clay; channelized flow conducting sediment to floodplain.
Rofc Overflow channels; vertically stratified sand, silt, and clay in floodplain channels occupied primarily when high-stage water overtops channel banks.
Rsl Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Ra Alluvial deposits, undifferentiated; sand, silt, and minor lenses of fine gravel.
HOLOCENE
Hob Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
Hcs Crevasse splay deposits; fine to coarse sand, with minor lenses of fine gravel deposited from breaching of natural or artifical levees.
Hdf Distributary fan deposits; sand, silt and clay.
Hofc Overflow channels; vertically stratified sand, silt, and clay in channels occupied when high-stage water overtops channel banks.
Hch Channel deposits; sorted sands and silts; fining upward.
Hb Channel bar deposits; fine sand, and silt deposited in or along channel lateral margins.
Hdc Distributary channel deposits, sand, silt, and clay.
Hsl Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Ha Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
Qn Basin deposits; fine sand, silt and clay, dark yellow to dark yellowish brown, under cultivation in 1937.
Qs Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.
PLEISTOCENE
Qmu Modesto Formation; upper member; unconsolidated gravel, sand, silt, and clay.
Qml Modesto Formation; lower member; unconsolidated to semi-consolidated gravel, sand, silt and clay.
Qru Riverbank Formation; upper member, semi-consolidated to consolidated gravel, sand, silt and minor clay.
Qrl Riverbank Formation; lower member; consolidated gravel, sand, silt, and clay, generally associated with strong duripan horizon.
Stratigraphic Correlation Chart
Time
Depositional Environment
Epoch
Channel deposits
Floodplain and alluvial-fan deposits
Flood basin deposits
Cultural deposits
Historical
Rch Rdc Rb Rsl Rms Rofc Rcu
Qa Ra Rcs Rdf Rob
Qs Qn
L SP
Holocene
Hch Hdc Hb Hsl
Ha Hdf Hcs Hob Hofc
Latest Pleistocene
Qmu
Qml
Pleistocene
Qru
Qrl
1:20,000
N
0 0.5 1 Miles
0 0.5 1 Kilometers
Map projection: UTM NAD83 Zone 10N
Topographic base USGS quadrangles:
USGS, Gilsizer Slough topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
USGS, Nicolaus topographic quadrangle, published 1952, remapped 1992; map scale 1:24,000, five foot contour interval.
USGS, Olivehurst topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
USGS, Sutter topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
USGS, Sutter Causeway topographic quadrangle published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
USGS, Yuba City topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
Plate 1 - Surficial Geologic Map of the Feather River, Southern Section
WLA
WILLIAM LETTIS & ASSOCIATES, INC.
Walnut Creek, CA
1881_URS_Feather_South.mxd
09/04/2009
Geologic Mapping by J. Pearce and A. Streig
Digital Cartography by M. Ticci
MARYSVILLE
YUBA CITY
SF-I
SF-II
SF-III
SF-IV

WILLIAM LETTIS & ASSOCIATES, INC.
1777 Botelho Drive, Suite 262, Walnut Creek, California 94596
tel (925) 256-6070 fax (925) 256-6076

September 8, 2009

Mr. Juan Vargas
URS Corporation
2870 Gateway Oaks Drive, Suite 150
Sacramento, CA 95833

RE: Surficial geologic mapping and geomorphic assessment, California Department of Water Resources Urban Levees Project, Northern Feather River, Sutter County, California

Dear Mr. Vargas:

This letter presents the surficial geologic mapping and preliminary geomorphic assessment of the northern Feather River study area, for the California Department of Water Resources (DWR) Urban Levees Project geotechnical characterization. The goal of this mapping and geomorphic assessment is to provide information on the type and distribution of surface and shallow subsurface deposits that likely underlie the project levees along the western bank of the Feather River between Thermalito Afterbay and Yuba City. The purpose of this study is to develop spatially continuous geologic map data and a conceptual model for stratigraphic interpretations between shallow boreholes. A primary goal is to provide a geologic framework for the geotechnical assessment of potential levee underseepage. This letter presents the technical approach, surficial geologic map, conceptual geomorphic model, and initial results based on map analysis and preliminary review of available Phase 1 geotechnical data.

We appreciated the opportunity to provide these geomorphic and geologic data and preliminary interpretations of the shallow stratigraphic conditions in the northern Feather River study area. Please do not hesitate to call any of the undersigned if there are any questions or comments.

Respectfully,

WILLIAM LETTIS & ASSOCIATES, INC.

Justin Pearce

Justin Pearce, C.E.G. 2421
Senior Geologist

Ashley R. Streig

Ashley Streig
Senior Staff Geologist

Keith Kelson

Keith Kelson, C.E.G. 1610
Principal Geologist

1.0 Introduction

This technical memorandum presents the results of surficial geologic mapping and geomorphic assessment along the north Feather River between Thermalito Afterbay and Yuba City, for the California Department of Water Resources Urban Levee program. The purpose of this study is to provide detailed information on the type and distribution of surface and shallow subsurface deposits that likely underlie the project levees, with respect to levee underseepage. This study involved integration and analysis of aerial photography, topographic, geologic, and soil maps, other historical documents, and review of readily available geotechnical exploration data. Synthesis of these data allowed us to assess the geomorphic processes responsible for the distribution of surficial deposits within the project area, and construct a preliminary conceptual model for stratigraphic interpretations. This technical memorandum is accompanied by the "Surficial Geologic Map of the Feather River, Northern Section".

1.1 Map and Report Preparation Quality Control

The surficial geologic map data and geomorphic interpretations presented in this memorandum were subject to quality control and quality assurance procedures as required by the Levee Geotechnical Evaluation Project Management Plan (PMP). The surficial geologic map data developed by this study were reviewed for accuracy and completeness through an internal review and an independent technical review by Dr. Janet Sowers of WLA. Results of QA/QC review were documented using PMP Exhibit 2.2-3 (Independent Technical Review Report) and are kept on file according filing control plan. Subsurface data shown on diagrams were provided as draft information, and were not verified for accuracy or completeness by this study.

2.0 Approach

The approach to developing a surficial geologic map of the northern Feather River project area (Figure 1, Plate 1) consisted of analysis of the following data:

- Aerial photography (black and white stereo-pairs taken in 1937, ~1:20,000-scale);
- early USGS topographic maps (i.e., 1911);
- published surficial geologic maps (Bussaca et al., 1989; Helley and Harwood, 1985; Creely, 1965);
- early and modern soil survey maps (Strahorn et al., 1909; Lytle, et al., 1988);
- other maps and documents (Page, 1985).

Synthesis of these data allow for the development of a detailed surficial geologic map that provides an initial understanding of primary geomorphic processes that have acted in the study area during recent and historical geologic time. Through this mapping, primary geomorphic features and associated surficial geologic deposits are identified, such as abandoned paleochannels, channel deposits, splay and overbank deposits and other deposits commonly associated with large active river systems. Knowledge of fluvial processes and the ability to recognize depositional environments in the geologic record are key to identifying locations along levees where underseepage is most likely to occur (Llopis et al., 2007).

The surficial geologic map was developed at the nominal scale of the aerial photography (1:20,000). This scale establishes the resolution of the map (Plate 1), such that display or analysis of the map data at a more detailed scale than 1:20,000 may introduce uncertainties beyond the original resolution of the data. The map unit boundaries shown on the surficial geologic map should be considered approximate, and accurate within 30 feet on either side of the line shown on the map. The 1937 aerial photographs are the primary data set for interpreting the surficial geologic deposits because: (1) they are the oldest high-quality images that pre-date much of the urbanization and landscape alteration within present-day Sutter and Butte Counties and, (2) these data represent a close approximation to the surficial deposits that were likely present at the ground surface prior to the construction of the levees. The 1937 photographs generally were taken in late summer or early autumn (i.e., August). By 1937, the area had experienced moderate cultivation that locally obscures geomorphic conditions. However, integration of data from the 1937 photography, old and recent topographic maps, geologic maps, soil surveys and historical documents provides sufficient information to delineate many of the pre-historical and historical surficial deposits in detail. Taken together, these data provide key insights to the characteristics of shallow deposits beneath the levees, as well as the geomorphic processes responsible for their distribution.

Additional floodplain deposition may have occurred after 1937, due to flood overflows, levee overtopping, or localized levee failure. A time series analysis that interprets successive aerial photographs taken after major flood events (i.e., USDA, black and white stereo-pairs taken in 1958, ~1:20,000-scale) or known local levee failures (i.e., 1986) may reveal additional information on surficial deposits in the northern Feather River area. Such analyses are beyond the scope of this study. The data and interpretations presented herein address the primary goal of characterizing the type and distribution of deposits likely present directly beneath the project levees that may be conducive to underseepage.

3.0 Geologic Setting

The northern Feather River study area lies in the central Sacramento Valley, between the Coast Ranges to the west and the Sierra Nevada foothills to the east. The Feather River drains the western slope of the Sierra Nevada, and emerges from the mountains south of Thermalito Afterbay (Figure 1). The river flows southward from Thermalito Afterbay, over middle –to late Pleistocene alluvium derived from the Sierra Nevada. The regional land surface is nearly flat, with a gentle west-southwest slope that flattens south of the Sutter Butte. The Feather River is entrenched into middle-to-late Pleistocene semi-consolidated sediments (i.e. Modesto Formation). Historical alluvium deposited by the Feather River is present between the modern levees, inset to the older geologic formations, and is present on the western floodplain as subdued natural levees that mantle the older geologic formations. In this study reach, west-flowing Honcut Creek is the only drainage tributary to the northern Feather River, with a confluence east of the town of Live Oak (Figure 1).

A primary influence on the historical processes in the river system was the hydraulic mining that began in the 1850's. Mining continued through the early 1900's in the Feather, Yuba and

Bear River watersheds, and abruptly introduced large quantities of sediment and drastically changed the geomorphic characteristics of these river systems (DWR, 2004; Ellis, 1939). Aggradation within the stream channels was a primary response to the introduction of substantial mining debris (James, 1999); consequently, post-1850 alluvial deposits are common throughout the study area.

4.0 Surficial Geologic Mapping

Previous geologic mapping along the northern Feather River and surrounding areas generalize the surficial deposits as: Quaternary alluvium (Qa) and Quaternary stream channel deposits (Qsc) are mapped within and proximal to the modern Feather River channel, (Bussaca et al., 1989; Helley and Harwood, 1985; Creely, 1965). These map units are considered Holocene in age (less than 11,000 years old). Late Pleistocene Modesto Formation (Qmu, Qml) is present as an escarpment along the western margin of the floodplain. These map units were delineated by Helley and Harwood (1985) at a regional scale (i.e., 1:62,500). The current analysis of the northern Feather River uses this geologic framework as a basis for more detailed mapping of Quaternary deposits and geomorphic features (Plate 1). The surficial geologic map units within the northern Feather River study area are described below, in order from oldest to youngest. Surficial geologic mapping for this study subdivides these general map units and delineates individual deposits based on relative age and depositional process or environment. The map units depicted on Plate 1 are primarily based on analysis of 1937 aerial photography, and thus the map essentially is a “snapshot” of geologic conditions at this time.

The oldest unit exposed along the Feather River is the lower member of the Riverbank Formation (Qrl) of Helley and Harwood (1985). The Riverbank Formation is a semi-consolidated, highly-dissected alluvial surface with textures of weathered gravel, sand and silt, and is associated with the presence of a well-developed hardpan (or, duripan) layer. This hardpan layer is a product of soil-forming processes over substantial geologic time, and reflects an ancient land surface that locally is buried by younger deposits. The Riverbank Formation is late to middle Pleistocene in age, and is estimated to be 130,000 to 450,000 yrs old (Helley and Harwood, 1985). The upper member (map unit Qru; Plate 1) is poorly consolidated dark brown to red alluvium consisting of gravel, sand, silt and minor clay (Busacca et al., 1989, Helley and Harwood, 1985). West of the Feather River, the Riverbank Formation is present near the town of East Biggs (Plate 1). Soils developed on the Riverbank formation are the Gridley clay loam and the Redding gravelly sandy loam (Carpenter et al., 1926).

The latest Pleistocene Modesto Formation is informally divided into two members: a lower (older) unit that is (about 29,000 to 49,000 years old), and consists of unconsolidated slightly weathered gravel, sand, silt and clay; and an upper member, a younger unit, that is about 12,000 to 26,000 years old (Helley and Harwood, 1985). The upper Modesto (map unit Qmu) consists of sand, silt, and some gravel, and is associated with a moderate amount of secondary (pedogenic) clay accumulation. This clay-rich horizon may form laterally continuous zones of low hydraulic conductivity, and may extend across boundaries between coarse and fine-grained strata within the latest Pleistocene alluvium. Soils developed on the Modesto Formation include the Gridley loam of Strahorn et al. (1909) and the Conejo complex of Lytle et al. (1988), both of which are associated with a shallow "siltstone" horizon, or duripan (hardpan).

Latest Holocene deposits overlie or are inset into the Modesto Formation, and are categorized as channel, floodplain, and basin deposits (stratigraphic correlation chart; Plate 1). Channel deposits include Holocene channels (Hch), sloughs (Hsl), in-stream or lateral bars (Hb), and meander scrolls (Hms). These deposits likely consist of fine to coarse sand, silty sand, and clayey sand, with trace fine gravel. Holocene channel deposits (Hch) present along the western map area as secondary channels, contain fining-upward sequences of sand, silt and clay. These sloughs (map unit Hsl) are former channels associated with Live Oak and Morrison Sloughs (Plate 1), and are likely filled with a fining upward sequence of silt and clay.

Holocene floodplain deposits include crevasse splays (Hcs), and overbank deposits (Hob) and are typically deposited by non-channelized flow. Crevasse splays (Hcs) are from breaching of river banks or natural levees and are usually sand rich. Overbank deposits form by localized overtopping of river banks or natural levees, and subsequent deposition from shallow sheet flow or standing water.

Undifferentiated Holocene and Quaternary alluvium (Ha and Qa, respectively) usually occur proximal to or within the river channel, (Plate 1). The undifferentiated map unit is used in areas where geomorphic features are obscured or obliterated by historical (1937-era) agriculture. The deposits within these agriculturally modified areas are assigned a relative age (Ha or Qa) based on overlapping and cross cutting relationships with the surrounding deposits as follows: Ha if the agriculture-modified area is mapped within or shown overlying Holocene deposits; Qa where it is difficult to evaluate the age based on the relationship with nearby deposits. Soils associated with these undifferentiated units (Qa) are the Sacramento silt loam and Sacramento fine sandy loam, (Strahorn et al., 1909), and the Columbia fine sandy loam of Lyle et al. (1988), which are poorly-developed soils commonly associated with relatively young deposits (i.e. Shlemon, 1967).

Historical deposits mapped in the Northern Feather Study area include channel and floodplain deposits, as well as artificial fill deposits (Plate 1). Historical deposits are estimated to be less than about 150 years old, dating from approximately 1800 to 1937. Historical stream channels (Rch), and overflow channels (Rofc) transport high stage water flow across the ground surface outboard of the levees. These channel deposits likely consist of silt and sand with traces of gravel. The upper few feet of these deposits probably are filled with debris derived from upstream hydraulic mining activities. Lateral bar deposits (Rb) and meander scrolls (Rms) are located adjacent to the present-day Feather River, and are generally present inboard (waterside) of the present-day Feather River levees. In the northern part of the study area, directly south of Thermalito, are multiple anastomosing chutes (map unit Rcu; Plate 1). These chutes are similar to overflow channels in that they transport water flow during high river stage across the ground surface outboard of the levees. These chutes are entrenched into fluvially deposited hydraulic mining debris, and likely have filled with re-worked mining debris. Historical sloughs transport water collected from sheet flow and overland flow west of the Feather River southerly toward the Sutter Basin (i.e., Live Oak and Morrison Slough). Slough deposits (Rsl) likely consist of fining-upward silt and clay. Historical flood plain deposits include crevasse splay (Rcs), and overbank (Rob) deposits, which generally consist of a gradational or abrupt fining upward sequence of sand, silt, and clay. Historical overbank (Rob) deposits are slightly finer grained sand, silt, and clay deposited via sheet flow. These historical deposits are differentiated from older deposits based on cross-cutting and superposition relationships relative to cultural features visible on the 1937 photographs.

Historical alluvial deposits (Ra), generally located between the Feather River channel levees, and on the land side of the levees in the area directly south of the Thermalito Afterbay, consist of undifferentiated sand, silt, and minor lenses of gravel. Soils associated with this sandy alluvium are the Columbia very fine sandy loam and Columbia loam, as shown on the Soil Survey Map of the Oroville Area (Carpenter et al., 1926). This series of soils has been correlated with Holocene age deposits by Shlemon (1967). Historical artificial fills are culturally-emplaced heterogeneous deposits, with varying amounts of clay, silt, sand, and gravel from local sources. These deposits include levee structures and canal levee systems (map unit L; Plate 1) and dredge tailings (map unit DT).

The distribution of historical and Holocene deposits shown on Plate 1 generally is consistent with early, less-detailed soil survey mapping along the western banks of the Feather River as areas of Marcuse clay loam, Gridley loam, Sacramento Series fine sand, sandy loam and silt loam and the Columbia very fine sandy loam soils (Strahorn et al., 1909; Carpenter et al., 1926). The Gridley loam occurs along the northern Feather River from the Thermalito Afterbay south to the confluence with the Bear River, and closely corresponds to the Modesto Formation of Helley and Harwood (1985). The relationship between the mapped surficial geologic units and the potential for underseepage is summarized below.

5.0 Geomorphic Conceptual Model

This section provides a preliminary geomorphic conceptual model based on general relationships among surface and subsurface geologic deposits along the western side of the Feather River, as described above and shown on Plate 1. This conceptual model provides a consistent basis for understanding the type and distribution surficial geologic deposits, primary geomorphic processes, and shallow subsurface stratigraphy in the study reach. This conceptual model does not address planform or gradient changes of the Feather River itself, nor the susceptibility of stream banks to erosion. Future studies of these changes would be valuable in understanding process response of the Feather River, and provide key data for estimating rates of channel changes (i.e. lateral migration). However, these analyses are not directly relevant to evaluating the possibility of underseepage with respect to levee stability.

Published geologic maps of the project area show a complex series of westward aggrading alluvial fans and terraces derived from erosion of the Sierra Nevada, identified as the Riverbank and Modesto Formations (Bussaca et al., 1989; Helley and Harwood, 1985; Creely, 1965). The Riverbank Formation and Modesto Formation in general are semi-consolidated to unconsolidated deposits characterized by intraformational paleochannels and lateral and vertical stratigraphic complexity related to past fluvial processes and buried paleo-topography. The oldest map unit, the Riverbank Formation unconformably overlies the Pliocene-Pleistocene age Laguna Formation, which consists of interbedded alluvial gravel, sand and silt (Busacca et al., 1989; Helley and Harwood, 1985). The overlying Pleistocene Riverbank Formation consists of very dense gravel deposits that are, in turn, overlain by a medium dense sand and gravelly sand package of the latest Pleistocene Modesto Formation (Busacca et al., 1989). The upper member of the Modesto Formation is exposed at the ground surface adjacent to the western bank of the Feather River. The Modesto Formation is locally mantled by unconsolidated, sand rich Holocene deposits (Plate 1). East of the Feather River the older stratigraphic units are uplifted and dissected and younger deposits are inset into them with older deposits buried beneath younger deposits. West of the Feather River, the stratigraphic units are found in typical succession. This is the result of overall westward tilting and uplift of the Sierra Nevada, incision along the tributary drainages (i.e. Honcut creek), and progradational fan deposition west of the river.

Surficial geologic mapping (Plate 1) shows differences in deposit type and distribution from north to south along the northern Feather River study area, which are primarily associated with proximity to the Sierra Nevada mountain front near Thermalito Afterbay. These differences illustrate the diversity of past geomorphic processes along the river and, as a consequence, the type of geologic deposits at and near the ground surface. The surficial geologic map allows the delineation of reaches along the river within which geomorphic processes and their associated deposits appear to be relatively consistent.

The northern Feather River project area is divided into three reaches based on characteristic deposit types and distributions. The levee reaches are numbered Northern Feather one through three (NF-I through NF-III), sequentially from north to south (Figure 2, Plate 1). This section describes the surficial geologic characteristics of Reach NF-I, NF-II, and NF-III of the Feather River between Thermalito Afterbay and Yuba City.

5.1 Reach NF-I

Reach NF-I extends from the Thermalito Afterbay to Reimer Road and is about 11.1 levee miles long (Plate 1). Widespread deposits of historical alluvium (map unit Ra) blanket the area adjacent to the Feather River along the length of this reach where the river flows in the Sacramento Valley. Much of this unconsolidated historical alluvium contains clasts from many source lithologies and is derived from hydraulic mining debris (James, 1999). A complex pattern of anastomosing chutes or cut-off channels (map unit Rcu) eroded the historical alluvium by 1937 (Ra). These chutes underlie the project levees along the length of this reach (Plate 1). Project levees were built after 1937 along NF-I, from Thermalito Afterbay south to Ord Ranch Road.

Hardpan horizons were not identified in subsurface data along this reach, suggesting a substantial thickness of unconsolidated alluvial deposits unconformably overlying the Modesto Formation. Three alluvial units were identified in subsurface data overlying a semi-consolidated alluvial unit that we identified as the lower member of the Modesto Formation. Boreholes revealed an approximately 20-foot-thick package of young, unconsolidated silty sands and sandy clays, above a 10 to 16 foot thick silty sand, and 15-to 20-foot-thick gravel bed (Figure 3).

Hydraulic mining debris was dredged for its gold content along the northern half of the river banks along this reach, from Lapkin Road at Thermalito Afterbay to the area just south of Almond Avenue (Plate 1). Some dredge tailing spoils were apparent in 1937 aerial photography, though the majority of dredge tailing spoils post-date these air photos. The full extent of dredging tailing is apparent in modern USGS topographic maps (i.e. USGS, Biggs topographic quadrangle, 1:24,000 scale, 1970) and is shown on this surficial geologic map (map unit DT). Chutes (map unit Rcu) present in 1937 aerial images, though now obliterated by dredge operations are shown as dotted contacts in the Surficial Geologic Map (Plate 1). In this area project levees either overlie or bound the western edge of the Dredge Tailings (map unit DT). South of the dredged areas, the levee along Ord Ranch Road overlies deposits that fill an abandoned channel meander, map unit Hch (Plate 1). This abandoned meander matches the present river geometry and possibly reflects a southward migration of this meander within the active channel.

5.2 Reach NF-II

The second reach of the north Feather River project area, NF-II, extends from Reimer Road to Sanders Road, and has a length of about 9.4 levee miles. In this reach the project levee is typically perched at the top of a 5- to 15-foot-high east-facing escarpment cut into the Modesto Formation. The active meander belt of the Feather River with its flood plain, meander scrolls, and channel deposits, lies to the east of the levee at the base of the escarpment. West of the escarpment, historical overbank (Rob) and crevasse splay (Rcd) deposits locally overlie the Modesto. They represent locations where flooding of the Feather River overtopped the escarpment in the past and are assumed to pre-date the construction of the levee. An extensive continuous Holocene natural levee deposit has not built up along reach II, in contrast to reach I. The river may be incised too deeply below the surface of the Modesto Formation for floods to regularly overtop the escarpment.

Most of the Reach II levee sits directly on Modesto Formation with about 3.5 of the 9.4 miles of the levee sitting on the above-mentioned Holocene overbank and crevasse splay deposits that overlie Modesto Formation. Borehole WL0009_004S (Plate 1), located in the southern portion of this reach, shows project levee fill directly above the hard, consolidated Modesto Formation.

5.3 Reach NF-III

Levee reach NF-III extends from Sanders Road at the north to Yuba City at the south, and is about 4 miles in length (Plate 1). Along this reach the project levee almost entirely overlies Historical alluvial deposits that mantle, or crosscut the Modesto Formation. Crevasse splay (Rcs), overflow channels (Rofc), historical alluvium (Ra), channel deposits (Rch), and overbank deposits (Rob) are present along this reach. Crevasse splay deposits are present at the northern end of NF-III (Sanders Road, Plate 1), directly adjacent to a westerly bend of the Feather River. Aerial photography from 1937 shows multiple generations of crevasse splay deposits at this location. The levee appears to be constructed overtop these deposits. A pump station is noted on the 1970's topographic map, suggesting this location may have had seepage problems.

Immediately south of Sanders Road, an overflow channel (map unit Rofc) diverges from the Feather River, transporting flow outboard of the levees, and flowing back into the river about 1.5 miles south at Rednall Road (Plate 1). The overflow channel likely consists of a fining upward sequence of sand, silt, clay and some gravel, and could be slightly incised into the Modesto Formation. Undifferentiated historical alluvium (map unit Ra) underlies the levees within the area directly east of these overflow channels. This alluvium was laid down over the surface of the Modesto Formation by unchannelized flow of the Feather River (Plate 1). Historical channel deposits (map unit Rch) from the Feather River underlie about 0.7 miles of the levees north of Rednall Road (Plate 1). Overbank deposits are present near Pease Road (Plate 1) and continue along the levee for about 0.5 miles. Historical crevasse splay and overbank deposits likely consist of a massive to fining upward sequence of sand and silt derived from upstream hydraulic mining activities.

6.0 Applications to the Urban Levee Project

Based on an initial analysis of surface and subsurface geologic and geomorphic data, the levee bordering the western side of the Feather River from the Thermalito Afterbay to Yuba City, overlies three different types of deposits, Reach NF-I overlies a thick package of historical alluvium, NF-II directly overlies the Modesto Formation with local areas of historical alluvium, and Reach NF-III directly overlies a continuous blanket of sediment derived from historical crevasse splay (Rcs), overflow channel (Rofc), alluvium (Ra), channel (Rch) and overbank (Rob) deposits, above the Modesto Formation. The preliminary conceptual surface and subsurface geologic relationships as they relate to levee structures and potential underseepage along each reach of the river are described below. This study does not account for any existing seepage mitigation structures (i.e. cutoff walls) that may be present.

Along Reach NF-I the levees are underlain by a package of young coarse-grained fluvial sediment, most likely of mining debris origin, and chutes filled with coarse grained fining upward sequences of sediment also derived from hydraulic mining debris (Figure 3). This material is laterally extensive and poorly consolidated, with localized chute deposits (map unit Rcu). The chutes extend beneath the levee with an orientation that is roughly orthogonal to the levee crest, and may provide relatively high conductivity pathways for levee underseepage within the already very permeable fluvial sediments. The sediments along the northern half of reach NF-I were dredged for gold during the first half of the 20th century, well-graded dredge tailings remain in these areas. Dredge tailings are unconsolidated and consist of silt, sand, and gravel. At the north near Vance Avenue the project levees appear to overlie these highly permeable tailings, and everywhere else bound the western edge of the tailing spoils. Levees along this entire reach are judged to be highly susceptible to underseepage.

Levee reach NF-II is likely underlain by a combination of coarse grained, semi-consolidated alluvium of the Modesto Formation and localized areas of historical, poorly consolidated coarse-grained avulsion deposits (overbank and crevasse splay deposits) overlying the Modesto Formation. These avulsion deposits likely are permeable and may provide localized areas susceptible to underseepage. Project levees underlain by the Modesto Formation likely are less susceptible to underseepage problems, however the natural variability within the Modesto may also provide local pathways for underseepage.

Levee reach NF-III generally consists of westward aggrading avulsion deposits overlying the Modesto Formation. The levee is underlain by coarse-grained, poorly consolidated silt, sand and gravel, blanketing the consolidated Modesto Formation and in some places incised into the Modesto Formation. These deposits likely are permeable and susceptible to underseepage.

In summary, lateral and vertical variabilities in the shallow subsurface deposits have resulted from past fluvial geomorphic processes. Surficial geologic mapping along the north Feather River allows reach classifications within which conditions may be relatively similar. The conceptual geomorphic framework suggests that stratigraphic relationships may promote localized levee underseepage, given certain hydraulic conditions throughout the Northern Feather River Study area, particularly along reach NF-I. Areas where levees may overlie historical or Holocene-age coarse grained deposits are of special concern. Further spatial analyses of the surficial geologic mapping and subsurface geotechnical exploration data are needed to better constrain and characterize areas that are most susceptible to underseepage in the study area. We anticipate that this conceptual model will be revised and updated as new information becomes available.

7.0 Limitations

This geomorphic assessment and associated data interpretation have been performed in accordance with the standard of care commonly used as the state-of-practice in the geologic engineering profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this geographic area performing the same services under similar circumstances during the same time period.

Discussions of surface and subsurface conditions summarized in this technical memorandum are based on geologic interpretations of subsurface soil data at limited exploration locations available to this assessment through September of 2007. Variations in subsurface conditions may exist between exploration locations, and the project team may not be able to identify all adverse conditions in the levee and its foundation. This memorandum is for the use and benefit of DWR. Use by any other party is at their own discretion and risk.

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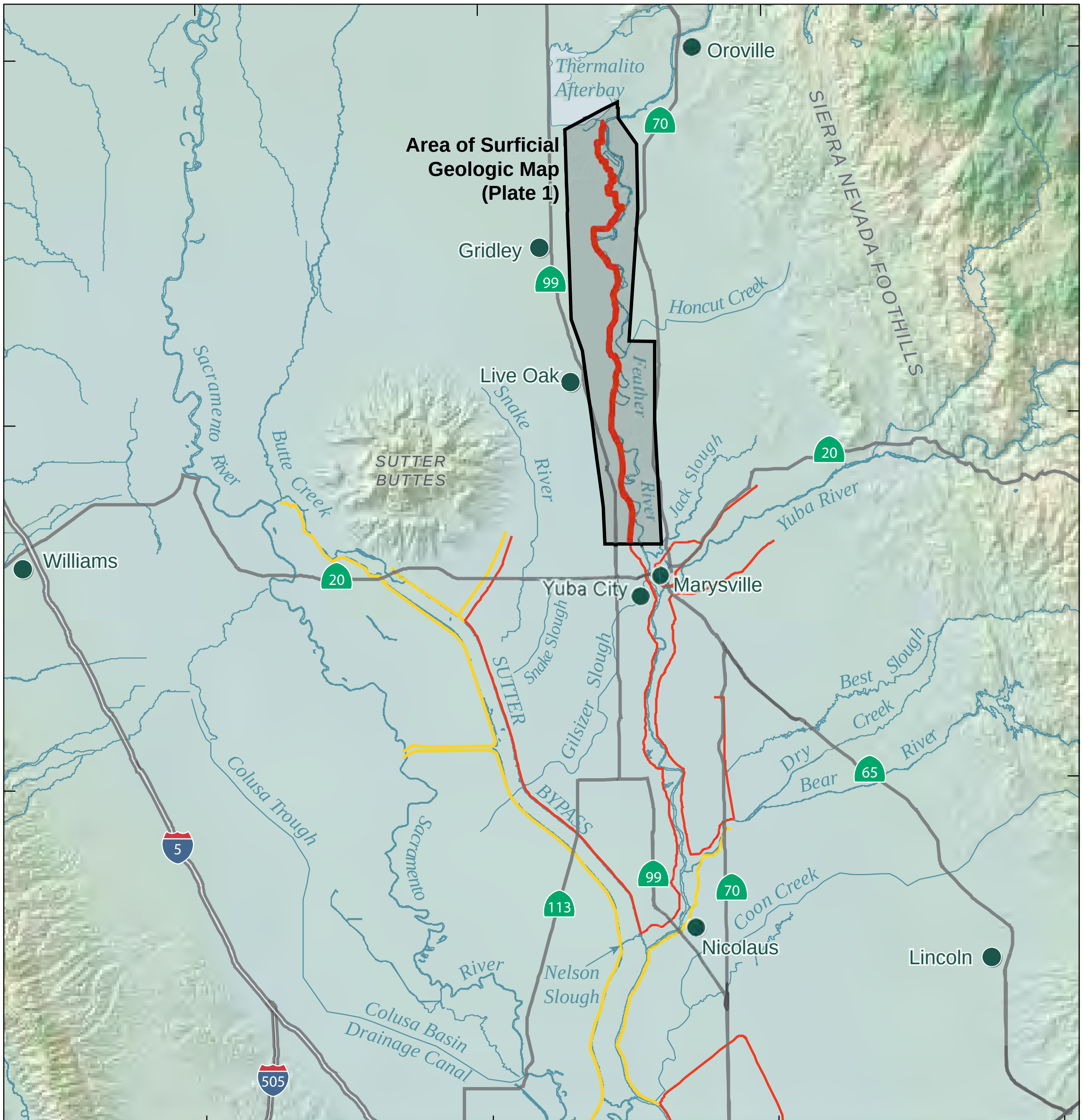
USGS, Oroville topographic quadrangle published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.

USGS, Palermo topographic quadrangle published 1952, remapped 1970; map scale 1:24,000, five foot contour interval.

USGS, Sutter topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.

USGS, Yuba City topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.



Sources: NAIP, 2006

Projection: UTM Zone 10, NAD83

Explanation

Northern Feather River project levee

Urban project levee

Other levee

N

0 5 10 mi

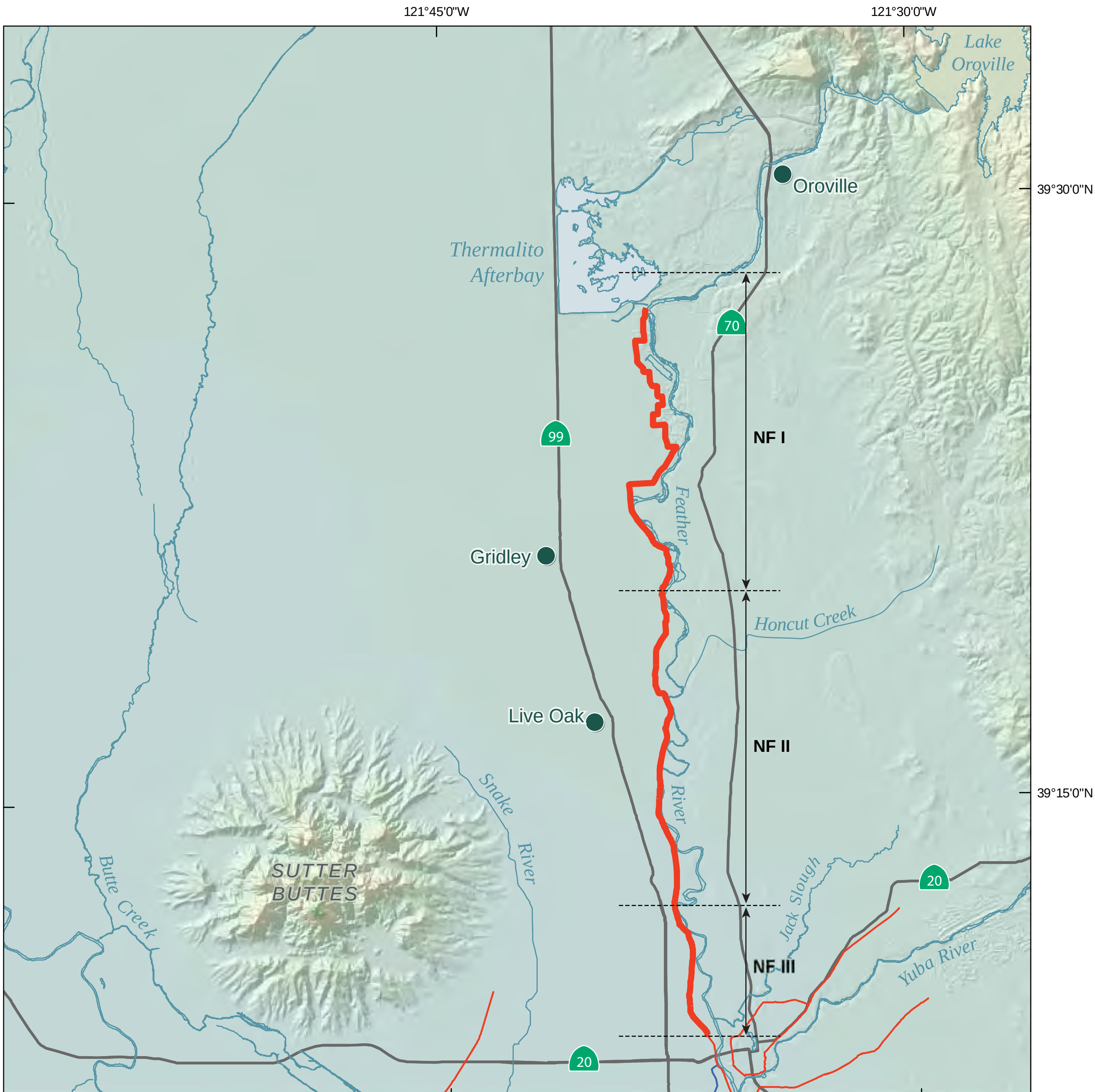
0 5 10 km

WLA

Map of Central Valley near Sutter Buttes, California

DWR URBAN LEVEE PROJECT

Figure 1



Sources: NAIP, 2006

Projection: UTM Zone 10, NAD83

Explanation

Northern Feather River project levee

Urban project levee

NF I — Reach of Northern Feather River Study Area

N

0 5 mi

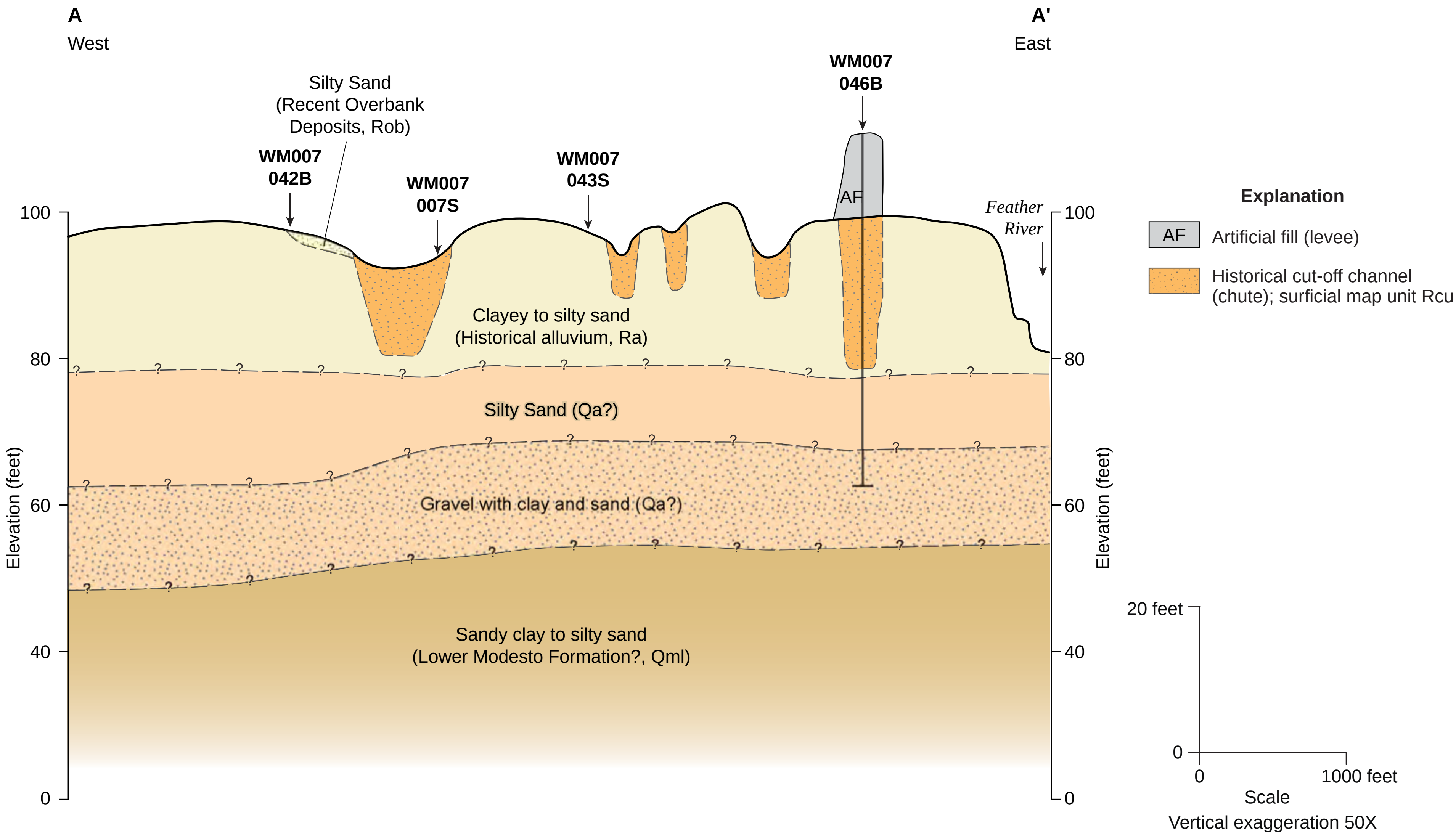
0 5 km

WLA

Map of the Reaches Identified along the Northern Feather River Study Area

DWR URBAN LEVEE PROJECT

Figure 2



WLA

Conceptual Subsurface Diagram across the Northern Feather River, A-A'

DWR URBAN LEVEE PROJECT

Figure 3

1881 North Feather River

Modified 09.03.09

Plate 1 - Surficial Geologic Map of the Feather River, Northern Section
This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on modern U.S. Geological Survey topographic base map prepared in 1952 and photo revised in 1973. See accompanying report for complete descriptions of map units, process descriptions and methodology.
Explanation
Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain; solid contacts within 30' of line shown on map.
Geologic unit destroyed by Dredge mining operations, contacts present in 1937 air photos and historical 1909-1910 series topographic maps.
NF-II
Geomorphic Reaches discussed in text for Northern Feather River study Area.
Project Levees
Canals as mapped from 1937 photos.
A
A'
Cross Section
WM0007_004S
Borehole locations from URS, Corp; locations from Sutter P2 Boring Locations, Region 1 Northern Area map.
W 1911
Water bodies shown on 1911 historic topographic maps.
W 1937
Water bodies, circa 1937.
BP
Borrow pits present in 1937.
Geologic Units
HISTORICAL
L
Levees (artificial fill), circa 1937.
DT
Dredge tailings; spoils material from gold dredge operations. Map unit boundary from USGS Biggs 7.5-minute quadrangle, 1970.
Rob
Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
Rcs
Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
Rch
Channel deposits; well sorted sands and fine gravels (Rch 1911: channels as shown on historic topo).
Rb
Channel bar deposits; fine gravel, sand, and silt deposited in or along channel lateral margins.
Rcu
Cut off channel (chute); occurs on insides of meander bends within the river channel; on flood plain – entrenchment of overflow channels into hydraulic mining debris.
Rofc
Overflow channels; vertically stratified sand, silt, and clay in floodplain channels occupied primarily when high-stage water overtops channel banks.
Rsl
Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Ra
Alluvial deposits, undifferentiated; sand, silt, and minor lenses of fine gravel.
HOLOCENE
Hob
Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
Ha
Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
Hch
Channel deposits; sorted sands and silts; fining upward.
Hms
Channel meander scroll deposits; sand, silt and clay from lateral channel migration.
Hsl
Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposit.
Qa
Alluvial deposits, undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
PLEISTOCENE
Qmu
Modesto Formation; upper member; unconsolidated gravel, sand, silt, and clay.
Qml
Modesto Formation; lower member; unconsolidated to semi-consolidated gravel, sand, silt and clay.
Qru
Riverbank Formation; upper member, semi-consolidated to consolidated gravel, sand, silt and minor clay.
Qrl
Riverbank Formation; lower member; consolidated gravel, sand, silt, and clay, generally associated with strong duripan horizon.
Stratigraphic Correlation Chart
Time
Depositional Environment
Epoch
Channel deposits
Floodplain and alluvial-fan deposits
Cultural deposits
Historical
Rch
Rb
Rcu
Rsl
Rofc
Qa
Ra
Rcs
Rob
L
DT
Holocene
Hch
Hms
Hsl
Ha
Hob
Latest Pleistocene
Pleistocene
Qmu
Qml
Qru
Qrl
1:20,000
N
0
0.5
1 Miles
0
0.5
1 Kilometers
Map projection: NAD83 UTM Zone 10N
Topo base USGS quadrangles:
USGS, Biggs topographic quadrangle, published 1952, remapped 1970; map scale 1:24,000, five foot contour interval.
USGS, Gridley topographic quadrangle, published 1952, remapped 1973; map scale 1:24,000, five foot contour interval.
USGS, Honcut topographic quadrangle, published 1952, remapped 1973; map scale 1:24,000, five foot contour interval.
USGS, Palermo topographic quadrangle, published 1952, remapped 1970; map scale 1:24,000, five foot contour interval.
USGS, Sutter topographic quadrangle published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
USGS, Yuba City topographic quadrangle, published 1952, photo revised 1973; map scale 1:24,000, five foot contour interval.
WLA WORK ORDERS 3 & 7 - SUTTER COUNTY
DWR URBAN LEVEE PROJECT
Plate 1 - Surficial Geologic Map
of the Feather River, Northern Section
WLA
WILLIAM LETTIS & ASSOCIATES, INC.
Walnut Creek, CA
1881_URS_Feather_North.mxd
09/04/2009
Geologic Mapping by J. Pearce and A. Streig
Digital Cartography by M. Ticci
THERMALITO BAY
LARKIN ROAD
ALMOND AVE.
NF-I
ORD RANCH RD
REIMER RD
LIVE OAK
NF-II
SANDERS RD
NF-III
MARYSVILLE
HONCUT

ENCLOSURE E

SEMI-PROBABALISTIC LEVEE STAGE-PERFORMANCE FUNCTIONS

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Wadsworth Canal
Basin and Reach: East Levee

Levee Mile: 0.83
Coordinates: 2170954.86 N; 6629916.3 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)									
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WSEWWC_010C	7					8					CL	0.0071	SM	2.8	394				
WSEWWC_001B	8.5					7					CL	0.0071	SM	2.8	394				
WSEWWC_009C	7					7					Cl	0.0071	SM	2.8	394				
WSEWWC_008C	6					5					CL	0.0071	SM	2.8	394				
WSEWWC_007C	3					9					CL	0.0071	SM	2.8	394				
WSEWWC_006C	8	7	2	13	29	2	7	3	14	43	CL	0.0071	SM	2.8	394	493	455	204287	92
WSEWWC_001A	10.5					11					CL, CH	0.0071	SM/SP-SM	11.14	1569				
WSEWWC_029B	4					7					CL	0.28	SP-SM	2.8	10				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WSEWWC_010C	CL	7	0.0071				7	SM	8	2.8							2.8
WSEWWC_001B	CL	8.5	0.0071				8.5	SM	7	2.8							2.8
WSEWWC_009C	Cl	7	0.0071				7	SM	7	2.8							2.8
WSEWWC_008C	CL	6	0.0071				6	SM	5	2.8							2.8
WSEWWC_007C	CL	3	0.0071				3	SM	9	2.8							2.8
WSEWWC_006C	CL	8	0.0071				8	SM	2	2.8							2.8
WSEWWC_001A	CL, CH	10.5	0.0071				10.5	SM	2.5	1.4	SP-SM	8.5	14				11.14
WSEWWC_029B	CL	4	0.28				4	SP-SM	7	2.8							2.8

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Wadsworth Canal
River Section: East Levee

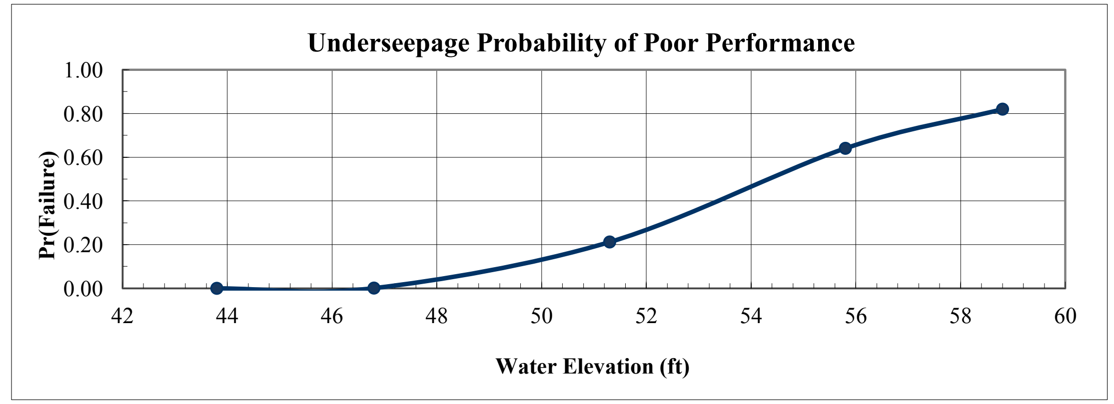
Levee Mile: 0.83
River Mile: 2170954.86 N; 6629916.3 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	493	455	92
Blanket Thickness (z)	7	2	29
Aquifer Thickness (d)	7	3	43

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	10	101	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	43.80	0.0000
Toe+3ft	3.00	46.80	0.0011
Half Height	7.50	51.30	0.2121
Crest-3ft	12.00	55.80	0.6407
Crest	15.00	58.80	0.8199

Crest	Rh
Head = 15.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	493	7.00	7.00	9.99	155.43	0.0263	8.75	1.25		
2	948	7.00	7.00	9.99	215.53	0.0214	9.90	1.41	0.164025	60.43
3	38	7.00	7.00	9.82	43.15	0.0455	4.20	0.60		
4	493	9.00	7.00	9.99	176.24	0.0244	9.20	1.02	0.093025	34.27
5	493	5.00	7.00	9.98	131.36	0.0289	8.13	1.63		
6	493	7.00	10.00	9.99	185.77	0.0337	9.39	1.34	0.014400	5.30
7	493	7.00	4.00	9.98	117.49	0.0175	7.71	1.10		
								Total	0.271450	100.00

E[I] = 1.250000
Var[I]= 0.271450
σ[I]= 0.521009
V(I) = 0.416807

E[ln I] = 0.143051
σ [ln I] = 0.400231

Ic= 0.80
ln(I crit) = -0.223144

β =	0.357421
F(z) =	0.180107
Pr(f) % =	81.989307

Crest-3ft	Rh
Head = 12.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	493	7.00	7.00	9.99	155.43	0.0263	7.00	1.00		
2	948	7.00	7.00	9.99	215.53	0.0214	7.92	1.13	0.105625	61.32
3	38	7.00	7.00	9.82	43.15	0.0455	3.36	0.48		
4	493	9.00	7.00	9.99	176.24	0.0244	7.36	0.82	0.057600	33.44
5	493	5.00	7.00	9.98	131.36	0.0289	6.50	1.30		
6	493	7.00	10.00	9.99	185.77	0.0337	7.51	1.07	0.009025	5.24
7	493	7.00	4.00	9.98	117.49	0.0175	6.17	0.88		
								Total	0.172250	100.00

E[I] = 1.000000
Var[I]= 0.172250
σ[I]= 0.415030
V(I) = 0.415030

E[ln I] = -0.079462
σ [ln I] = 0.398654

Ic= 0.80
ln(I crit) = -0.223144

β =	-0.199327
F(z) =	0.359268
Pr(f) % =	64.073178

Half Height	Rh
Head = 7.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	493	7.00	7.00	9.99	155.43	0.0263	4.38	0.63		
2	948	7.00	7.00	9.99	215.53	0.0214	4.95	0.71	0.042025	61.69
3	38	7.00	7.00	9.82	43.15	0.0455	2.10	0.30		
4	493	9.00	7.00	9.99	176.24	0.0244	4.60	0.51	0.022500	33.03
5	493	5.00	7.00	9.98	131.36	0.0289	4.07	0.81		
6	493	7.00	10.00	9.99	185.77	0.0337	4.69	0.67	0.003600	5.28
7	493	7.00	4.00	9.98	117.49	0.0175	3.86	0.55		
								Total	0.068125	100.00

E[I] = 0.630000
Var[I]= 0.068125
σ[I]= 0.261008
V(I) = 0.414298

E[ln I] = -0.541239
σ [ln I] = 0.398004

Ic= 0.80
ln(I crit) = -0.223144

β =	-1.359885
F(z) =	0.787921
Pr(f) % =	21.207929

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	493	7.00	7.00	9.99	155.43	0.0263	1.75	0.25		
2	948	7.00	7.00	9.99	215.53	0.0214	1.98	0.28	0.006400	56.89
3	38	7.00	7.00	9.82	43.15	0.0455	0.84	0.12		
4	493	9.00	7.00	9.99	176.24	0.0244	1.84	0.20	0.004225	37.56
5	493	5.00	7.00	9.98	131.36	0.0289	1.63	0.33		
6	493	7.00	10.00	9.99	185.77	0.0337	1.88	0.27	0.000625	5.56
7	493	7.00	4.00	9.98	117.49	0.0175	1.54	0.22		
								Total	0.011250	100.00

E[I] = 0.250000
Var[I]= 0.011250
σ[I]= 0.106066
V(I) = 0.424264

E[ln I] = -1.469052
σ [ln I] = 0.406835

Ic= 0.80
ln(I crit) = -0.223144

β =	-3.610930
F(z) =	0.998902
Pr(f) % =	0.109769

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Wadsworth Canal
River Section: East Levee

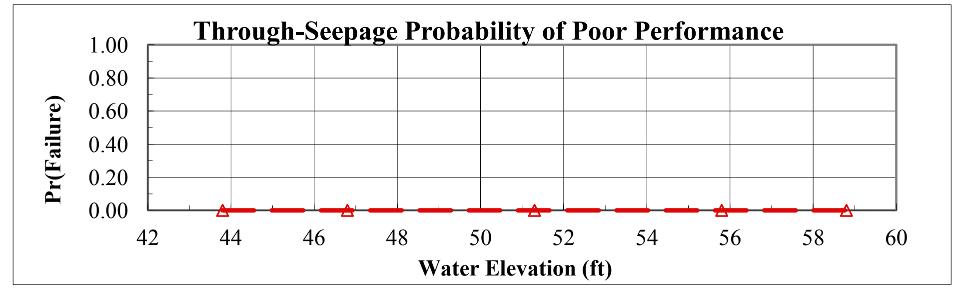
Levee Mile: 0.83
River Mile: 2170954.86 N; 6629916.3 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	43.80	0.0000
Toe+3ft	3.00	46.80	0.000000
Half Height	7.50	51.30	0.000000
Crest-3ft	12.00	55.80	0.000000
Crest	15.00	58.80	0.000000

Crest | **Head =** 15.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 12.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Half Height | **Head =** 7.50 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

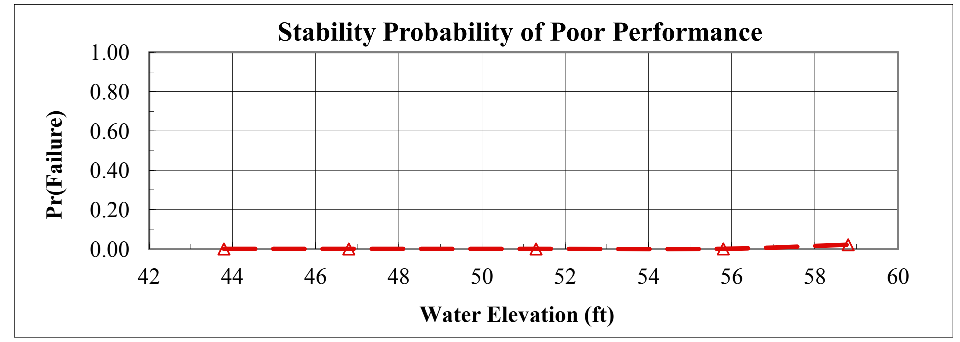
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal
River Section: East Levee

Levee Mile: 0.83
River Mile: 2170954.86 N; 6629916.3 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee ϕ'	31	4	12.00
Levee γ	125	6	5.00
Foundation c'	150	50	33.00
Foundation γ	115	6	5.00
Foundation ϕ'	28	3	12.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	43.80	0.0000
Toe+3ft	3.00	46.80	0.000000
Half Height	7.50	51.30	0.000000
Crest-3ft	12.00	55.80	0.000322
Crest	15.00	58.80	0.021343

Crest | **Head =** 15.00 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.50		
2	27	125	150	115	28	1.48	0.001225	1.44
3	35	125	150	115	28	1.55		
4	31	119	150	115	28	1.52	0.000169	0.20
5	31	131	150	115	28	1.50		
6	31	125	100	115	28	1.11	0.074529	87.51
7	31	125	200	115	28	1.66		
8	31	125	150	109	28	1.44	0.005402	6.34
9	31	125	150	121	28	1.58		
10	31	125	150	115	25	1.45	0.003844	4.51
11	31	125	150	115	31	1.57		

E[FS] = 1.504000   E[ln FS] = 0.389648   Total 0.085169   100.00
Var[FS]= 0.085169
σ[FS]= 0.291838   σ[ln FS]= 0.192251   β = 2.026769
V(FS) = 0.194041   F(z) = 0.021343
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 2.134304

Crest-3ft | **Head =** 12.00 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.80		
2	27	125	150	115	28	1.75	0.001936	2.11
3	35	125	150	115	28	1.84		
4	31	119	150	115	28	1.83	0.000812	0.88
5	31	131	150	115	28	1.77		
6	31	125	100	115	28	1.38	0.078400	85.41
7	31	125	200	115	28	1.94		
8	31	125	150	109	28	1.73	0.004556	4.96
9	31	125	150	121	28	1.86		
10	31	125	150	115	25	1.72	0.006084	6.63
11	31	125	150	115	31	1.87		

E[FS] = 1.796000   E[ln FS] = 0.571533   Total 0.091789   100.00
Var[FS]= 0.091789
σ[FS]= 0.302966   σ[ln FS]= 0.167507   β = 3.411982
V(FS) = 0.168689   F(z) = 0.000322
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.032246

Half Height | **Head =** 7.50 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

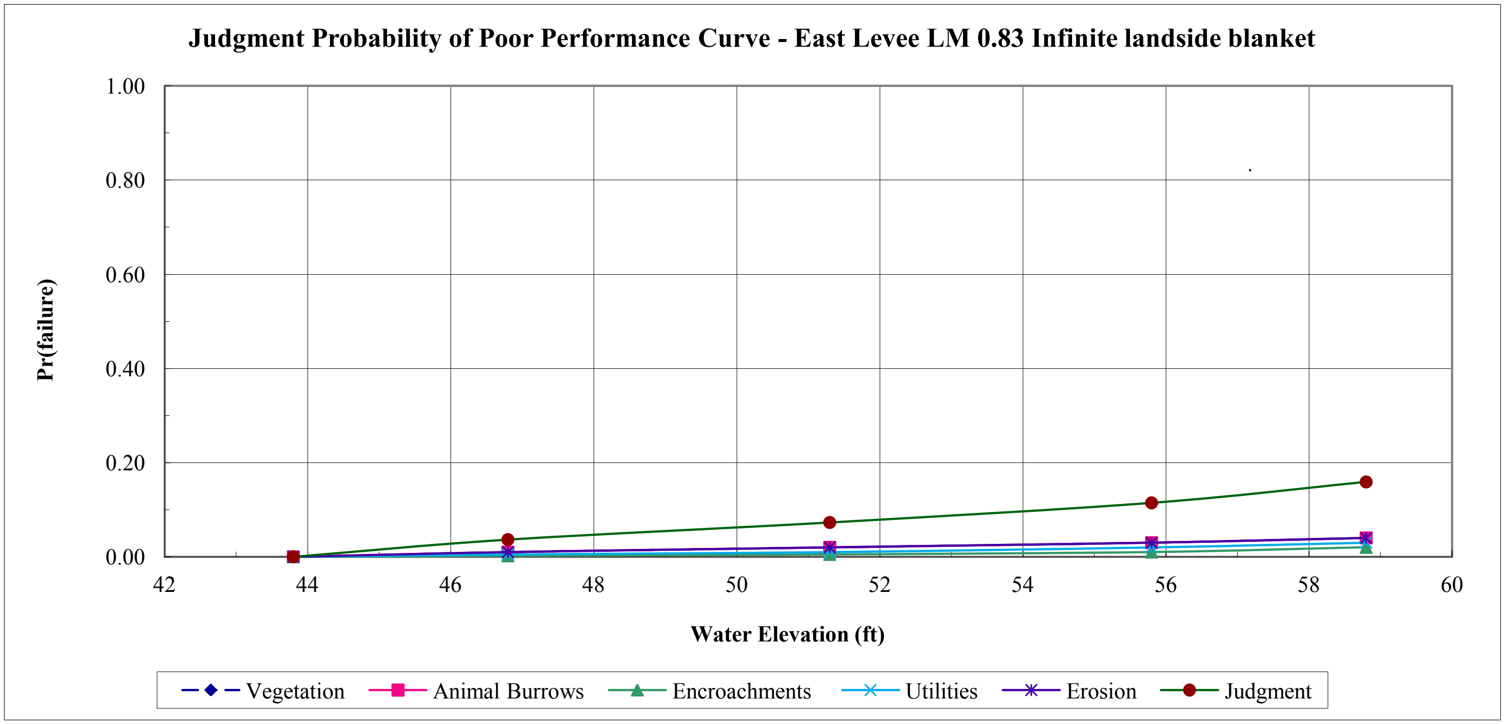
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal
River Section: East Levee

Levee Mile: 0.83
River Mile: 2170954.86 N
Analysis Case: Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/13/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
43.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
46.80	0.0100	0.9900	0.0100	0.9900	0.0020	0.9980	0.0050	0.9950	0.0100	0.9900	0.0365	0.9635
51.30	0.0200	0.9800	0.0200	0.9800	0.0050	0.9950	0.0100	0.9900	0.0200	0.9800	0.0729	0.9271
55.80	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0300	0.9700	0.1145	0.8855
58.80	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

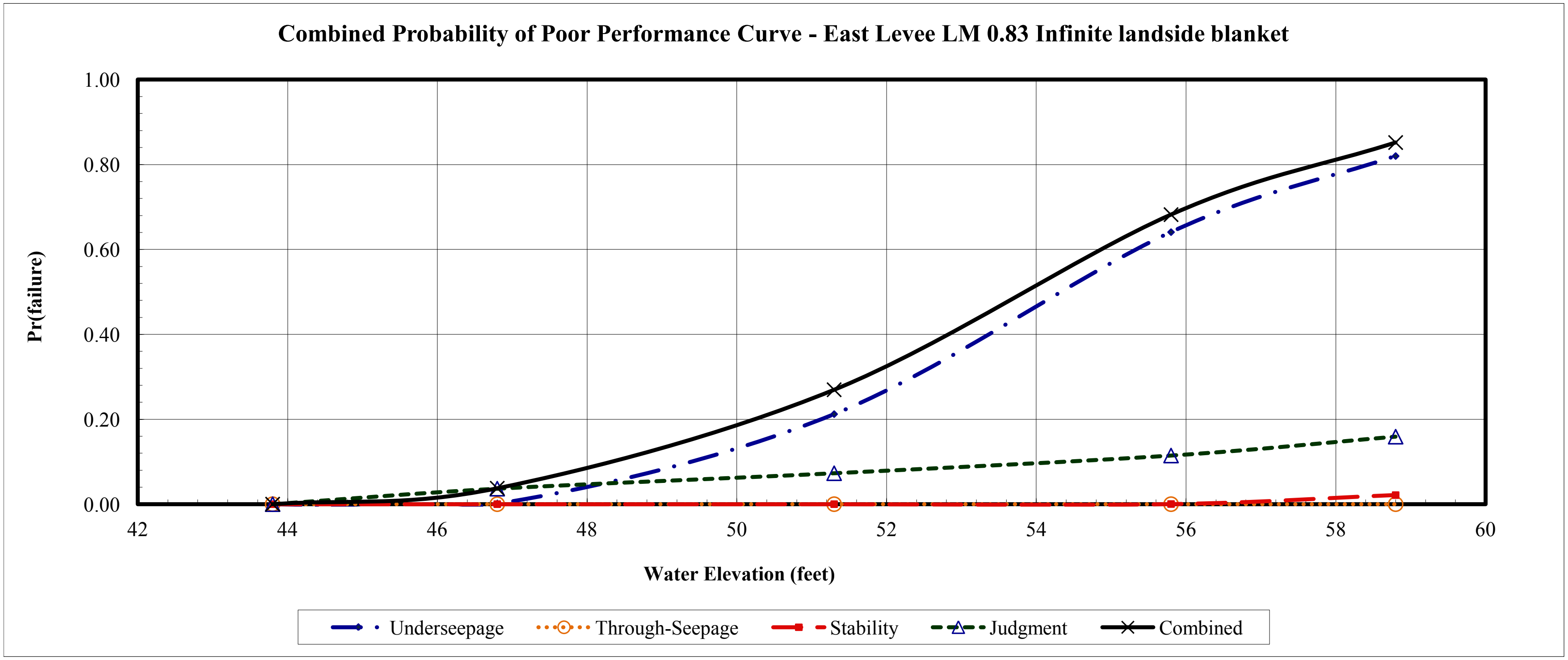
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal
River Section: East Levee

Levee Mile: 0.83
River Mile: 2170954.86 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 58.80
L/S Toe Elev.: 43.80
W/S Toe Elev.: 41.80

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/13/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
43.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
46.80	0.0011	0.9989	0.0000	1.0000	0.0000	1.0000	0.0365	0.9635	0.0375	0.9625
51.30	0.2121	0.7879	0.0000	1.0000	0.0000	1.0000	0.0729	0.9271	0.2695	0.7305
55.80	0.6407	0.3593	0.0000	1.0000	0.0003	0.9997	0.1145	0.8855	0.6820	0.3180
58.80	0.8199	0.1801	0.0000	1.0000	0.0213	0.9787	0.1590	0.8410	0.8518	0.1482



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Sutter Bypass
Basin and Reach: East Levee

Levee Mile: 6.20
Coordinates: 2158855 N; 6631970 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: 9/13/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WSESBP_011B	14					9					sCL/CH	0.007	SP-SM	10	1429				
WSESBP_015B	9					16.5					CLs,CL,ML	0.1	SM	3	30				
WSESBP_016B	10.5					9					SM/CL	0.007	SP-SM	10	1429				
WSESBP_017B	8	10	4	25	40	2	10	6	43	60	CL,SM	0.3	SP-SM	10	33	725	650	427548	90
WSESBP_018B	14					8					CL,SM/CL	0.007	SP-SM,ML,sML	3	429				
WSESBP_019B	4.8					17.5					CL/ML	0.01	SP-SM	10	1000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WSESBP_011B	sCL	6	0.007	CH	8	0.007	14	SP-SM	9	10							10
WSESBP_015B	CLs,CL,ML	9	0.1				9	SM	16.5	3							3
WSESBP_016B	SM	2	0.007	CL	8.5	0.007	10.5	SP-SM	9	10							10
WSESBP_017B	CL,SM	8	0.3				8	SP-SM	2	10							10
WSESBP_018B	CL,SM	2.5	0.007	CL	11.5	0.007	14	SP-SM,ML,sML	8	3							3
WSESBP_019B	CL	4.5	0.01	ML	3	0.1	4.8	SP-SM	17.5	10							10

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

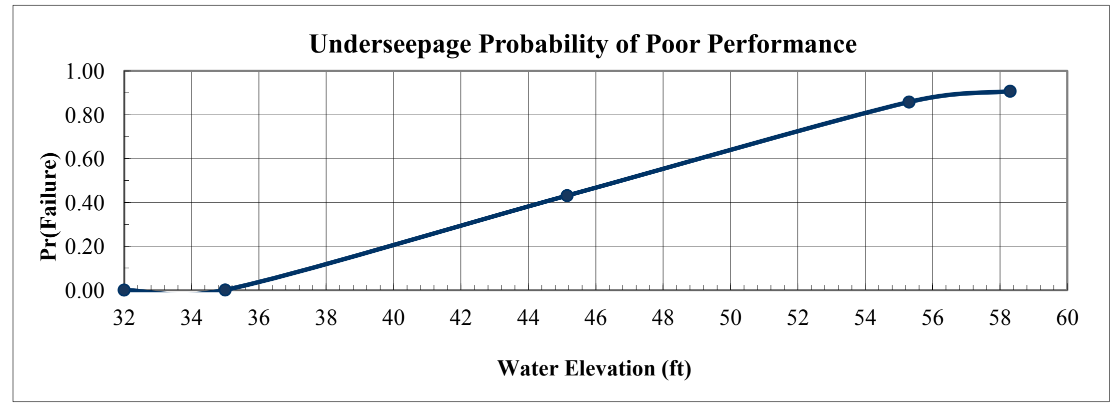
Levee Mile: 6.20
River Mile: 2158855 N; 6631970 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: 9/13/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	725	650	90
Blanket Thickness (z)	10	4	40
Aquifer Thickness (d)	10	6	60

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	15	150	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.00	0.0000
Toe+3ft	3.00	35.00	0.0004
Half Height	13.15	45.15	0.4311
Crest-3ft	23.30	55.30	0.8583
Crest	26.30	58.30	0.9076

Crest	Rh
Head = 26.30	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	16.31	1.63		
2	1375	10.00	10.00	14.99	370.81	0.0187	18.20	1.82	0.207025	33.43
3	75	10.00	10.00	14.85	86.60	0.0398	9.06	0.91		
4	725	14.00	10.00	14.99	318.59	0.0207	17.33	1.24	0.366025	59.11
5	725	6.00	10.00	14.97	208.57	0.0268	14.68	2.45		
6	725	10.00	16.00	14.99	340.59	0.0316	17.72	1.77	0.046225	7.46
7	725	10.00	4.00	14.96	170.29	0.0119	13.36	1.34		
								Total	0.619275	100.00

E[I] = 1.630000
Var[I]= 0.619275
σ[I]= 0.786940
V(I) = 0.482785

E[ln I] = 0.383822
σ [ln I] = 0.457730

β =	0.838533
F(z) =	0.092414
Pr(f) % =	90.758579

Ic= 0.80

ln(I crit) = -0.223144

Crest-3ft	Rh
Head = 23.30	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	14.45	1.45		
2	1375	10.00	10.00	14.99	370.81	0.0187	16.13	1.61	0.164025	33.59
3	75	10.00	10.00	14.85	86.60	0.0398	8.02	0.80		
4	725	14.00	10.00	14.99	318.59	0.0207	15.35	1.10	0.286225	58.62
5	725	6.00	10.00	14.97	208.57	0.0268	13.01	2.17		
6	725	10.00	16.00	14.99	340.59	0.0316	15.70	1.57	0.038025	7.79
7	725	10.00	4.00	14.96	170.29	0.0119	11.84	1.18		
								Total	0.488275	100.00

E[I] = 1.450000
Var[I]= 0.488275
σ[I]= 0.698767
V(I) = 0.481908

E[ln I] = 0.267149
σ [ln I] = 0.456979

β =	0.584597
F(z) =	0.141658
Pr(f) % =	85.834162

Ic= 0.80

ln(I crit) = -0.223144

Half Height	Rh
Head = 13.15	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	8.15	0.82		
2	1375	10.00	10.00	14.99	370.81	0.0187	9.10	0.91	0.052900	34.13
3	75	10.00	10.00	14.85	86.60	0.0398	4.53	0.45		
4	725	14.00	10.00	14.99	318.59	0.0207	8.66	0.62	0.090000	58.06
5	725	6.00	10.00	14.97	208.57	0.0268	7.34	1.22		
6	725	10.00	16.00	14.99	340.59	0.0316	8.86	0.89	0.012100	7.81
7	725	10.00	4.00	14.96	170.29	0.0119	6.68	0.67		
								Total	0.155000	100.00

E[I] = 0.820000
Var[I]= 0.155000
σ[I]= 0.393700
V(I) = 0.480122

E[ln I] = -0.302168
σ [ln I] = 0.455450

β =	-0.663450
F(z) =	0.568874
Pr(f) % =	43.112558

Ic= 0.80

ln(I crit) = -0.223144

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	1.86	0.19		
2	1375	10.00	10.00	14.99	370.81	0.0187	2.08	0.21	0.003025	35.38
3	75	10.00	10.00	14.85	86.60	0.0398	1.03	0.10		
4	725	14.00	10.00	14.99	318.59	0.0207	1.98	0.14	0.004900	57.31
5	725	6.00	10.00	14.97	208.57	0.0268	1.68	0.28		
6	725	10.00	16.00	14.99	340.59	0.0316	2.02	0.20	0.000625	7.31
7	725	10.00	4.00	14.96	170.29	0.0119	1.52	0.15		
								Total	0.008550	100.00

E[I] = 0.190000
Var[I]= 0.008550
σ[I]= 0.092466
V(I) = 0.486664

E[ln I] = -1.767012
σ [ln I] = 0.461044

β =	-3.832633
F(z) =	0.999594
Pr(f) % =	0.040605

Ic= 0.80

ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

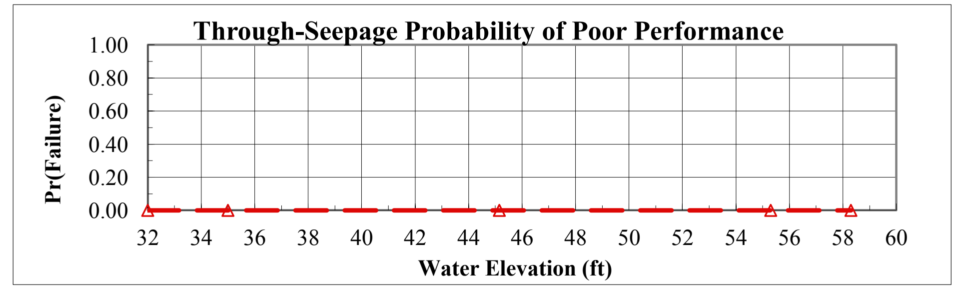
Levee Mile: 6.20
River Mile: 2158855 N; 6631970 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: 9/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.00	0.0000
Toe+3ft	3.00	35.00	0.000000
Half Height	13.15	45.15	0.000000
Crest-3ft	23.30	55.30	0.000000
Crest	26.30	58.30	0.000000

Crest | **Head =** 26.30 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 23.30 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height | **Head =** 13.15 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

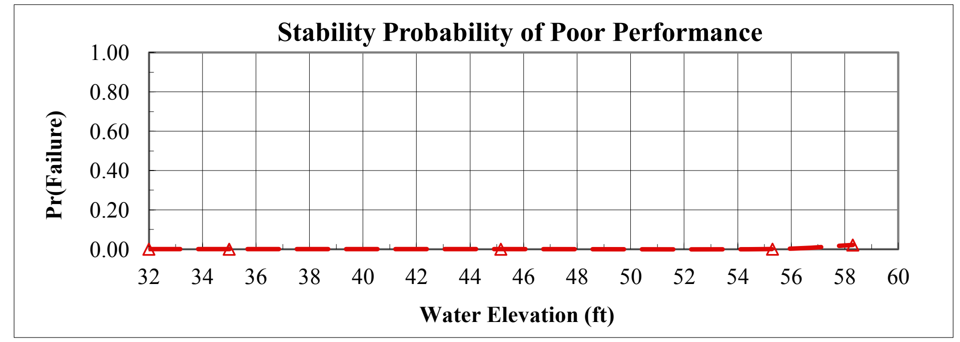
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 6.20
River Mile: 2158855 N; 6631970 E
Analysis Case Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: 9/13/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Levee ϕ'	31	4	12.00
Levee γ	125	6	5.00
Foundation c'	150	50	33.00
Foundation γ	115	6	5.00
Foundation ϕ'	28	3	12.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.00	0.0000
Toe+3ft	3.00	35.00	0.000000
Half Height	13.15	45.15	0.000000
Crest-3ft	23.30	55.30	0.000322
Crest	26.30	58.30	0.021343

Crest	Head =	26.30	Pr(f)=0	NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.50		
2	27	125	150	115	28	1.48	0.001225	1.44
3	35	125	150	115	28	1.55		
4	31	119	150	115	28	1.52	0.000169	0.20
5	31	131	150	115	28	1.50		
6	31	125	100	115	28	1.11	0.074529	87.51
7	31	125	200	115	28	1.66		
8	31	125	150	109	28	1.44	0.005402	6.34
9	31	125	150	121	28	1.58		
10	31	125	150	115	25	1.45	0.003844	4.51
11	31	125	150	115	31	1.57		
E[FS] =	1.504000			E[ln FS] =	0.389648	Total	0.085169	100.00
Var[FS]=	0.085169							
σ[FS]=	0.291838			σ[ln FS]=	0.192251		β =	2.026769
V(FS) =	0.194041						F(z) =	0.021343
FS req'd =	1.00			ln(FS req'd) =	0.000000		Pr(f) % =	2.134304

Crest-3ft	Head =	23.30	Pr(f)=0	NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.80		
2	27	125	150	115	28	1.75	0.001936	2.11
3	35	125	150	115	28	1.84		
4	31	119	150	115	28	1.83	0.000812	0.88
5	31	131	150	115	28	1.77		
6	31	125	100	115	28	1.38	0.078400	85.41
7	31	125	200	115	28	1.94		
8	31	125	150	109	28	1.73	0.004556	4.96
9	31	125	150	121	28	1.86		
10	31	125	150	115	25	1.72	0.006084	6.63
11	31	125	150	115	31	1.87		
E[FS] =	1.796000			E[ln FS] =	0.571533	Total	0.091789	100.00
Var[FS]=	0.091789							
σ[FS]=	0.302966			σ[ln FS]=	0.167507		β =	3.411982
V(FS) =	0.168689						F(z) =	0.000322
FS req'd =	1.00			ln(FS req'd) =	0.000000		Pr(f) % =	0.032246

Half Height	Head =	13.15	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			
E[FS] =				E[ln FS] =		Total		
Var[FS]=								
σ[FS]=				σ[ln FS]=			β =	
V(FS) =							F(z) =	
FS req'd =	1.00			ln(FS req'd) =	0.000000		Pr(f) % =	0.000000

Toe+3ft	Head =	3.00	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			
E[FS] =				E[ln FS] =		Total		
Var[FS]=								
σ[FS]=				σ[ln FS]=			β =	
V(FS) =							F(z) =	
FS req'd =	1.00			ln(FS req'd) =	0.000000		Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

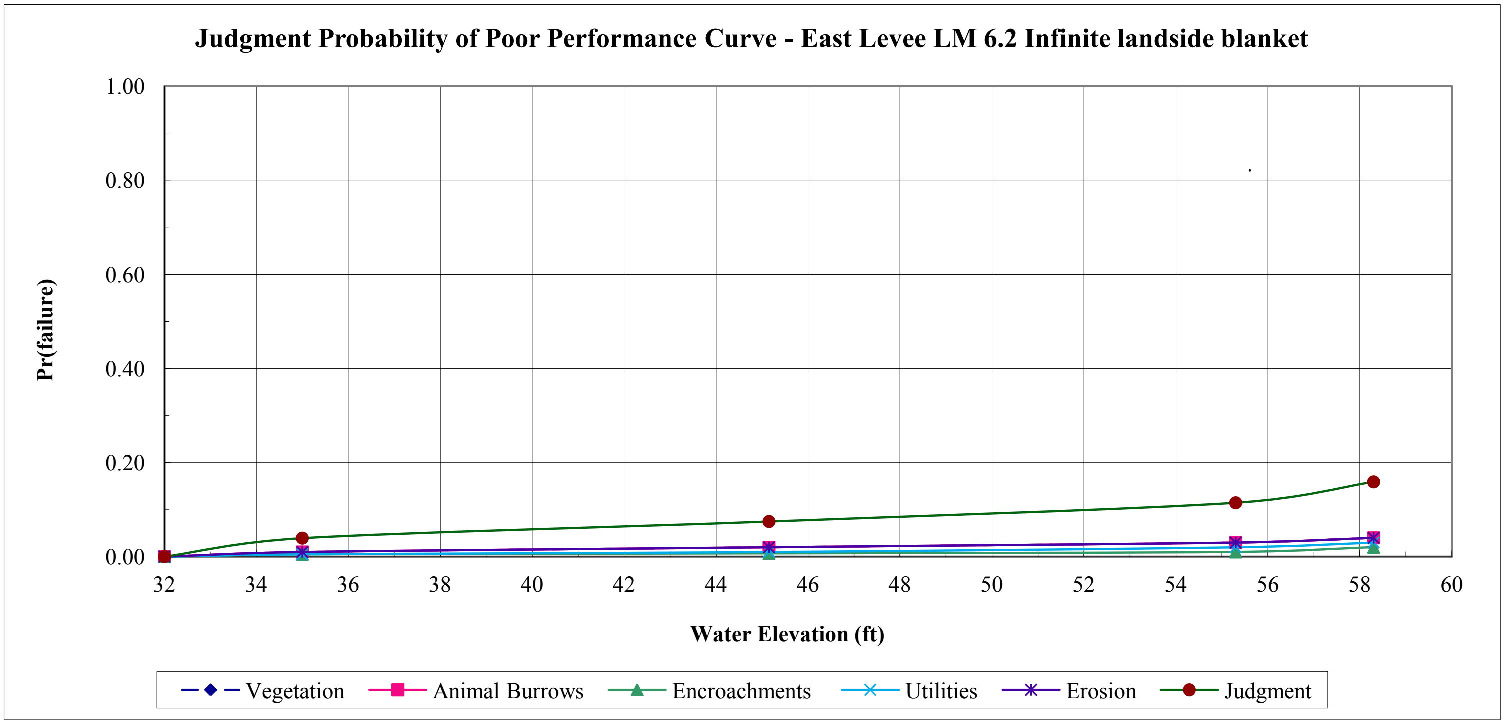
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 6.20
River Mile: 2158855 N; 6
Analysis Case: Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: 9/13/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
32.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
35.00	0.0100	0.9900	0.0100	0.9900	0.0050	0.9950	0.0050	0.9950	0.0100	0.9900	0.0394	0.9606
45.15	0.0200	0.9800	0.0200	0.9800	0.0070	0.9930	0.0100	0.9900	0.0200	0.9800	0.0747	0.9253
55.30	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0300	0.9700	0.1145	0.8855
58.30	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

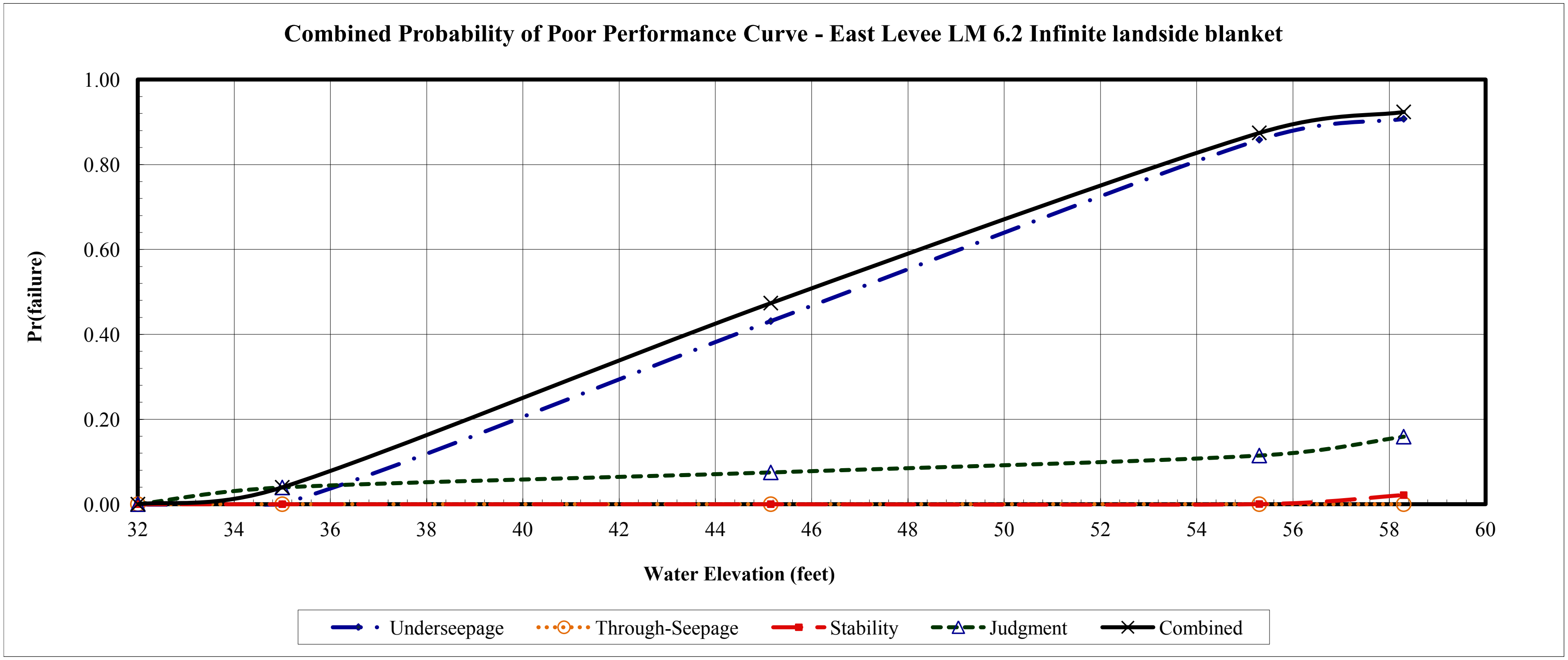
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 6.20
River Mile: 2158855 N; 663
Analysis Case: Infinite landside blanket

Crest Elev.: 58.30
L/S Toe Elev.: 32.00
W/S Toe Elev.: 32.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: 9/13/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
32.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
35.00	0.0004	0.9996	0.0000	1.0000	0.0000	1.0000	0.0394	0.9606	0.0398	0.9602
45.15	0.4311	0.5689	0.0000	1.0000	0.0000	1.0000	0.0747	0.9253	0.4736	0.5264
55.30	0.8583	0.1417	0.0000	1.0000	0.0003	0.9997	0.1145	0.8855	0.8746	0.1254
58.30	0.9076	0.0924	0.0000	1.0000	0.0213	0.9787	0.1590	0.8410	0.9239	0.0761



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Sutter Bypass
Basin and Reach: East Levee

Levee Mile: 17.30
Coordinates: 2113476.9763 N; 6655398.0817 E
Analysis Case Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 13 August 2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WSESBP_001B	4					4					CH,sML	0.14	SM	3	21				
WSESBP_002B	26.23					10.5					sCL,SC/ML	0.007	SM	3	429				
WSESBP_003B	11					2					sCL,CH	0.007	SM	3	429				
WSESBP_004B	27	15	10	111	67	31	13	13	164	98	sML, CL,SC	0.1	SM/SP-SM	8.4	84	1117	1398	1830429	98
WSESBP_005B	7.65					31.5					sCL/CH, sML	0.007	SP-SM/SW-SM	10	1429				
WSESBP_008B	23					7					sCL, CH	0.007	SP-SM/SW-SM	10	1429				
WSESBP_009B	8.25					3					CLs/ML	0.007	SP	28	4000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WSESBP_001B	CH,sML	4	0.14				4	SM	4	3							3
WSESBP_002B	sCL,SC	26	0.007	ML	4.5	0.14	26.23	SM	10.5	3							3
WSESBP_003B	sCL,CH	11	0.007				11	SM	2	3							3
WSESBP_004B	sML, CL,SC	27	0.1				27	SM	7.1	3	SP-SM	23.9	10				8.4
WSESBP_005B	sCL	7.25	0.007	CH, sML	5.75	0.1	7.65	SP-SM	13	10	SW-SM	18.5	10				10
WSESBP_008B	sCL, CH	23	0.007				23	SP-SM	5	10	SW-SM	2	10				10
WSESBP_009B	CLs	8	0.007	ML	5	0.14	8.25	SP	3	28							28

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

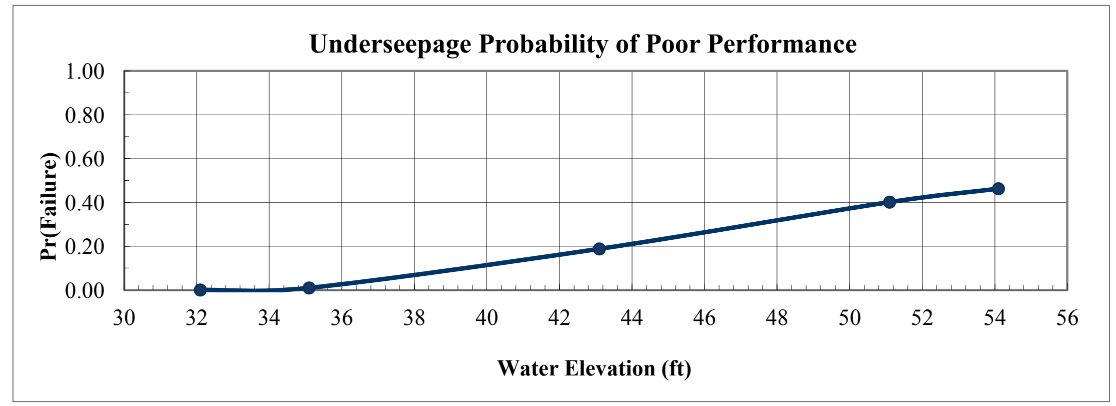
Levee Mile: 17.30
River Mile: 2113476.9763 N; 6655398.0817 E
Analysis Case Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 13 August 2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	1117	1095	98
Blanket Thickness (z)	15	10	67
Aquifer Thickness (d)	13	13	98

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	15	150	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.10	0.0000
Toe+3ft	3.00	35.10	0.0094
Half Height	11.00	43.10	0.1876
Crest-3ft	19.00	51.10	0.4011
Crest	22.00	54.10	0.4623

Crest	Rh
Head =	22.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1117	15.00	13.00	14.99	466.71	0.0206	16.25	1.08		
2	2212	15.00	13.00	15.00	656.71	0.0158	17.58	1.17	0.140625	10.48
3	22	15.00	13.00	14.75	66.00	0.0563	6.29	0.42		
4	1117	25.05	13.00	15.00	603.12	0.0169	17.27	0.69	1.060900	79.04
5	1117	4.95	13.00	14.98	268.10	0.0300	13.62	2.75		
6	1117	15.00	25.74	15.00	656.71	0.0313	17.58	1.17	0.140625	10.48
7	1117	15.00	0.26	14.75	66.00	0.0011	6.29	0.42		
								Total	1.342150	100.00

E[I] = 1.080000
Var[I]= 1.342150
σ[I]= 1.158512
V(I) = 1.072696

E[ln I] = -0.305930
σ [ln I] = 0.875090
ln(I crit) = -0.223144

Ic=	0.80

β =	-0.349599
F(z) =	0.537685
Pr(f) % =	46.231478

Crest-3ft	Rh
Head =	19.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1117	15.00	13.00	14.99	466.71	0.0206	14.04	0.94		
2	2212	15.00	13.00	15.00	656.71	0.0158	15.18	1.01	0.105625	10.53
3	22	15.00	13.00	14.75	66.00	0.0563	5.43	0.36		
4	1117	25.05	13.00	15.00	603.12	0.0169	14.92	0.60	0.792100	78.95
5	1117	4.95	13.00	14.98	268.10	0.0300	11.76	2.38		
6	1117	15.00	25.74	15.00	656.71	0.0313	15.18	1.01	0.105625	10.53
7	1117	15.00	0.26	14.75	66.00	0.0011	5.43	0.36		
								Total	1.003350	100.00

E[I] = 0.940000
Var[I]= 1.003350
σ[I]= 1.001674
V(I) = 1.065610

E[ln I] = -0.441232
σ [ln I] = 0.871041
ln(I crit) = -0.223144

Ic=	0.80

β =	-0.506557
F(z) =	0.598852
Pr(f) % =	40.114814

Half Height	Rh
Head =	11.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1117	15.00	13.00	14.99	466.71	0.0206	8.13	0.54		
2	2212	15.00	13.00	15.00	656.71	0.0158	8.79	0.59	0.036100	10.54
3	22	15.00	13.00	14.75	66.00	0.0563	3.15	0.21		
4	1117	25.05	13.00	15.00	603.12	0.0169	8.64	0.34	0.270400	78.93
5	1117	4.95	13.00	14.98	268.10	0.0300	6.81	1.38		
6	1117	15.00	25.74	15.00	656.71	0.0313	8.79	0.59	0.036100	10.54
7	1117	15.00	0.26	14.75	66.00	0.0011	3.15	0.21		
								Total	0.342600	100.00

E[I] = 0.540000
Var[I]= 0.342600
σ[I]= 0.585320
V(I) = 1.083927

E[ln I] = -1.004677
σ [ln I] = 0.881465
ln(I crit) = -0.223144

Ic=	0.80

β =	-1.139780
F(z) =	0.812361
Pr(f) % =	18.763921

Toe+3ft	Rh
Head =	3.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1117	15.00	13.00	14.99	466.71	0.0206	2.22	0.15		
2	2212	15.00	13.00	15.00	656.71	0.0158	2.40	0.16	0.002500	9.61
3	22	15.00	13.00	14.75	66.00	0.0563	0.86	0.06		
4	1117	25.05	13.00	15.00	603.12	0.0169	2.36	0.09	0.021025	80.79
5	1117	4.95	13.00	14.98	268.10	0.0300	1.86	0.38		
6	1117	15.00	25.74	15.00	656.71	0.0313	2.40	0.16	0.002500	9.61
7	1117	15.00	0.26	14.75	66.00	0.0011	0.86	0.06		
								Total	0.026025	100.00

E[I] = 0.150000
Var[I]= 0.026025
σ[I]= 0.161323
V(I) = 1.075484

E[ln I] = -2.281402
σ [ln I] = 0.876678
ln(I crit) = -0.223144

Ic=	0.80

β =	-2.602327
F(z) =	0.990558
Pr(f) % =	0.944249

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

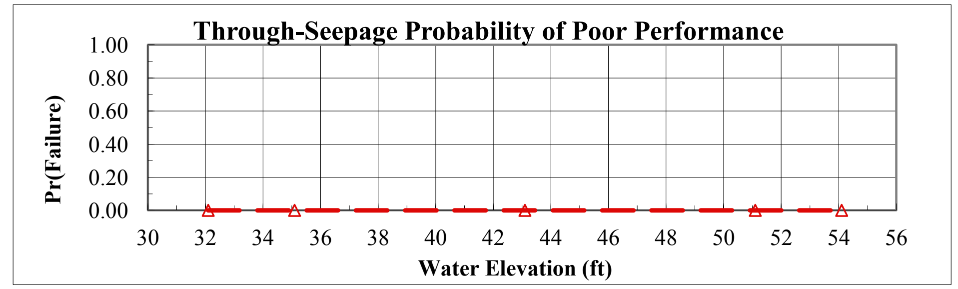
Levee Mile: 17.30
River Mile: 2113476.9763 N; 6655398.0817 E
Analysis Case Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 13 August 2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	15	1.5	10.00
Initial Porosity (n)	45	4.50	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.10	0.0000
Toe+3ft	3.00	35.10	0.000000
Half Height	11.00	43.10	0.000000
Crest-3ft	19.00	51.10	0.000000
Crest	22.00	54.10	0.000000

Crest | **Head =** 22.00 | **Horizontal Gradient (Ix) =** 0.297

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 19.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Half Height | **Head =** 11.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

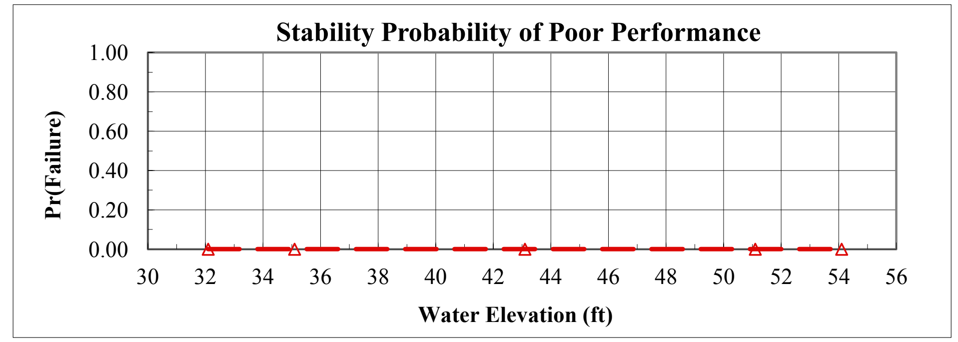
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 17.30
River Mile: 2113476.9763 N; 6655398.0817 E
Analysis Case Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 13 August 2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Levee ϕ'	31	4	12.00
Levee γ	125	6	5.00
Foundation c'	150	50	33.00
Foundation γ	115	6	5.00
Foundation ϕ'	28	3	12.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	32.10	0.0000
Toe+3ft	3.00	35.10	0.000000
Half Height	11.00	43.10	0.000000
Crest-3ft	19.00	51.10	0.000000
Crest	22.00	54.10	0.000076

Crest | **Head =** 22.00 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.44		
2	27	125	150	115	28	1.43	0.000020	0.11
3	35	125	150	115	28	1.44		
4	31	119	150	115	28	1.45	0.000090	0.48
5	31	131	150	115	28	1.43		
6	31	125	100	115	28	1.41	0.014520	77.49
7	31	125	200	115	28	1.65		
8	31	125	150	109	28	1.50	0.000012	0.07
9	31	125	150	121	28	1.51		
10	31	125	150	115	25	1.37	0.004096	21.86
11	31	125	150	115	31	1.50		

E[FS] = 1.439000 | E[ln FS] = 0.359444 | Total 0.018739 100.00
Var[FS]= 0.018739
σ[FS]= 0.136890 | σ[ln FS]= 0.094915 | β = 3.787021
V(FS) = 0.095129 | F(z) = 0.000076
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | Pr(f) % = 0.007623

Crest-3ft | **Head =** 19.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | Pr(f) % = 0.000000

Half Height | **Head =** 11.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | Pr(f) % = 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

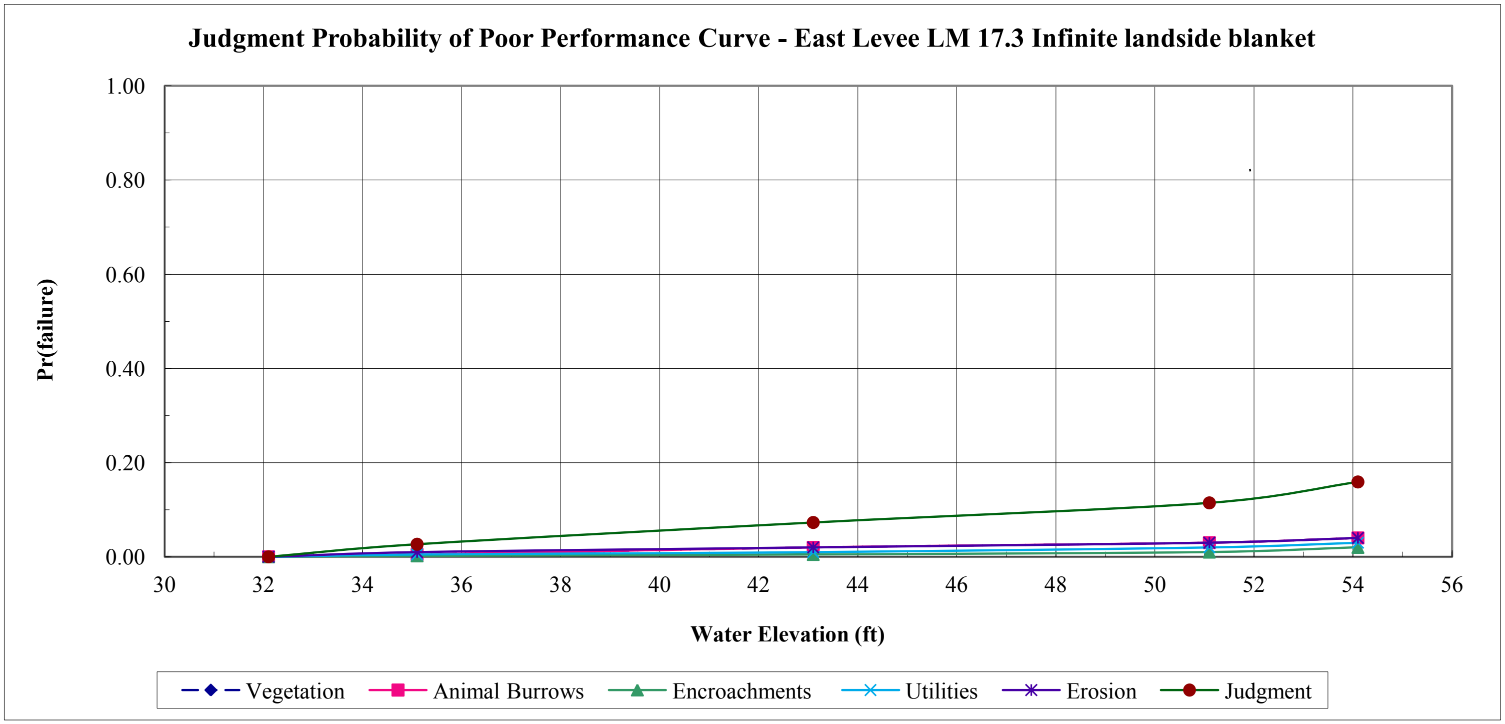
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 17.30
River Mile: 2113476.9763
Analysis Case: Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 13 A

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
32.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
35.10	0.0050	0.9950	0.0050	0.9950	0.0020	0.9980	0.0050	0.9950	0.0100	0.9900	0.0267	0.9733
43.10	0.0200	0.9800	0.0200	0.9800	0.0050	0.9950	0.0100	0.9900	0.0200	0.9800	0.0729	0.9271
51.10	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0300	0.9700	0.1145	0.8855
54.10	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

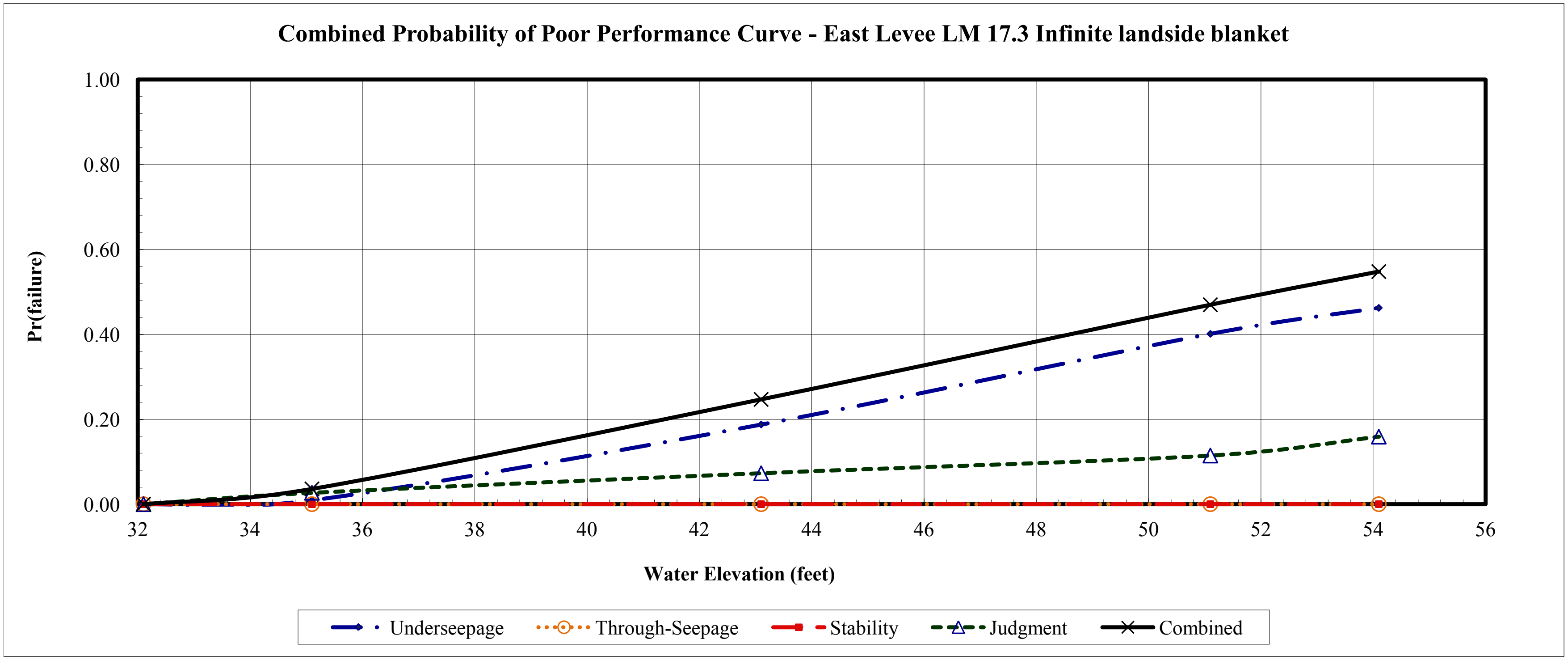
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: East Levee

Levee Mile: 17.30
River Mile: 2113476.9763 N
Analysis Case: Infinite landside blanket

Crest Elev.: 54.10
L/S Toe Elev.: 32.10
W/S Toe Elev.: 37.78

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 13 Augu

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
32.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
35.10	0.0094	0.9906	0.0000	1.0000	0.0000	1.0000	0.0267	0.9733	0.0359	0.9641
43.10	0.1876	0.8124	0.0000	1.0000	0.0000	1.0000	0.0729	0.9271	0.2468	0.7532
51.10	0.4011	0.5989	0.0000	1.0000	0.0000	1.0000	0.1145	0.8855	0.4697	0.5303
54.10	0.4623	0.5377	0.0000	1.0000	0.0001	0.9999	0.1590	0.8410	0.5478	0.4522



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River South
Basin and Reach: MA 3

Levee Mile: 4.92
Coordinates: 2106963.58 N; 6679261.24E
Analysis Case Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 09/12/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WM0003_012B	16	22	5	133	23	75	73	28	1849	38	SC-SM/CL,CLs	0.007	SM,GP-GM,SP.	10	1429	2172	1974	3422159	91
WM0003_016B	18.5					17.5					MLS	0.14	SM-SPg	1.12	8				
WM0003_004S	25					88					MLs	0.14	M, SPg,GW-GI	3	21				
WM0003_020B	22.6					78					CLs/sML,CL	0.007	SMSW-SM,GW	30	4286				
WM0003_022B	24					89.5					CLs	0.007	M, SPM-SM.SI	30	4286				
WL0001_020B	30					89.5					CLs,CL	0.01	SMg,SP-SC,GP	30	3000				
WL0001_025B	14																		
WL0001_031B	25.75																		

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WM0003_012B	SC-SM	1	0.007	CL,CLs	15	0.007	16	SM,GP-GM,SP.	75	10							10
WM0003_016B	MLS	18.5	0.14				18.5	SM-SPg	17.5	1.12							1.12
WM0003_004S	MLs	25	0.14				25	M, SPg,GW-GI	88	3							3
WM0003_020B	CLs	22	0.007	sML,CL	12	0.14	22.6	SMSW-SM,GW	78	30							30
WM0003_022B	CLs	24	0.007				24	M, SPM-SM.SI	89.5	30							30
WL0001_020B	CLs,CL	30	0.01				30	SMg,SP-SC,GP	89.5	30							30
WL0001_025B	sCL, SM, CL	14	0.01				14	SP-SC, SP-SM	4	3							3

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: MA 3

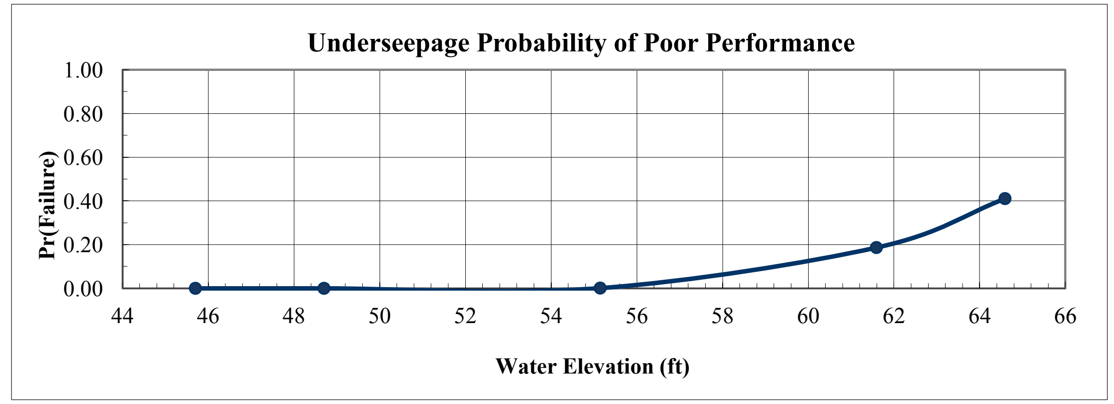
Levee Mile: 4.92
River Mile: 2106963.58 N; 6679261.24E
Analysis Case Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 09/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	2172	1977	91
Blanket Thickness (z)	22	5	23
Aquifer Thickness (d)	73	28	38

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	45.70	0.0000
Toe+3ft	3.00	48.70	0.0000
Half Height	9.45	55.15	0.0011
Crest-3ft	15.89	61.59	0.1867
Crest	18.89	64.59	0.4106

Crest	Rh
Head = 18.89	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2172	22.00	73.00	49.99	1867.68	0.0357	17.23	0.78		
2	4149	22.00	73.00	49.99	2581.19	0.0264	17.66	0.80	0.005625	14.75
3	195	22.00	73.00	49.87	560.30	0.0986	14.30	0.65		
4	2172	27.06	73.00	49.99	2071.36	0.0324	17.38	0.64	0.032400	84.98
5	2172	16.94	73.00	49.98	1638.88	0.0401	17.02	1.00		
6	2172	22.00	100.74	49.99	2194.03	0.0424	17.46	0.79	0.000100	0.26
7	2172	22.00	45.26	49.98	1470.61	0.0274	16.83	0.77		
								Total	0.038125	100.00

E[I] = 0.780000 E[ln I] = -0.278851
Var[I]= 0.038125
σ[I]= 0.195256 σ [ln I] = 0.246535
V(I) = 0.250329

Ic= 0.80 ln(I crit) = -0.223144

β =	-1.131083
F(z) =	0.589385
Pr(f) % =	41.061547

Crest-3ft	Rh
Head = 15.89	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2172	22.00	73.00	49.99	1867.68	0.0357	14.49	0.66		
2	4149	22.00	73.00	49.99	2581.19	0.0264	14.85	0.68	0.004225	14.84
3	195	22.00	73.00	49.87	560.30	0.0986	12.03	0.55		
4	2172	27.06	73.00	49.99	2071.36	0.0324	14.62	0.54	0.024025	84.37
5	2172	16.94	73.00	49.98	1638.88	0.0401	14.32	0.85		
6	2172	22.00	100.74	49.99	2194.03	0.0424	14.69	0.67	0.000225	0.79
7	2172	22.00	45.26	49.98	1470.61	0.0274	14.16	0.64		
								Total	0.028475	100.00

E[I] = 0.660000 E[ln I] = -0.447176
Var[I]= 0.028475
σ[I]= 0.168745 σ [ln I] = 0.251638
V(I) = 0.255675

Ic= 0.80 ln(I crit) = -0.223144

β =	-1.777061
F(z) =	0.813347
Pr(f) % =	18.665317

Half Height	Rh
Head = 9.45	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2172	22.00	73.00	49.99	1867.68	0.0357	8.61	0.39		
2	4149	22.00	73.00	49.99	2581.19	0.0264	8.83	0.40	0.001225	13.00
3	195	22.00	73.00	49.87	560.30	0.0986	7.15	0.33		
4	2172	27.06	73.00	49.99	2071.36	0.0324	8.69	0.32	0.008100	85.94
5	2172	16.94	73.00	49.98	1638.88	0.0401	8.51	0.50		
6	2172	22.00	100.74	49.99	2194.03	0.0424	8.73	0.40	0.000100	1.06
7	2172	22.00	45.26	49.98	1470.61	0.0274	8.42	0.38		
								Total	0.009425	100.00

E[I] = 0.390000 E[ln I] = -0.971669
Var[I]= 0.009425
σ[I]= 0.097082 σ [ln I] = 0.245197
V(I) = 0.248929

Ic= 0.80 ln(I crit) = -0.223144

β =	-3.962806
F(z) =	0.998866
Pr(f) % =	0.113378

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2172	22.00	73.00	49.99	1867.68	0.0357	2.74	0.12		
2	4149	22.00	73.00	49.99	2581.19	0.0264	2.80	0.13	0.000225	19.57
3	195	22.00	73.00	49.87	560.30	0.0986	2.27	0.10		
4	2172	27.06	73.00	49.99	2071.36	0.0324	2.76	0.10	0.000900	78.26
5	2172	16.94	73.00	49.98	1638.88	0.0401	2.70	0.16		
6	2172	22.00	100.74	49.99	2194.03	0.0424	2.77	0.13	0.000025	2.17
7	2172	22.00	45.26	49.98	1470.61	0.0274	2.67	0.12		
								Total	0.001150	100.00

E[I] = 0.120000 E[ln I] = -2.158680
Var[I]= 0.001150
σ[I]= 0.033912 σ [ln I] = 0.277187
V(I) = 0.282597

Ic= 0.80 ln(I crit) = -0.223144

β =	-7.787820
F(z) =	1.000000
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: MA 3

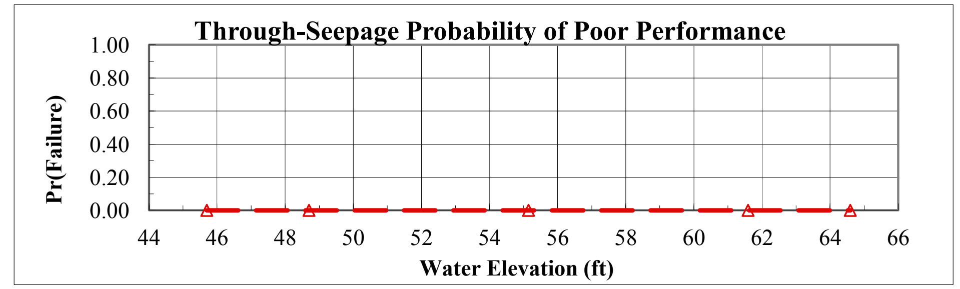
Levee Mile: 4.92
River Mile: 2106963.58 N; 6679261.24E
Analysis Case Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 09/12/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	45.70	0.0000
Toe+3ft	3.00	48.70	0.000000
Half Height	9.45	55.15	0.000000
Crest-3ft	15.89	61.59	0.000000
Crest	18.89	64.59	0.000000

Crest	Head =	18.89	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   Pr(f) % = 0.000000

Crest-3ft	Head =	15.89	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   Pr(f) % = 0.000000

Half Height	Head =	9.45	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   Pr(f) % = 0.000000

Toe+3ft	Head =	3.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

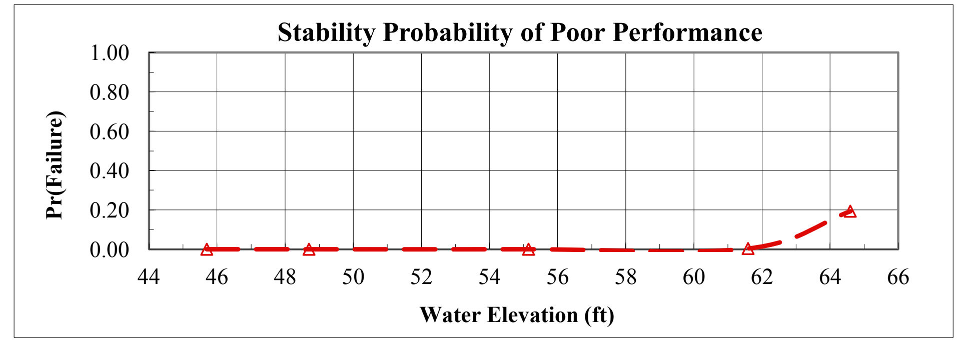
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: MA 3

Levee Mile: 4.92
River Mile: 2106963.58 N; 6679261.24E
Analysis Case Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 09/12/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee φ'	29	1	5.00
Levee γ	120	6	5.00
Foundation c'	150	17	11.50
Foundation γ	120	6	5.00
Foundation φ'	31	4	11.50



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	45.70	0.0000
Toe+3ft	3.00	48.70	0.000000
Half Height	9.45	55.15	0.000000
Crest-3ft	15.89	61.59	0.003757
Crest	18.89	64.59	0.193409

Crest | **Head =** 18.89 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.09		
2	28	120	150	120	31	1.20	0.010816	93.19
3	30	120	150	120	31	0.99		
4	29	114	150	120	31	1.07	0.000306	2.64
5	29	126	150	120	31	1.11		
6	29	120	133	120	31	1.09	0.000484	4.17
7	29	120	167	120	31	1.05		
8	29	120	150	114	31	1.09	0.000000	0.00
9	29	120	150	126	31	1.09		
10	29	120	150	120	27	1.09	0.000000	0.00
11	29	120	150	120	35	1.09		

E[FS] = 1.094000 | E[ln FS] = 0.085015 | Total 0.011606 100.00
Var[FS]= 0.011606
σ[FS]= 0.107732 | σ[ln FS]= 0.098238 | β = 0.865401
V(FS) = 0.098476 | F(z) = 0.193409
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 19.340941

Crest-3ft | **Head =** 15.89 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.25		
2	28	120	150	120	31	1.35	0.009604	90.64
3	30	120	150	120	31	1.16		
4	29	114	150	120	31	1.24	0.000144	1.36
5	29	126	150	120	31	1.26		
6	29	120	133	120	31	1.25	0.000784	7.40
7	29	120	167	120	31	1.19		
8	29	120	150	114	31	1.25	0.000064	0.60
9	29	120	150	126	31	1.23		
10	29	120	150	120	27	1.25	0.000000	0.00
11	29	120	150	120	35	1.25		

E[FS] = 1.250000 | E[ln FS] = 0.219764 | Total 0.010596 100.00
Var[FS]= 0.010596
σ[FS]= 0.102937 | σ[ln FS]= 0.082210 | β = 2.673193
V(FS) = 0.082349 | F(z) = 0.003757
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.375665

Half Height | **Head =** 9.45 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

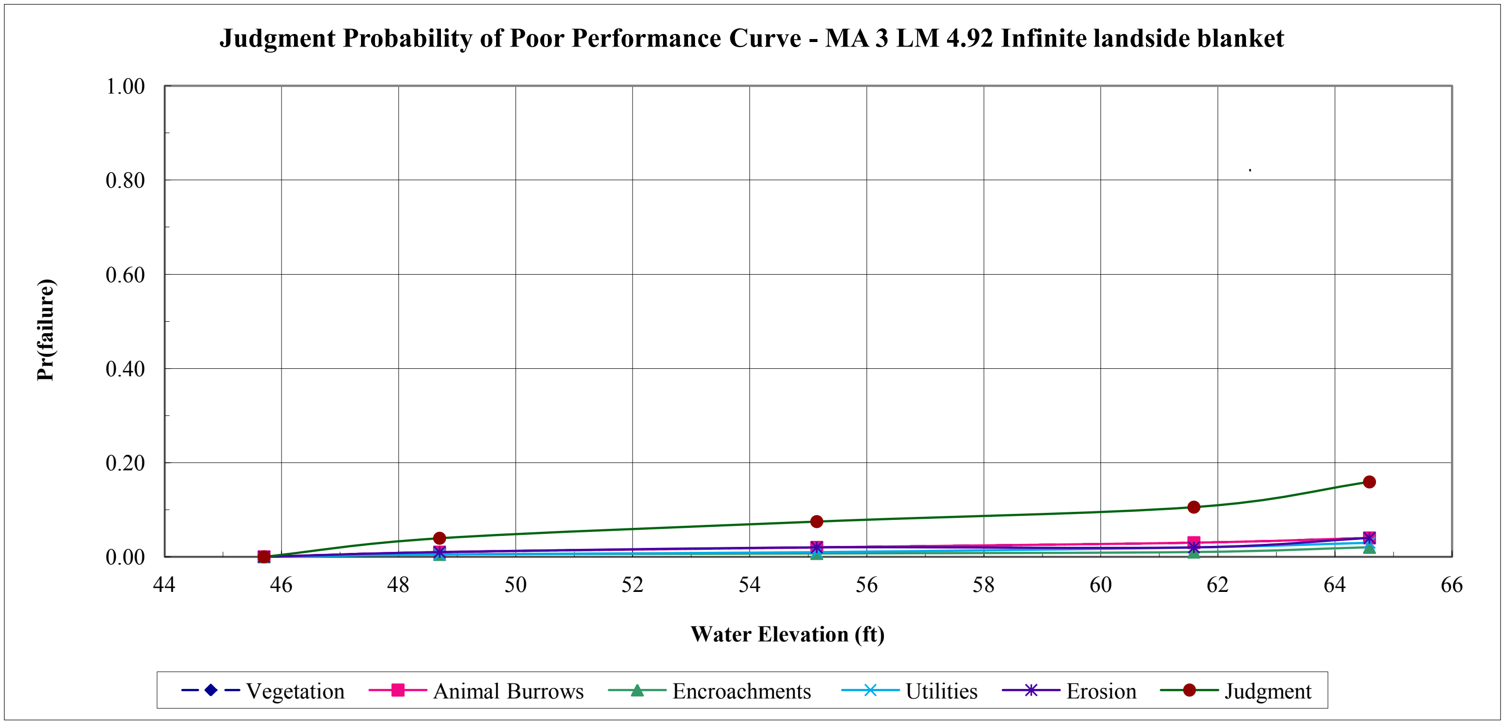
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: MA 3

Levee Mile: 4.92
River Mile: 2106963.58 N
Analysis Case: Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 09/12

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
45.70	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
48.70	0.0100	0.9900	0.0100	0.9900	0.0050	0.9950	0.0050	0.9950	0.0100	0.9900	0.0394	0.9606
55.15	0.0200	0.9800	0.0200	0.9800	0.0070	0.9930	0.0100	0.9900	0.0200	0.9800	0.0747	0.9253
61.59	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0200	0.9800	0.1054	0.8946
64.59	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

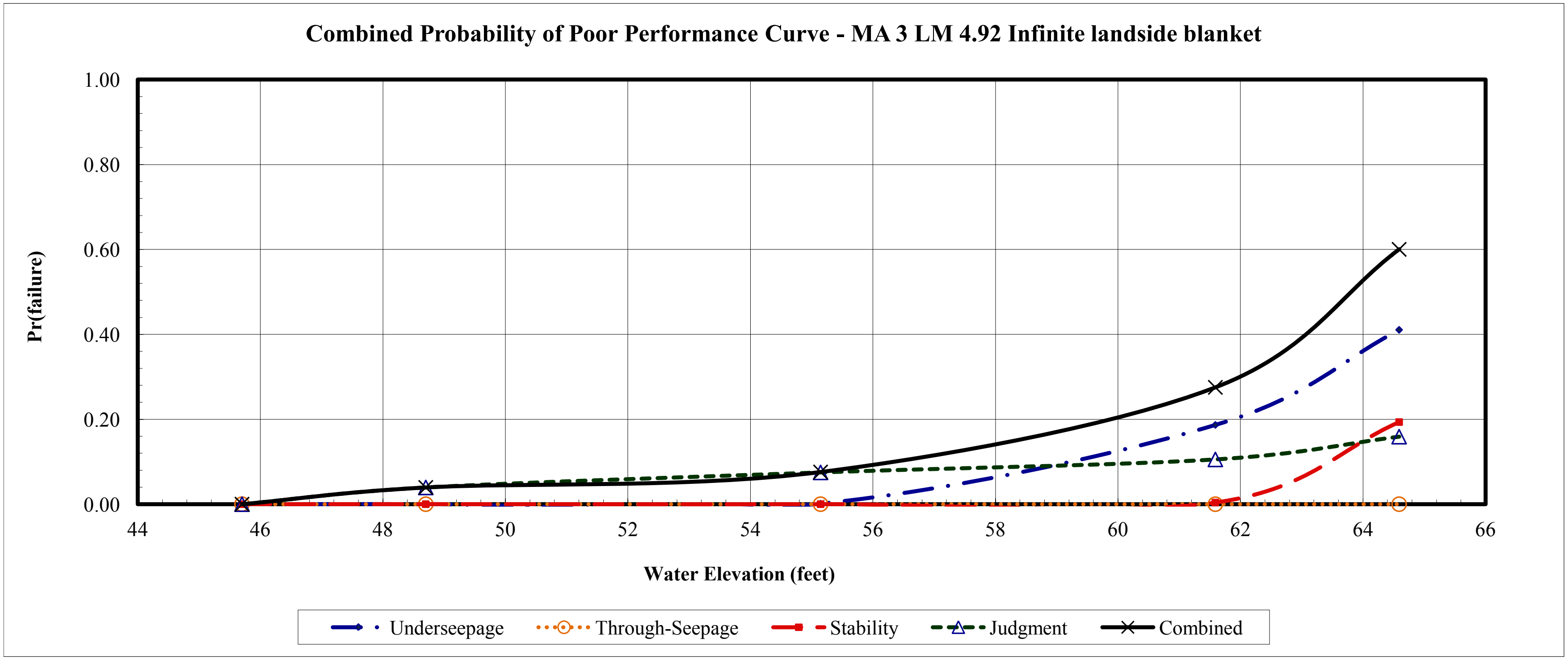
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: MA 3

Levee Mile: 4.92
River Mile: 2106963.58 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 64.59
L/S Toe Elev.: 45.70
W/S Toe Elev.: 45.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 09/12/2(

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
45.70	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
48.70	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0394	0.9606	0.0394	0.9606
55.15	0.0011	0.9989	0.0000	1.0000	0.0000	1.0000	0.0747	0.9253	0.0758	0.9242
61.59	0.1867	0.8133	0.0000	1.0000	0.0038	0.9962	0.1054	0.8946	0.2751	0.7249
64.59	0.4106	0.5894	0.0000	1.0000	0.1934	0.8066	0.1590	0.8410	0.6002	0.3998



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River South
Basin and Reach: LD 1

Levee Mile: 3.99
Coordinates: 2127081.8143 N; 6676331.1294E
Analysis Case Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.M. Bolton
Date: Updated 09/26/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WL0001_033B	5.29	14	9	84	66	3	26	31	767	98	CH/SM	0.01	SM	2.8	280	10526	24260	244824623	98
WL0001_005S	10.35					97					CL,CLs/sML	0.007	M,SW,SP-SM, S	10	1429				
WL0001_039B	24.04					35.5					, CL, sCL, CH/s	0.001	SM,SW,SW-SM	70	70000				
WL0001_006S	6.2					28					CL/SC/ML	0.007	M, SW, GW,GW	70	10000				
2F-97-1	13					11					CL, CLs	0.01	SC, SM, SP-SM	1	100				
WL0001_041B	11					5					CL,sMH	0.01	SM	1.2	120				
WL0001_007S	25.5					11					CLs,CL, CLs	0.007	SM, SP-SM	14	2000				
WL0001_046B	25.18					16					sCLs,sML,MH/	0.01	SP-SM	2.8	280				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WL0001_033B	CH	5	0.01	SM	4	0.14	5.29	SM	3	2.8							2.8
WL0001_005S	CL,CLs	10	0.007	sML	7	0.14	10.35	M,SW,SP-SM, S	97	10							10
WL0001_039B	Ls, CL, sCL, C	24	0.001	sML	4	0.1	24.04	SM,SW,SW-SM	35.5	70							70
WL0001_006S	CL	6	0.007	SC/ML	4	0.14	6.2	M, SW, GW,GW	28	70							70
2F-97-1	CL, CLs	13	0.01				13	SC, SM, SP-SM	11	1							1
WL0001_041B	CL,sMH	11	0.01				11	SM	5	1.2							1.2
WL0001_007S	CLs,CL, CLs	25.5	0.007				25.5	SM, SP-SM	11	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

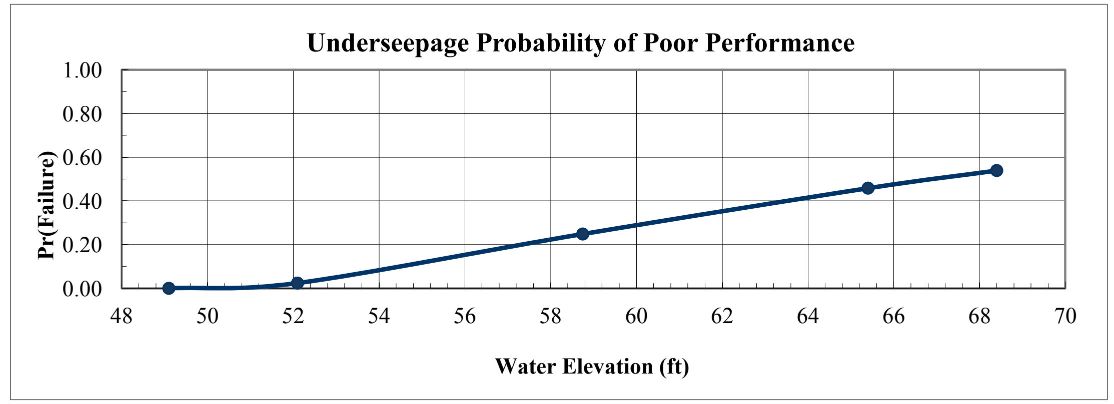
Levee Mile: 3.99
River Mile: 2127081.8143 N; 6676331.1294E
Analysis Case Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.M. Bolton
Date: Updated 09/26/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	10526	10315	98
Blanket Thickness (z)	14	9	66
Aquifer Thickness (d)	26	25	98

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	49.10	0.0000
Toe+3ft	3.00	52.10	0.0240
Half Height	9.65	58.75	0.2485
Crest-3ft	16.30	65.40	0.4584
Crest	19.30	68.40	0.5390

Crest	Rh
Head = 19.30	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	10526	13.63	26.00	49.99	1931.07	0.0123	17.65	1.30		
2	20841	13.63	26.00	49.99	2717.26	0.0090	18.10	1.33	0.055225	2.69
3	211	13.63	26.00	49.45	273.09	0.0575	11.65	0.86		
4	10526	22.62	26.00	49.99	2488.01	0.0097	18.00	0.80	1.946025	94.63
5	10526	4.63	26.00	49.97	1126.00	0.0199	16.64	3.59		
6	10526	13.63	51.48	49.99	2717.26	0.0178	18.10	1.33	0.055225	2.69
7	10526	13.63	0.52	49.45	273.09	0.0011	11.65	0.86		
								Total	2.056475	100.00

E[I] = 1.300000
Var[I]= 2.056475
σ[I]= 1.434041
V(I) = 1.103109

E[ln I] = -0.135679
σ [ln I] = 0.892237

Ic=	0.80

ln(I crit) = -0.223144

β =	-0.152066
F(z) =	0.460955
Pr(f) % =	53.904505

Crest-3ft	Rh
Head = 16.30	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	10526	13.63	26.00	49.99	1931.07	0.0123	14.91	1.09		
2	20841	13.63	26.00	49.99	2717.26	0.0090	15.29	1.12	0.040000	2.72
3	211	13.63	26.00	49.45	273.09	0.0575	9.84	0.72		
4	10526	22.62	26.00	49.99	2488.01	0.0097	15.20	0.67	1.392400	94.57
5	10526	4.63	26.00	49.97	1126.00	0.0199	14.05	3.03		
6	10526	13.63	51.48	49.99	2717.26	0.0178	15.29	1.12	0.040000	2.72
7	10526	13.63	0.52	49.45	273.09	0.0011	9.84	0.72		
								Total	1.472400	100.00

E[I] = 1.090000
Var[I]= 1.472400
σ[I]= 1.213425
V(I) = 1.113234

E[ln I] = -0.316902
σ [ln I] = 0.897863

Ic=	0.80

ln(I crit) = -0.223144

β =	-0.352951
F(z) =	0.541583
Pr(f) % =	45.841661

Half Height	Rh
Head = 9.65	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	10526	13.63	26.00	49.99	1931.07	0.0123	8.83	0.65		
2	20841	13.63	26.00	49.99	2717.26	0.0090	9.05	0.66	0.013225	2.56
3	211	13.63	26.00	49.45	273.09	0.0575	5.82	0.43		
4	10526	22.62	26.00	49.99	2488.01	0.0097	9.00	0.40	0.490000	94.88
5	10526	4.63	26.00	49.97	1126.00	0.0199	8.32	1.80		
6	10526	13.63	51.48	49.99	2717.26	0.0178	9.05	0.66	0.013225	2.56
7	10526	13.63	0.52	49.45	273.09	0.0011	5.82	0.43		
								Total	0.516450	100.00

E[I] = 0.650000
Var[I]= 0.516450
σ[I]= 0.718645
V(I) = 1.105607

E[ln I] = -0.830069
σ [ln I] = 0.893629

Ic=	0.80

ln(I crit) = -0.223144

β =	-0.928875
F(z) =	0.751485
Pr(f) % =	24.851517

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	10526	13.63	26.00	49.99	1931.07	0.0123	2.74	0.20		
2	20841	13.63	26.00	49.99	2717.26	0.0090	2.81	0.21	0.001600	3.10
3	211	13.63	26.00	49.45	273.09	0.0575	1.81	0.13		
4	10526	22.62	26.00	49.99	2488.01	0.0097	2.80	0.12	0.048400	93.80
5	10526	4.63	26.00	49.97	1126.00	0.0199	2.59	0.56		
6	10526	13.63	51.48	49.99	2717.26	0.0178	2.81	0.21	0.001600	3.10
7	10526	13.63	0.52	49.45	273.09	0.0011	1.81	0.13		
								Total	0.051600	100.00

E[I] = 0.200000
Var[I]= 0.051600
σ[I]= 0.227156
V(I) = 1.135782

E[ln I] = -2.023714
σ [ln I] = 0.910248

Ic=	0.80

ln(I crit) = -0.223144

β =	-2.223255
F(z) =	0.976042
Pr(f) % =	2.395820

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

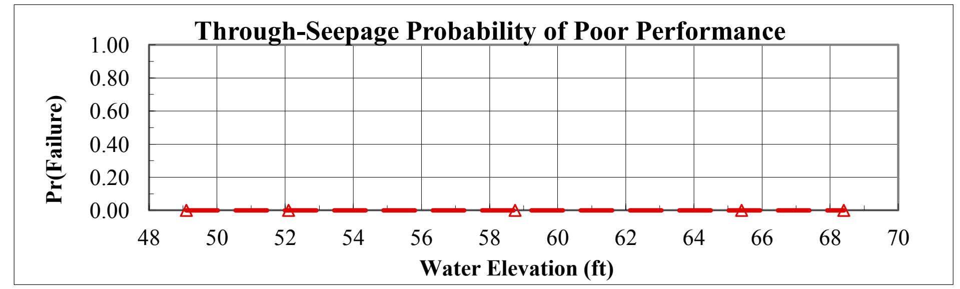
Levee Mile: 3.99
River Mile: 2127081.8143 N; 6676331.1294E
Analysis Case Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.M. Bolton
Date: Updated 09/26/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	49.10	0.0000
Toe+3ft	3.00	52.10	0.000000
Half Height	9.65	58.75	0.000000
Crest-3ft	16.30	65.40	0.000000
Crest	19.30	68.40	0.000000

Crest | **Head =** 19.30 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 16.30 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height | **Head =** 9.65 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

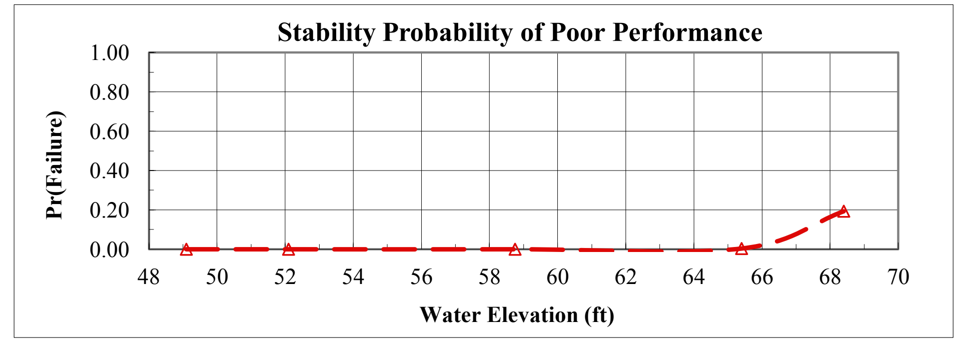
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 3.99
River Mile: 2127081.8143 N; 6676331.1294E
Analysis Case Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.M. Bolton
Date: Updated 09/26/2012

Random Variables				
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %	
Levee ϕ'	29	1	5.00	
Levee γ	120	6	5.00	
Foundation c'	150	17	11.50	
Foundation γ	120	6	5.00	
Foundation ϕ'	31	4	11.50	



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	49.10	0.0000
Toe+3ft	3.00	52.10	0.000000
Half Height	9.65	58.75	0.000000
Crest-3ft	16.30	65.40	0.003757
Crest	19.30	68.40	0.193409

Crest	Head =	19.30	Pr(f)=0	NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.09		
2	28	120	150	120	31	1.20	0.010816	93.19
3	30	120	150	120	31	0.99		
4	29	114	150	120	31	1.07	0.000306	2.64
5	29	126	150	120	31	1.11		
6	29	120	133	120	31	1.09	0.000484	4.17
7	29	120	167	120	31	1.05		
8	29	120	150	114	31	1.09	0.000000	0.00
9	29	120	150	126	31	1.09		
10	29	120	150	120	27	1.09	0.000000	0.00
11	29	120	150	120	35	1.09		

E[FS] =	1.094000	E[ln FS] =	0.085015	Total	0.011606	100.00
Var[FS]=	0.011606					
σ[FS]=	0.107732	σ[ln FS]=	0.098238		β =	0.865401
V(FS) =	0.098476				F(z) =	0.193409
FS req'd =	1.00	ln(FS req'd) =	0.000000		Pr(f) % =	19.340941

Crest-3ft	Head =	16.30	Pr(f)=0	NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.25		
2	28	120	150	120	31	1.35	0.009604	90.64
3	30	120	150	120	31	1.16		
4	29	114	150	120	31	1.24	0.000144	1.36
5	29	126	150	120	31	1.26		
6	29	120	133	120	31	1.25	0.000784	7.40
7	29	120	167	120	31	1.19		
8	29	120	150	114	31	1.25	0.000064	0.60
9	29	120	150	126	31	1.23		
10	29	120	150	120	27	1.25	0.000000	0.00
11	29	120	150	120	35	1.25		

E[FS] =	1.250000	E[ln FS] =	0.219764	Total	0.010596	100.00
Var[FS]=	0.010596					
σ[FS]=	0.102937	σ[ln FS]=	0.082210		β =	2.673193
V(FS) =	0.082349				F(z) =	0.003757
FS req'd =	1.00	ln(FS req'd) =	0.000000		Pr(f) % =	0.375665

Half Height	Head =	9.65	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] =		E[ln FS] =		Total		
Var[FS]=						
σ[FS]=		σ[ln FS]=			β =	
V(FS) =					F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000		Pr(f) % =	0.000000

Toe+3ft	Head =	3.00	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] =		E[ln FS] =		Total		
Var[FS]=						
σ[FS]=		σ[ln FS]=			β =	
V(FS) =					F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000		Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

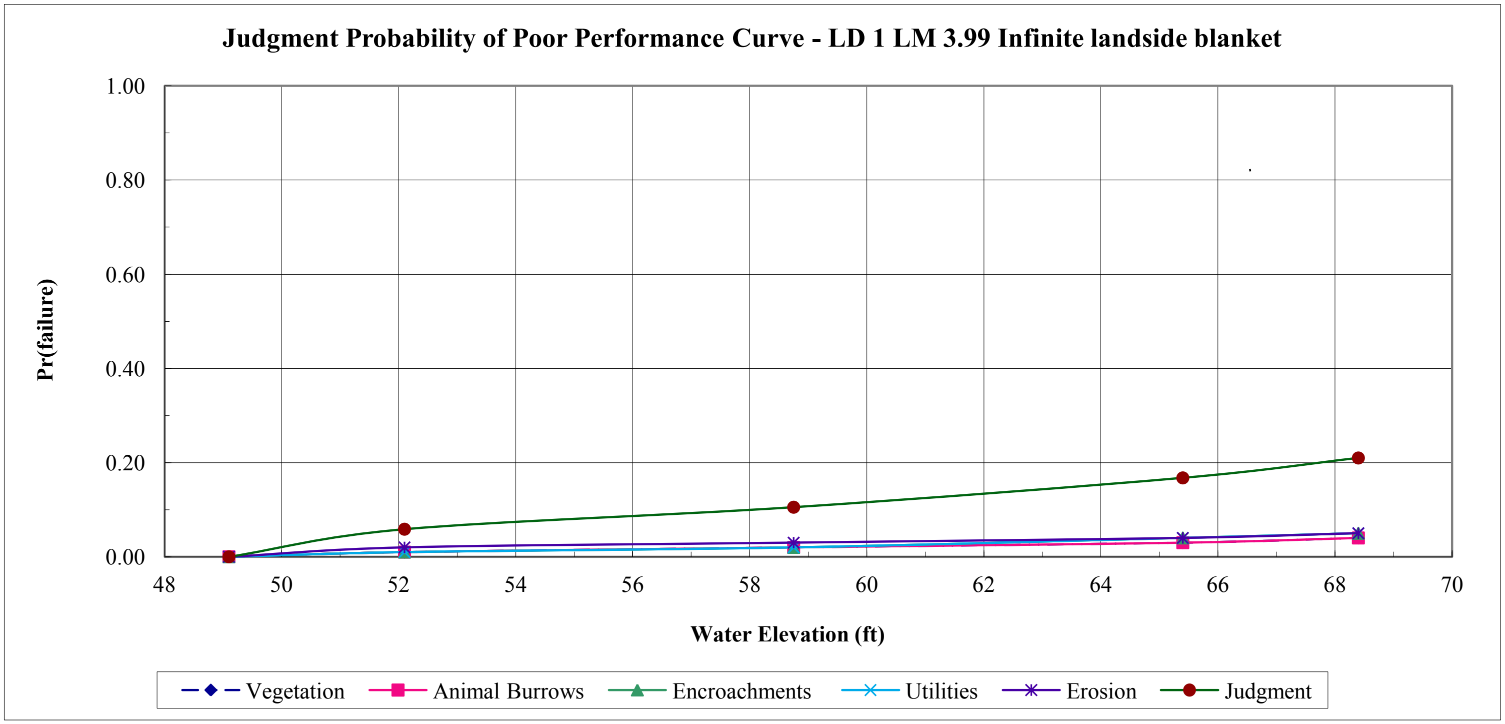
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 3.99
River Mile: 2127081.8143
Analysis Case: Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.I
Date: Updated 09/26

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
49.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
52.10	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0200	0.9800	0.0586	0.9414
58.75	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0300	0.9700	0.1053	0.8947
65.40	0.0300	0.9700	0.0300	0.9700	0.0400	0.9600	0.0400	0.9600	0.0400	0.9600	0.1676	0.8324
68.40	0.0400	0.9600	0.0400	0.9600	0.0500	0.9500	0.0500	0.9500	0.0500	0.9500	0.2098	0.7902



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

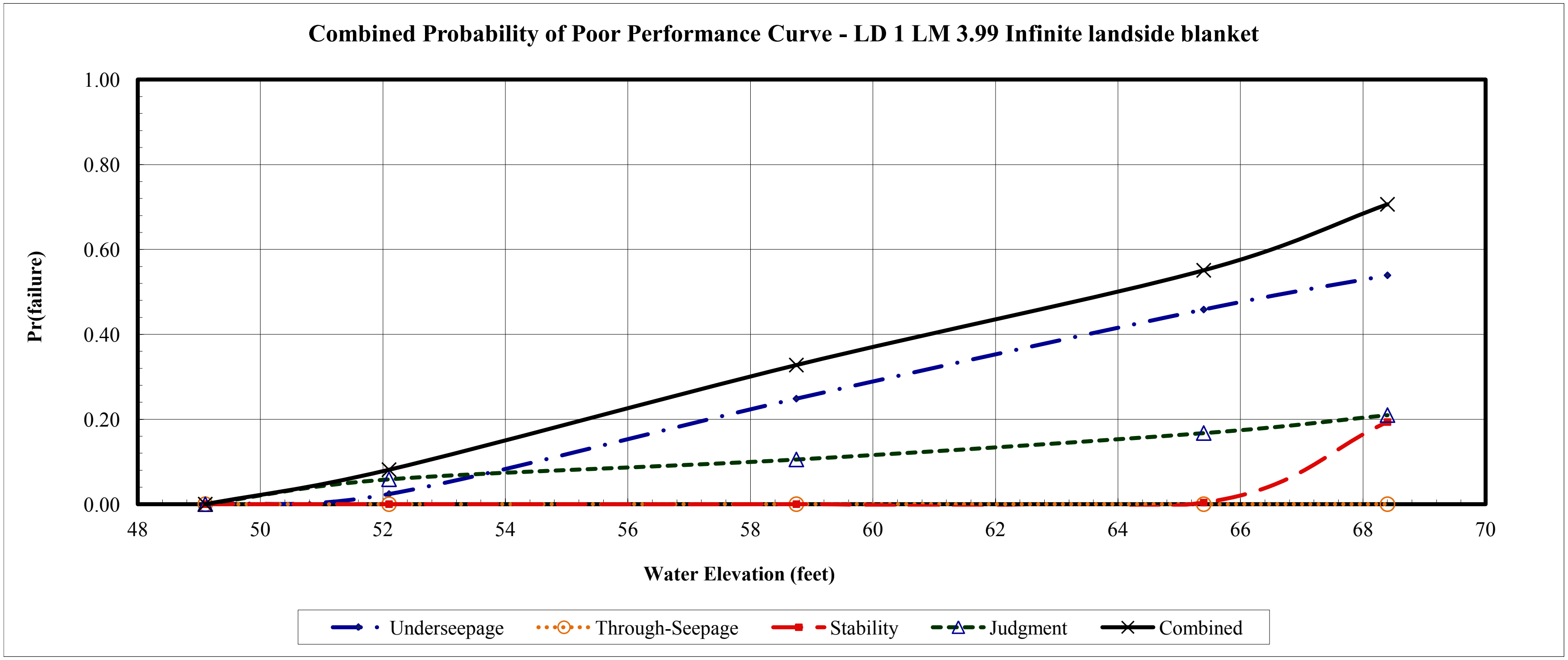
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 3.99
River Mile: 2127081.8143 N
Analysis Case: Infinite landside blanket

Crest Elev.: 68.40
L/S Toe Elev.: 49.10
W/S Toe Elev.: 40.00

Analysis By: T. Huynh
Checked By: E.W. James\J.M.
Date: Updated 09/26/2(

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
49.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
52.10	0.0240	0.9760	0.0000	1.0000	0.0000	1.0000	0.0586	0.9414	0.0812	0.9188
58.75	0.2485	0.7515	0.0000	1.0000	0.0000	1.0000	0.1053	0.8947	0.3276	0.6724
65.40	0.4584	0.5416	0.0000	1.0000	0.0038	0.9962	0.1676	0.8324	0.5509	0.4491
68.40	0.5390	0.4610	0.0000	1.0000	0.1934	0.8066	0.2098	0.7902	0.7062	0.2938



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River South
Basin and Reach: LD 1

Levee Mile: 9.31
Coordinates: 2156078.18 N; 6673804.98 E
Analysis Case Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
SL0001_001B	20	17	9	92	53	14	15	13	162	87	CL, ML	0.007	SP-SM	14	2000	773	797	343652	98
WL0001_059B	25					14.9					CL, CH, sML	0.01	SM	1.2	120				
WL0001_012S	31					16.5					CL, sML	0.01	/GW-GM, GP-(	5.73	573				
2F-07-05	8.5					2.5					sML	0.14	SM	1.2	9				
2F-07-06	12					5					sML	0.14	SM	1.2	9				
2F-07-01	10					40					CL, ML	0.01	/SW-SM, GM S	13	1300				
WL0001_064B	12										ML, CL	0.01	SP-SM	14	1400				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
SL0001_001B	CL, ML	20	0.007				20	SP-SM	14	14							14
WL0001_059B	CL, CH, sML	25	0.01				25	SM	14.9	1.2							1.2
WL0001_012S	CL, sML	31	0.01				31	SM	8	1.2	;W-GM, GP-GN	8.5	10				5.73
2F-07-05	sML	8.5	0.14				8.5	SM	2.5	1.2							1.2
2F-07-06	sML	12	0.14				12	SM	5	1.2							1.2
2F-07-01	CL, ML	10	0.01				10	SP	30	14	;W-SM, GM SV	10	10				13
WL0001_064B	ML, CL	12	0.01				12	SP-SM	9	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

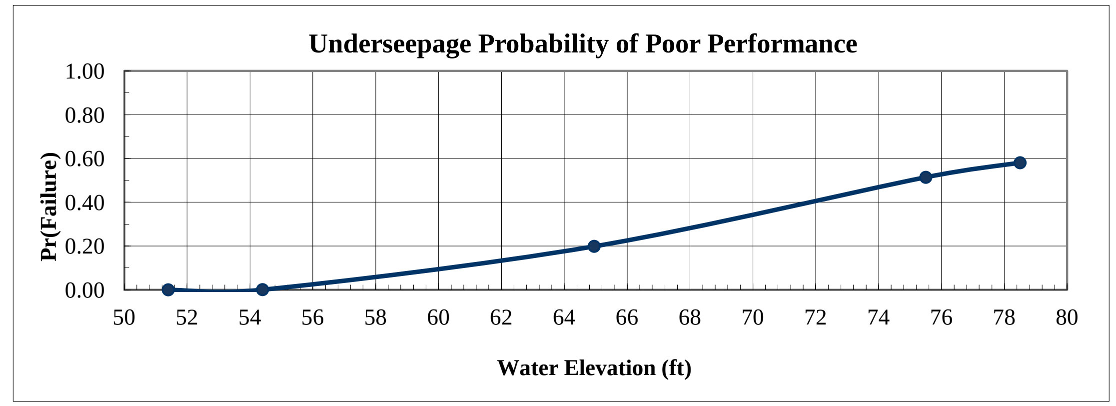
Levee Mile: 9.31
River Mile: 2156078.18 N; 6673804.98 E
Analysis Case Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	773	758	98
Blanket Thickness (z)	17	9	53
Aquifer Thickness (d)	15	13	87

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	51.40	0.0000
Toe+3ft	3.00	54.40	0.0008
Half Height	13.55	64.95	0.1986
Crest-3ft	24.10	75.50	0.5140
Crest	27.10	78.50	0.5805

Crest	Rh
Head = 27.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	773	17.00	15.00	49.79	443.98	0.0240	19.29	1.13		
2	1531	17.00	15.00	49.89	624.73	0.0186	21.04	1.24	0.164025	24.22
3	15	17.00	15.00	41.56	62.79	0.0640	7.26	0.43		
4	773	26.01	15.00	49.86	549.17	0.0206	20.41	0.78	0.455625	67.28
5	773	7.99	15.00	49.56	304.37	0.0310	17.04	2.13		
6	773	17.00	28.05	49.89	607.13	0.0356	20.91	1.23	0.057600	8.50
7	773	17.00	1.95	48.44	160.08	0.0058	12.82	0.75		
								Total	0.677250	100.00

E[I] = 1.130000
Var[I]= 0.677250
σ[I]= 0.822952
V(I) = 0.728276

E[ln I] = -0.090542
σ [ln I] = 0.652319
ln(I crit) = -0.223144

Ic=	0.80

β =	-0.138801
F(z) =	0.419459
Pr(f) % =	58.054056

Crest-3ft	Rh
Head = 24.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	773	17.00	15.00	49.79	443.98	0.0240	17.15	1.01		
2	1531	17.00	15.00	49.89	624.73	0.0186	18.71	1.10	0.129600	24.28
3	15	17.00	15.00	41.56	62.79	0.0640	6.46	0.38		
4	773	26.01	15.00	49.86	549.17	0.0206	18.15	0.70	0.360000	67.45
5	773	7.99	15.00	49.56	304.37	0.0310	15.16	1.90		
6	773	17.00	28.05	49.89	607.13	0.0356	18.59	1.09	0.044100	8.26
7	773	17.00	1.95	48.44	160.08	0.0058	11.40	0.67		
								Total	0.533700	100.00

E[I] = 1.010000
Var[I]= 0.533700
σ[I]= 0.730548
V(I) = 0.723315

E[ln I] = -0.200451
σ [ln I] = 0.648693
ln(I crit) = -0.223144

Ic=	0.80

β =	-0.309008
F(z) =	0.486047
Pr(f) % =	51.395286

Half Height	Rh
Head = 13.55	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	773	17.00	15.00	49.79	443.98	0.0240	9.64	0.57		
2	1531	17.00	15.00	49.89	624.73	0.0186	10.52	0.62	0.042025	24.60
3	15	17.00	15.00	41.56	62.79	0.0640	3.63	0.21		
4	773	26.01	15.00	49.86	549.17	0.0206	10.21	0.39	0.115600	67.66
5	773	7.99	15.00	49.56	304.37	0.0310	8.52	1.07		
6	773	17.00	28.05	49.89	607.13	0.0356	10.45	0.61	0.013225	7.74
7	773	17.00	1.95	48.44	160.08	0.0058	6.41	0.38		
								Total	0.170850	100.00

E[I] = 0.570000
Var[I]= 0.170850
σ[I]= 0.413340
V(I) = 0.725158

E[ln I] = -0.773396
σ [ln I] = 0.650042
ln(I crit) = -0.223144

Ic=	0.80

β =	-1.189764
F(z) =	0.801360
Pr(f) % =	19.864030

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	773	17.00	15.00	49.79	443.98	0.0240	2.14	0.13		
2	1531	17.00	15.00	49.89	624.73	0.0186	2.33	0.14	0.002025	23.68
3	15	17.00	15.00	41.56	62.79	0.0640	0.80	0.05		
4	773	26.01	15.00	49.86	549.17	0.0206	2.26	0.09	0.005625	65.79
5	773	7.99	15.00	49.56	304.37	0.0310	1.89	0.24		
6	773	17.00	28.05	49.89	607.13	0.0356	2.31	0.14	0.000900	10.53
7	773	17.00	1.95	48.44	160.08	0.0058	1.42	0.08		
								Total	0.008550	100.00

E[I] = 0.130000
Var[I]= 0.008550
σ[I]= 0.092466
V(I) = 0.711279

E[ln I] = -2.244922
σ [ln I] = 0.639845
ln(I crit) = -0.223144

Ic=	0.80

β =	-3.508538
F(z) =	0.999211
Pr(f) % =	0.078941

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

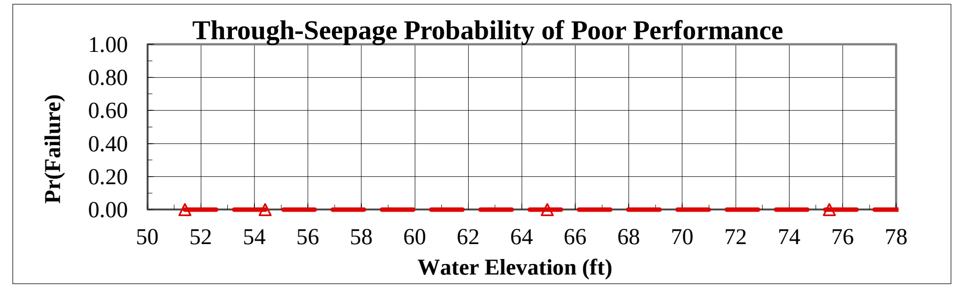
Levee Mile: 9.31
River Mile: 2156078.18 N; 6673804.98 E
Analysis Case Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	51.40	0.0000
Toe+3ft	3.00	54.40	0.000000
Half Height	13.55	64.95	0.000000
Crest-3ft	24.10	75.50	0.000000
Crest	27.10	78.50	0.000000

Crest	**Head =**	27.10	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =

FS req'd =	1.00	ln(FS req'd) =	0.000000	**β =**	
				F(z) =	
				Pr(f) % =	0.000000

Crest-3ft	**Head =**	24.10	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =

FS req'd =	1.00	ln(FS req'd) =	0.000000	**β =**	
				F(z) =	
				Pr(f) % =	0.000000

Half Height	**Head =**	13.55	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =

FS req'd =	1.00	ln(FS req'd) =	0.000000	**β =**	
				F(z) =	
				Pr(f) % =	0.000000

Toe+3ft	**Head =**	3.00	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =

FS req'd =	1.00	ln(FS req'd) =	0.000000	**β =**	
				F(z) =	
				Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

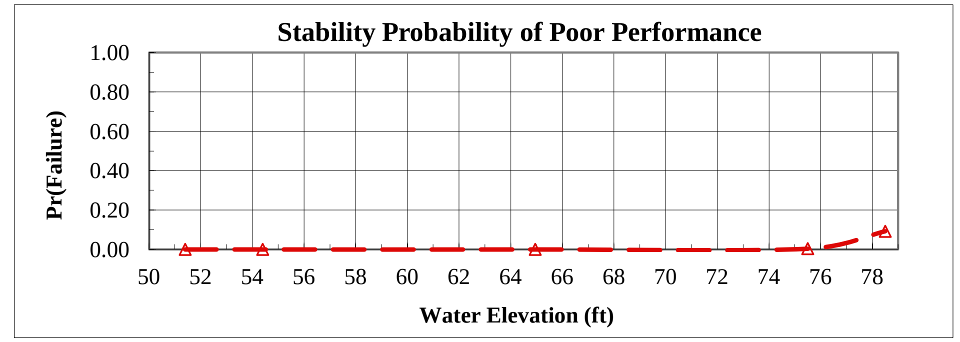
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 9.31
River Mile: 2156078.18 N; 6673804.98 E
Analysis Case Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables				
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %	
Levee ϕ'	29	1	5.00	
Levee γ	120	6	5.00	
Foundation c'	150	17	11.50	
Foundation γ	120	6	5.00	
Foundation ϕ'	31	4	11.50	



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	51.40	0.0000
Toe+3ft	3.00	54.40	0.000000
Half Height	13.55	64.95	0.000000
Crest-3ft	24.10	75.50	0.003757
Crest	27.10	78.50	0.091721

Crest | **Head =** 27.10 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.50		
2	28	120	150	120	31	1.69	0.024806	13.98
3	30	120	150	120	31	1.37		
4	29	114	150	120	31	1.46	0.001560	0.88
5	29	126	150	120	31	1.54		
6	29	120	133	120	31	1.70	0.136530	76.92
7	29	120	167	120	31	0.96		
8	29	120	150	114	31	1.53	0.003782	2.13
9	29	120	150	126	31	1.41		
10	29	120	150	120	27	1.58	0.010816	6.09
11	29	120	150	120	35	1.37		

E[FS] = 1.499000 | E[ln FS] = 0.366785 | Total 0.177495 | 100.00
Var[FS]= 0.177495
σ[FS]= 0.421302 | σ[ln FS]= 0.275730 | β = 1.330229
V(FS) = 0.281055 | F(z) = 0.091721
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 9.172136

Crest-3ft | **Head =** 24.10 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.25		
2	28	120	150	120	31	1.35	0.009604	90.64
3	30	120	150	120	31	1.16		
4	29	114	150	120	31	1.24	0.000144	1.36
5	29	126	150	120	31	1.26		
6	29	120	133	120	31	1.25	0.000784	7.40
7	29	120	167	120	31	1.19		
8	29	120	150	114	31	1.25	0.000064	0.60
9	29	120	150	126	31	1.23		
10	29	120	150	120	27	1.25	0.000000	0.00
11	29	120	150	120	35	1.25		

E[FS] = 1.250000 | E[ln FS] = 0.219764 | Total 0.010596 | 100.00
Var[FS]= 0.010596
σ[FS]= 0.102937 | σ[ln FS]= 0.082210 | β = 2.673193
V(FS) = 0.082349 | F(z) = 0.003757
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.375665

Half Height | **Head =** 13.55 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

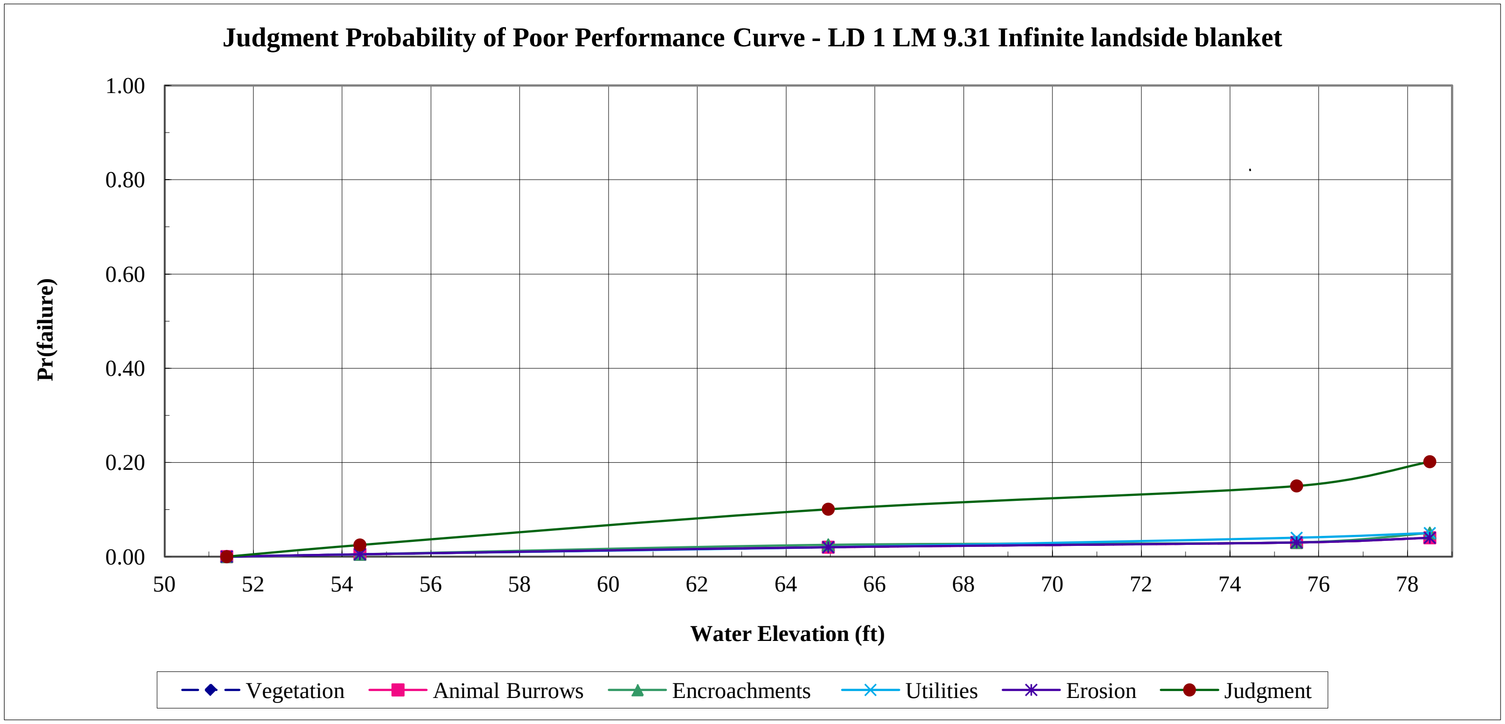
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 9.31
River Mile: 2156078.18 N
Analysis Case: Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 2/21/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
51.40	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
54.40	0.0050	0.9950	0.0050	0.9950	0.0050	0.9950	0.0050	0.9950	0.0050	0.9950	0.0248	0.9752
64.95	0.0200	0.9800	0.0200	0.9800	0.0250	0.9750	0.0200	0.9800	0.0200	0.9800	0.1007	0.8993
75.50	0.0300	0.9700	0.0300	0.9700	0.0300	0.9700	0.0400	0.9600	0.0300	0.9700	0.1501	0.8499
78.50	0.0400	0.9600	0.0400	0.9600	0.0500	0.9500	0.0500	0.9500	0.0400	0.9600	0.2015	0.7985



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

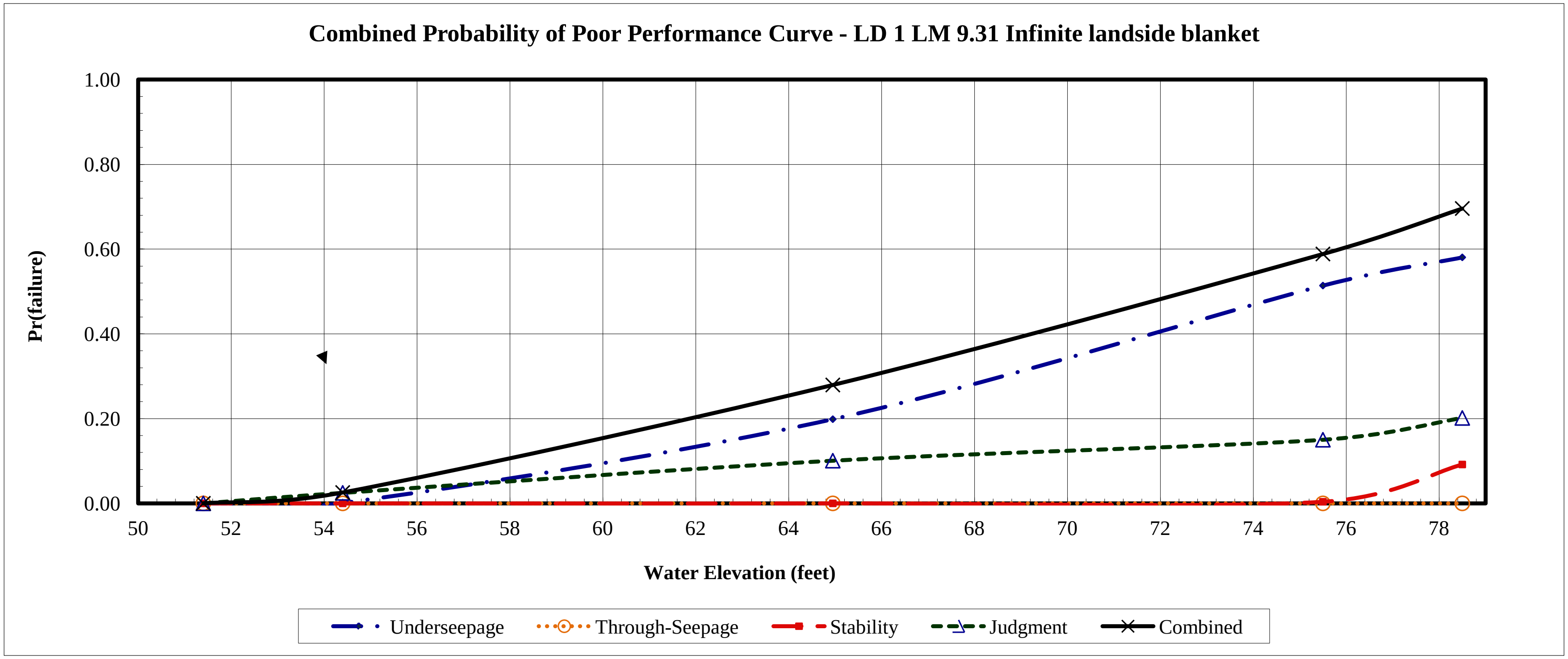
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 1

Levee Mile: 9.31
River Mile: 2156078.18 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 78.50
L/S Toe Elev.: 51.40
W/S Toe Elev.: 53.70

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 2/21/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
51.40	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
54.40	0.0008	0.9992	0.0000	1.0000	0.0000	1.0000	0.0248	0.9752	0.0255	0.9745
64.95	0.1986	0.8014	0.0000	1.0000	0.0000	1.0000	0.1007	0.8993	0.2793	0.7207
75.50	0.5140	0.4860	0.0000	1.0000	0.0038	0.9962	0.1501	0.8499	0.5885	0.4115
78.50	0.5805	0.4195	0.0000	1.0000	0.0917	0.9083	0.2015	0.7985	0.6958	0.3042



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River South
Basin and Reach: LD 9

Levee Mile: 0.52
Coordinates: 2188213.88 N; 6668679.41 E
Analysis Case Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)				
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WL0009_001S	19.5	13	5	55	38	52.5	28	17	383	61
WL0009_007A	10					50.8				
WL0009_007A	7.5					27.5				
WL0009_12B	13					10.8				
2F-07-7	10					17.5				
SL0009_001B	10					24				
WL0009_009B	20					11				

Boring #	Hydraulic Conductivity Vairables (Kb and Kf)								
	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
	Material	Kb (ft/day)	Material	Kf (ft/day)					
WL0009_001S	CL	0.007	SP-SM/SP-SC	14	2000	1730	1217	1685521	70
WL0009_007A	CL, CLs	0.007	SM, SP, SM/SW	17.94	2563				
WL0009_007A	CL, ML	0.007	SP-SM, SP-SC	14	2000				
WL0009_12B	CL-ML	0.007	SP-SM/SW	23.46	3351				
2F-07-7	ML	0.14	SW/SP-SM	26	186				
SL0009_001B	SC, CL, ML	1.4	SP-SM/SP-SC	14	10				
WL0009_009B	ML, CL	0.007	SM, SP-SM	14	2000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WL0009_001S	CL	19.5	0.007				19.5	SP-SM	47	14	SP-SC	5.5	14				14
WL0009_007A	CL, CLs	10	0.007				10	SP-SM, SP, SM	36.5	14	SW-SM	14.3	28				17.94
WL0009_007A	CL, ML	7.5	0.007				7.5	SP-SM, SP-SC	27.5	14							14
WL0009_12B	CL-ML	13	0.007				13	SP-SM	3.5	14	SW	7.3	28				23.46
2F-07-7	ML	10	0.14				10	SW	15	28	SP-SM	2.5	14				26
SL0009_001B	SC, CL, ML	10	1.4				10	SP-SM	12	14	SP-SC	12	14				14
WL0009_009B	ML, CL	20	0.007				20	SM, SP-SM	11	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 9

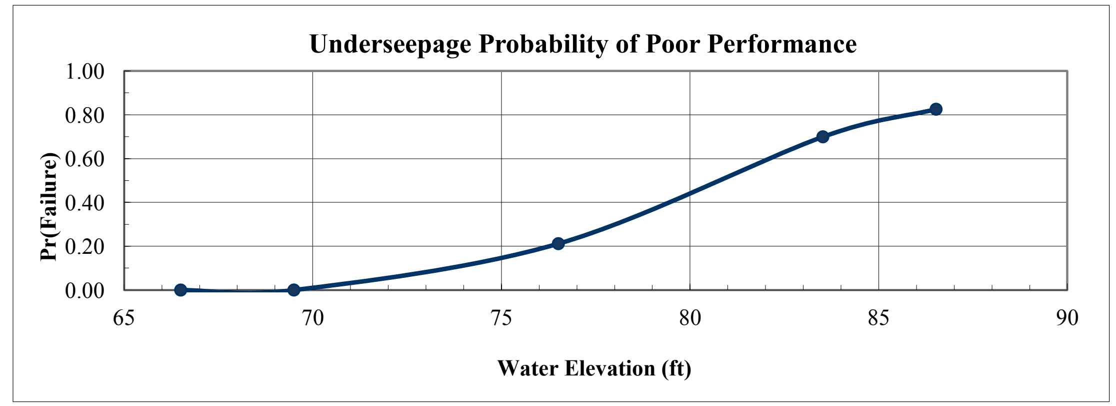
Levee Mile: 0.52
River Mile: 2188213.88 N; 6668679.41 E
Analysis Case Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	1730	1217	70
Blanket Thickness (z)	13	5	38
Aquifer Thickness (d)	28	17	61

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f) Stability
Toe	0.00	66.50	0.0000
Toe+3ft	3.00	69.50	0.0001
Half Height	10.01	76.51	0.2117
Crest-3ft	17.02	83.52	0.6995
Crest	20.02	86.52	0.8254

Crest	Rh
Head =	20.02

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1730	13.00	28.00	49.93	793.55	0.0288	16.32	1.26		
2	2947	13.00	28.00	49.96	1035.72	0.0230	17.06	1.31	0.012100	4.40
3	513	13.00	28.00	49.78	432.12	0.0458	14.14	1.09		
4	1730	18.00	28.00	49.95	933.77	0.0251	16.79	0.93	0.255025	92.66
5	1730	8.00	28.00	49.89	622.51	0.0349	15.53	1.94		
6	1730	13.00	45.00	49.96	1006.01	0.0379	16.98	1.31	0.008100	2.94
7	1730	13.00	11.00	49.83	497.38	0.0162	14.70	1.13		
								Total	0.275225	100.00

E[I] = 1.260000
Var[I]= 0.275225
σ[I]= 0.524619
V(I) = 0.416364

E[ln I] = 0.151176
σ [ln I] = 0.399838

Ic=	0.80

ln(I crit) = -0.223144

β =	0.378094
F(z) =	0.174591
Pr(f) % =	82.540925

Crest-3ft	Rh
Head =	17.02

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1730	13.00	28.00	49.93	793.55	0.0288	13.87	1.07		
2	2947	13.00	28.00	49.96	1035.72	0.0230	14.50	1.12	0.010000	4.99
3	513	13.00	28.00	49.78	432.12	0.0458	12.02	0.92		
4	1730	18.00	28.00	49.95	933.77	0.0251	14.27	0.79	0.184900	92.21
5	1730	8.00	28.00	49.89	622.51	0.0349	13.20	1.65		
6	1730	13.00	45.00	49.96	1006.01	0.0379	14.44	1.11	0.005625	2.81
7	1730	13.00	11.00	49.83	497.38	0.0162	12.50	0.96		
								Total	0.200525	100.00

E[I] = 1.070000
Var[I]= 0.200525
σ[I]= 0.447800
V(I) = 0.418505

E[ln I] = -0.013038
σ [ln I] = 0.401737

Ic=	0.80

ln(I crit) = -0.223144

β =	-0.032453
F(z) =	0.300489
Pr(f) % =	69.951062

Half Height	Rh
Head =	10.01

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1730	13.00	28.00	49.93	793.55	0.0288	8.16	0.63		
2	2947	13.00	28.00	49.96	1035.72	0.0230	8.53	0.66	0.003600	5.32
3	513	13.00	28.00	49.78	432.12	0.0458	7.07	0.54		
4	1730	18.00	28.00	49.95	933.77	0.0251	8.39	0.47	0.062500	92.32
5	1730	8.00	28.00	49.89	622.51	0.0349	7.77	0.97		
6	1730	13.00	45.00	49.96	1006.01	0.0379	8.49	0.65	0.001600	2.36
7	1730	13.00	11.00	49.83	497.38	0.0162	7.35	0.57		
								Total	0.067700	100.00

E[I] = 0.630000
Var[I]= 0.067700
σ[I]= 0.260192
V(I) = 0.413004

E[ln I] = -0.540782
σ [ln I] = 0.396853

Ic=	0.80

ln(I crit) = -0.223144

β =	-1.362674
F(z) =	0.788258
Pr(f) % =	21.174185

Toe+3ft	Rh
Head =	3.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1730	13.00	28.00	49.93	793.55	0.0288	2.45	0.19		
2	2947	13.00	28.00	49.96	1035.72	0.0230	2.56	0.20	0.000400	6.40
3	513	13.00	28.00	49.78	432.12	0.0458	2.12	0.16		
4	1730	18.00	28.00	49.95	933.77	0.0251	2.52	0.14	0.005625	90.00
5	1730	8.00	28.00	49.89	622.51	0.0349	2.33	0.29		
6	1730	13.00	45.00	49.96	1006.01	0.0379	2.54	0.20	0.000225	3.60
7	1730	13.00	11.00	49.83	497.38	0.0162	2.20	0.17		
								Total	0.006250	100.00

E[I] = 0.190000
Var[I]= 0.006250
σ[I]= 0.079057
V(I) = 0.416089

E[ln I] = -1.740569
σ [ln I] = 0.399594

Ic=	0.80

ln(I crit) = -0.223144

β =	-4.355841
F(z) =	0.999927
Pr(f) % =	0.007311

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 9

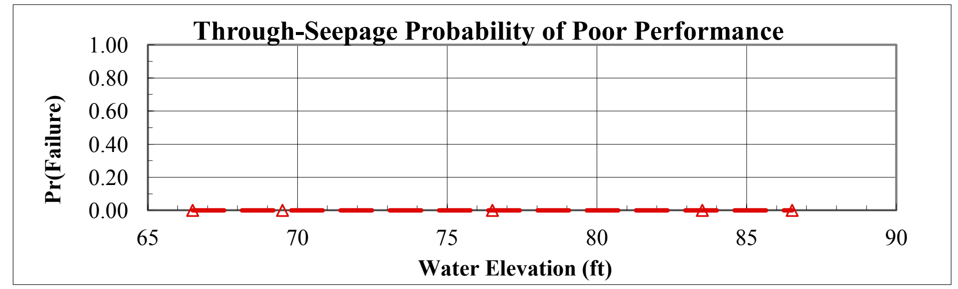
Levee Mile: 0.52
River Mile: 2188213.88 N; 6668679.41 E
Analysis Case Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f) Stability
Toe	0.00	66.50	0.0000
Toe+3ft	3.00	69.50	0.000000
Half Height	10.01	76.51	0.000000
Crest-3ft	17.02	83.52	0.000000
Crest	20.02	86.52	0.000000

Crest | **Head =** 20.02 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 17.02 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height | **Head =** 10.01 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

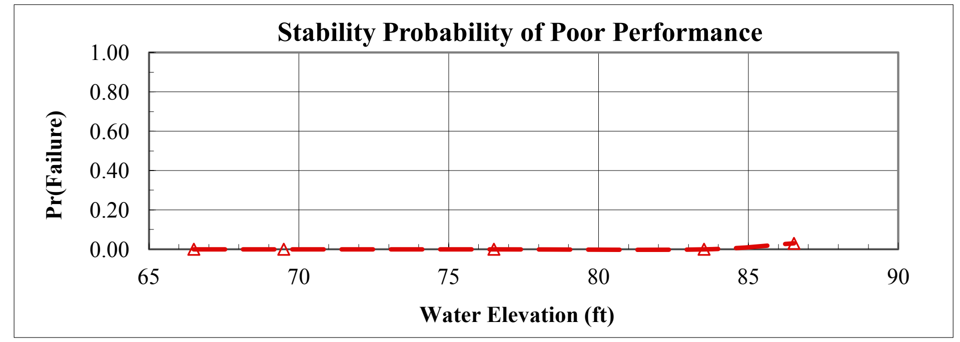
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 9

Levee Mile: 0.52
River Mile: 2188213.88 N; 6668679.41 E
Analysis Case Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	29	4	13.00
Levee γ	120	48	40.00
Foundation Cohesion	150	11	7.00
Foundation γ	120	16	13.00
Foundation Φ	31	12	40.00



Analysis Case	Head	Elevation	Pr(f) Stability
Toe	0.00	66.50	0.0000
Toe+3ft	3.00	69.50	0.000000
Half Height	10.01	76.51	0.000000
Crest-3ft	17.02	83.52	0.000000
Crest	20.02	86.52	0.029687

Crest | **Head =** 20.02 | **Pr(f)=0** NO

Run	Levee Φ	Levee γ	Foundation Cohesion	Foundation g	Foundation Φ	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.29		
2	25	120	150	120	31	1.22	0.000576	2.01
3	33	120	150	120	31	1.27		
4	29	72	150	120	31	1.30	0.003025	10.58
5	29	168	150	120	31	1.19		
6	29	120	139	120	31	1.23	0.000110	0.39
7	29	120	161	120	31	1.21		
8	29	120	150	104	31	1.33	0.013225	46.24
9	29	120	150	136	31	1.10		
10	29	120	150	120	19	1.36	0.011664	40.78
11	29	120	150	120	43	1.14		

E[FS] = 1.290000 | E[ln FS] = 0.246122 | Total 0.028600 100.00
Var[FS]= 0.028600
σ[FS]= 0.169116 | σ[ln FS]= 0.130540 | β = 1.885419
V(FS) = 0.131098 | F(z) = 0.029687
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 2.968665

Crest-3ft | **Head =** 17.02 | **Pr(f)=0** YES

Run	Levee Φ	Levee γ	Foundation Cohesion	Foundation g	Foundation Φ	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.25		
2	25	120	150	120	31	1.35	0.009604	90.64
3	33	120	150	120	31	1.16		
4	29	72	150	120	31	1.24	0.000144	1.36
5	29	168	150	120	31	1.26		
6	29	120	139	120	31	1.25	0.000784	7.40
7	29	120	161	120	31	1.19		
8	29	120	150	104	31	1.25	0.000064	0.60
9	29	120	150	136	31	1.23		
10	29	120	150	120	19	1.25	0.000000	0.00
11	29	120	150	120	43	1.25		

E[FS] = | E[ln FS] = | Total 0.010596 100.00
Var[FS]= 0.010596
σ[FS]= 0.102937 | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Half Height | **Head =** 10.01 | **Pr(f)=0** YES

Run	Levee Φ	Levee γ	Foundation Cohesion	Foundation g	Foundation Φ	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	25	120	150	120	31			
3	33	120	150	120	31			
4	29	72	150	120	31			
5	29	168	150	120	31			
6	29	120	139	120	31			
7	29	120	161	120	31			
8	29	120	150	104	31			
9	29	120	150	136	31			
10	29	120	150	120	19			
11	29	120	150	120	43			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee Φ	Levee γ	Foundation Cohesion	Foundation g	Foundation Φ	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	25	120	150	120	31			
3	33	120	150	120	31			
4	29	72	150	120	31			
5	29	168	150	120	31			
6	29	120	139	120	31			
7	29	120	161	120	31			
8	29	120	150	104	31			
9	29	120	150	136	31			
10	29	120	150	120	19			
11	29	120	150	120	43			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

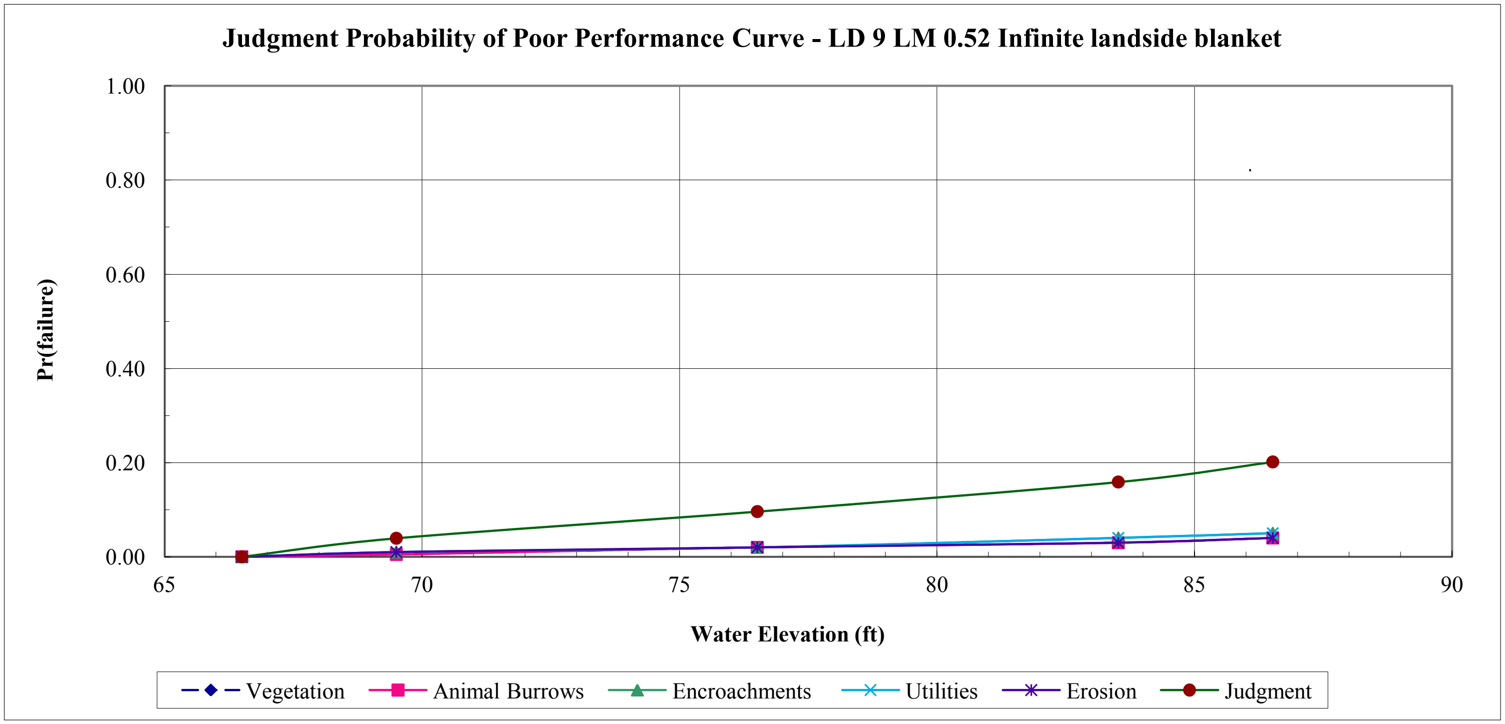
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 9

Levee Mile: 0.52
River Mile: 2188213.88 N
Analysis Case: Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/12/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
66.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
69.50	0.0050	0.9950	0.0050	0.9950	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0394	0.9606
76.51	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0961	0.9039
83.52	0.0300	0.9700	0.0300	0.9700	0.0400	0.9600	0.0400	0.9600	0.0300	0.9700	0.1589	0.8411
86.52	0.0400	0.9600	0.0400	0.9600	0.0500	0.9500	0.0500	0.9500	0.0400	0.9600	0.2015	0.7985



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

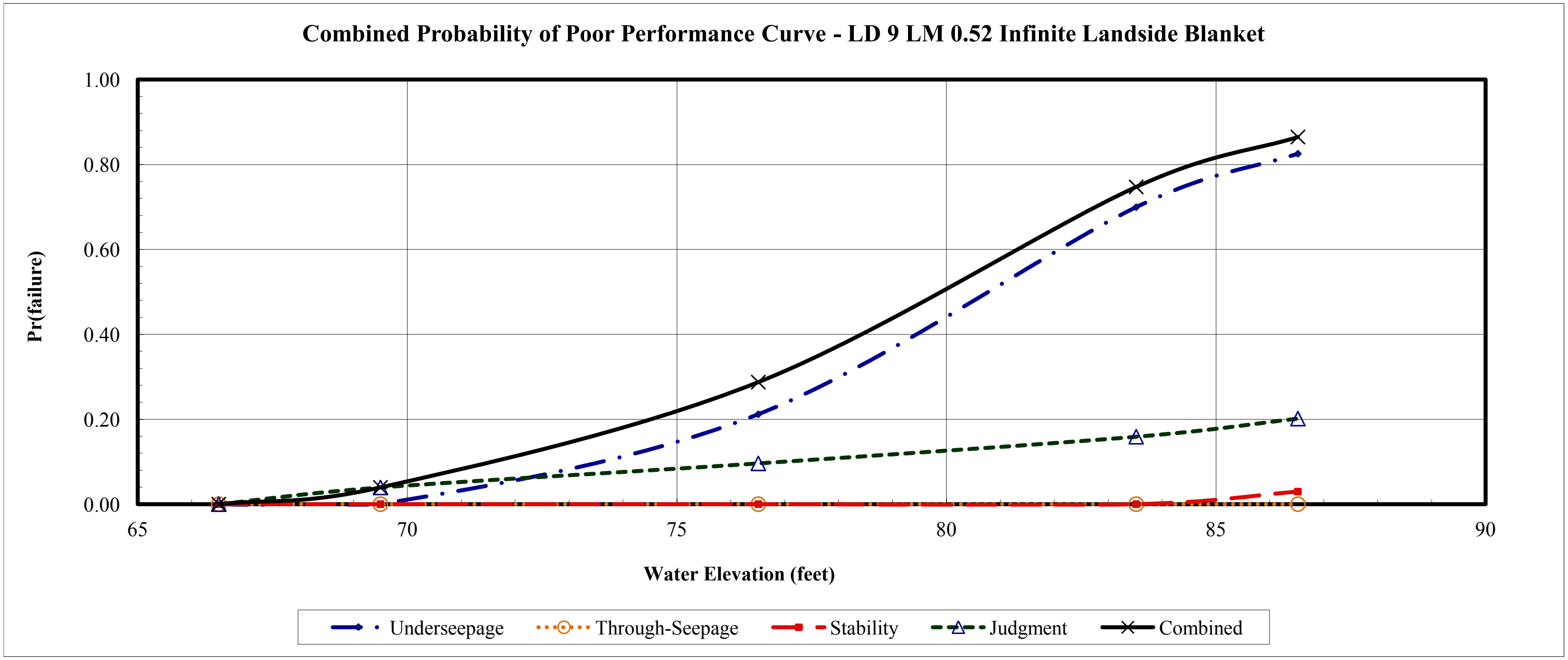
Project: Sutter Feasibility Study
Study Area: Feather River South
River Section: LD 9

Levee Mile: 0.52
River Mile: 2188213.88 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 86.52
L/S Toe Elev.: 66.50
W/S Toe Elev.: 58.90

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/12/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
66.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
69.50	0.0001	0.9999	0.0000	1.0000	0.0000	1.0000	0.0394	0.9606	0.0394	0.9606
76.51	0.2117	0.7883	0.0000	1.0000	0.0000	1.0000	0.0961	0.9039	0.2875	0.7125
83.52	0.6995	0.3005	0.0000	1.0000	0.0000	1.0000	0.1589	0.8411	0.7473	0.2527
86.52	0.8254	0.1746	0.0000	1.0000	0.0297	0.9703	0.2015	0.7985	0.8647	0.1353



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River North
Basin and Reach: MA 16

Levee Mile: 0.90
Coordinates: 2224154.37 N; 6664999.34 E
Analysis Case Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WM0016_011B	5	6	5	24	76	9.5	16	8	70	53	CL-ML	0.007	SP-SM/SP	25.79	3684	2633	1742	3033812	66
WM0016_010C	15					30					CL-ML	0.007	SP	28	4000				
WM0016_012C	5					10					CL-ML	0.007	SP	28	4000				
WM0016_009C	3					14					CL-ML	0.0284	SP	28	986				
WM0016_001A	4					15					CL	0.0284	SW-SM	14	493				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WM0016_011B	CL-ML	5	0.007				5	SP-SM	1.5	14	SP	8	28				25.79
WM0016_010C	CL-ML	18	0.007				18	SP	30	28							28
WM0016_012C	CL-ML	5	0.007				5	SP	10	28							28
WM0016_009C	CL-ML	3	0.0284				3	SP	14	28							28
WM0016_001A	CL	4	0.0284				4	SW-SM	15	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

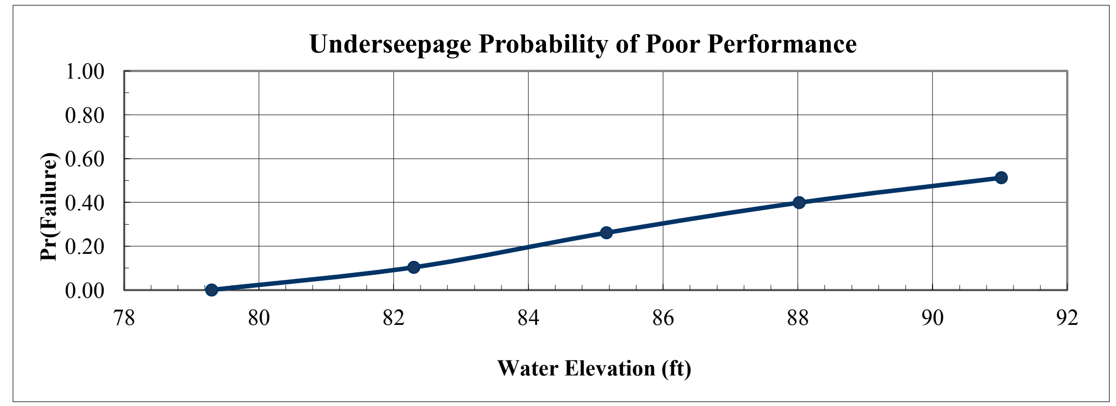
Levee Mile: 0.90
River Mile: 2224154.37 N; 6664999.34 E
Analysis Case Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	2633	1742	66
Blanket Thickness (z)	6	5	76
Aquifer Thickness (d)	16	8	53

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	15	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	79.30	0.0000
Toe+3ft	3.00	82.30	0.1036
Half Height	5.86	85.16	0.2614
Crest-3ft	8.72	88.02	0.3990
Crest	11.72	91.02	0.5127

Crest	Rh
Head = 11.72	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2633	6.40	15.70	15.00	514.32	0.0238	9.14	1.43		
2	4374	6.40	15.70	15.00	662.98	0.0194	9.62	1.50	0.018225	0.45
3	891	6.40	15.70	14.99	299.19	0.0353	7.89	1.23		
4	2633	11.28	15.70	15.00	682.76	0.0190	9.67	0.86	4.040100	99.33
5	2633	1.52	15.70	14.98	250.77	0.0397	7.43	4.88		
6	2633	6.40	24.05	15.00	636.55	0.0308	9.55	1.49	0.009025	0.22
7	2633	6.40	7.35	14.99	351.94	0.0148	8.30	1.30		
								Total	4.067350	100.00

E[I] = 1.430000 E[ln I] = -0.189799
Var[I]= 4.067350
σ[I]= 2.016767 σ [ln I] = 1.046397
V(I) = 1.410327

β =	-0.181383
F(z) =	0.487289
Pr(f) % =	51.271073

Ic= 0.80 ln(I crit) = -0.223144

Crest-3ft	Rh
Head = 8.72	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2633	6.40	15.70	15.00	514.32	0.0238	6.80	1.06		
2	4374	6.40	15.70	15.00	662.98	0.0194	7.16	1.12	0.010000	0.44
3	891	6.40	15.70	14.99	299.19	0.0353	5.87	0.92		
4	2633	11.28	15.70	15.00	682.76	0.0190	7.19	0.64	2.235025	99.34
5	2633	1.52	15.70	14.98	250.77	0.0397	5.53	3.63		
6	2633	6.40	24.05	15.00	636.55	0.0308	7.10	1.11	0.004900	0.22
7	2633	6.40	7.35	14.99	351.94	0.0148	6.18	0.97		
								Total	2.249925	100.00

E[I] = 1.060000 E[ln I] = -0.491441
Var[I]= 2.249925
σ[I]= 1.499975 σ [ln I] = 1.048532
V(I) = 1.415071

β =	-0.468694
F(z) =	0.600978
Pr(f) % =	39.902203

Ic= 0.80 ln(I crit) = -0.223144

Half Height	Rh
Head = 5.86	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2633	6.40	15.70	15.00	514.32	0.0238	4.57	0.71		
2	4374	6.40	15.70	15.00	662.98	0.0194	4.81	0.75	0.004225	0.42
3	891	6.40	15.70	14.99	299.19	0.0353	3.95	0.62		
4	2633	11.28	15.70	15.00	682.76	0.0190	4.83	0.43	1.010025	99.34
5	2633	1.52	15.70	14.98	250.77	0.0397	3.71	2.44		
6	2633	6.40	24.05	15.00	636.55	0.0308	4.77	0.75	0.002500	0.25
7	2633	6.40	7.35	14.99	351.94	0.0148	4.15	0.65		
								Total	1.016750	100.00

E[I] = 0.710000 E[ln I] = -0.894615
Var[I]= 1.016750
σ[I]= 1.008340 σ [ln I] = 1.050833
V(I) = 1.420197

β =	-0.851339
F(z) =	0.738585
Pr(f) % =	26.141473

Ic= 0.80 ln(I crit) = -0.223144

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	2633	6.40	15.70	15.00	514.32	0.0238	2.34	0.37		
2	4374	6.40	15.70	15.00	662.98	0.0194	2.46	0.38	0.000900	0.34
3	891	6.40	15.70	14.99	299.19	0.0353	2.02	0.32		
4	2633	11.28	15.70	15.00	682.76	0.0190	2.47	0.22	0.265225	99.43
5	2633	1.52	15.70	14.98	250.77	0.0397	1.90	1.25		
6	2633	6.40	24.05	15.00	636.55	0.0308	2.44	0.38	0.000625	0.23
7	2633	6.40	7.35	14.99	351.94	0.0148	2.12	0.33		
								Total	0.266750	100.00

E[I] = 0.370000 E[ln I] = -1.534901
Var[I]= 0.266750
σ[I]= 0.516478 σ [ln I] = 1.039855
V(I) = 1.395888

β =	-1.476073
F(z) =	0.896432
Pr(f) % =	10.356768

Ic= 0.80 ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

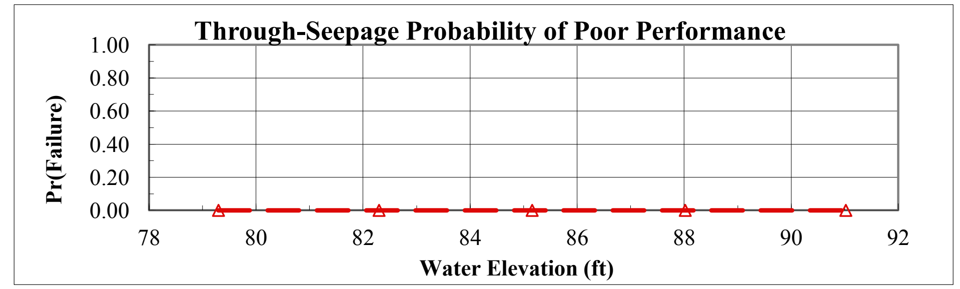
Levee Mile: 0.90
River Mile: 2224154.37 N; 6664999.34 E
Analysis Case Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	15	1.5	10.00
Initial Porosity (n)	45	4.50	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	79.30	0.0000
Toe+3ft	3.00	82.30	0.000000
Half Height	5.86	85.16	0.000000
Crest-3ft	8.72	88.02	0.000000
Crest	11.72	91.02	0.000000

Crest	**Head =**	11.72	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft	**Head =**	8.72	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height	**Head =**	5.86	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft	**Head =**	3.00	**Horizontal Gradient (Ix) =**	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	15.00	45.00	1.00E-10	3628.12			
2	13.50	45.00	1.00E-10	3265.31			
3	16.50	45.00	1.00E-10	3990.93			
4	15.00	40.50	1.00E-10	3441.94			
5	15.00	49.50	1.00E-10	3805.21			
6	15.00	45.00	7.00E-11	4336.44			
7	15.00	45.00	1.30E-10	3182.07			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

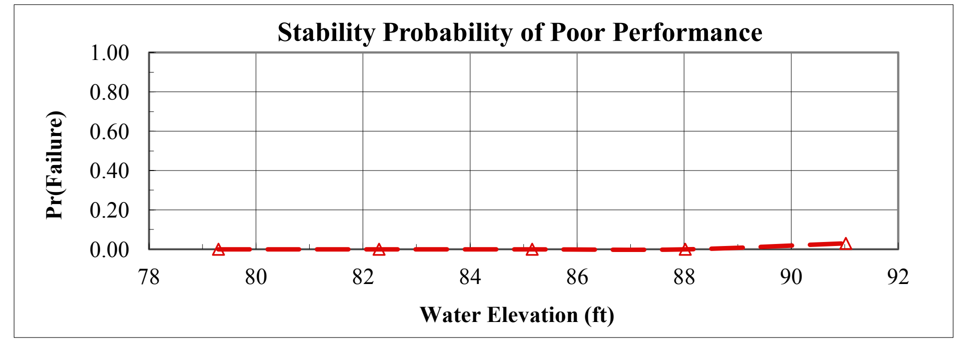
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 0.90
River Mile: 2224154.37 N; 6664999.34 E
Analysis Case Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Levee ϕ'	31	4	12.00
Levee γ	125	6	5.00
Foundation c'	150	50	33.00
Foundation γ	115	6	5.00
Foundation ϕ'	28	3	12.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	79.30	0.0000
Toe+3ft	3.00	82.30	0.000000
Half Height	5.86	85.16	0.000000
Crest-3ft	8.72	88.02	0.000000
Crest	11.72	91.02	0.029687

Crest	Head =	11.72	Pr(f)=0	NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.29		
2	27	125	150	115	28	1.22	0.000576	2.01
3	35	125	150	115	28	1.27		
4	31	119	150	115	28	1.30	0.003025	10.58
5	31	131	150	115	28	1.19		
6	31	125	100	115	28	1.23	0.000110	0.39
7	31	125	200	115	28	1.21		
8	31	125	150	109	28	1.33	0.013225	46.24
9	31	125	150	121	28	1.10		
10	31	125	150	115	25	1.36	0.011664	40.78
11	31	125	150	115	31	1.14		
E[FS] =	1.290000				E[ln FS] =	0.246122	Total 0.028600	100.00

Var[FS]=	0.028600				
σ[FS]=	0.169116	σ[ln FS]=	0.130540	β =	1.885419
V(FS) =	0.131098			F(z) =	0.029687
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	2.968665

Crest-3ft	Head =	8.72	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28	1.25		
2	27	125	150	115	28	1.35	0.009604	90.64
3	35	125	150	115	28	1.16		
4	31	119	150	115	28	1.24	0.000144	1.36
5	31	131	150	115	28	1.26		
6	31	125	100	115	28	1.25	0.000784	7.40
7	31	125	200	115	28	1.19		
8	31	125	150	109	28	1.25	0.000064	0.60
9	31	125	150	121	28	1.23		
10	31	125	150	115	25	1.25	0.000000	0.00
11	31	125	150	115	31	1.25		
E[FS] =					E[ln FS] =		Total 0.010596	100.00

Var[FS]=	0.010596				
σ[FS]=	0.102937	σ[ln FS]=		β =	
V(FS) =				F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	0.000000

Half Height	Head =	5.86	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			
E[FS] =					E[ln FS] =		Total	

Var[FS]=					
σ[FS]=		σ[ln FS]=		β =	
V(FS) =				F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	0.000000

Toe+3ft	Head =	3.00	Pr(f)=0	YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	31	125	150	115	28			
2	27	125	150	115	28			
3	35	125	150	115	28			
4	31	119	150	115	28			
5	31	131	150	115	28			
6	31	125	100	115	28			
7	31	125	200	115	28			
8	31	125	150	109	28			
9	31	125	150	121	28			
10	31	125	150	115	25			
11	31	125	150	115	31			
E[FS] =					E[ln FS] =		Total	

Var[FS]=					
σ[FS]=		σ[ln FS]=		β =	
V(FS) =				F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

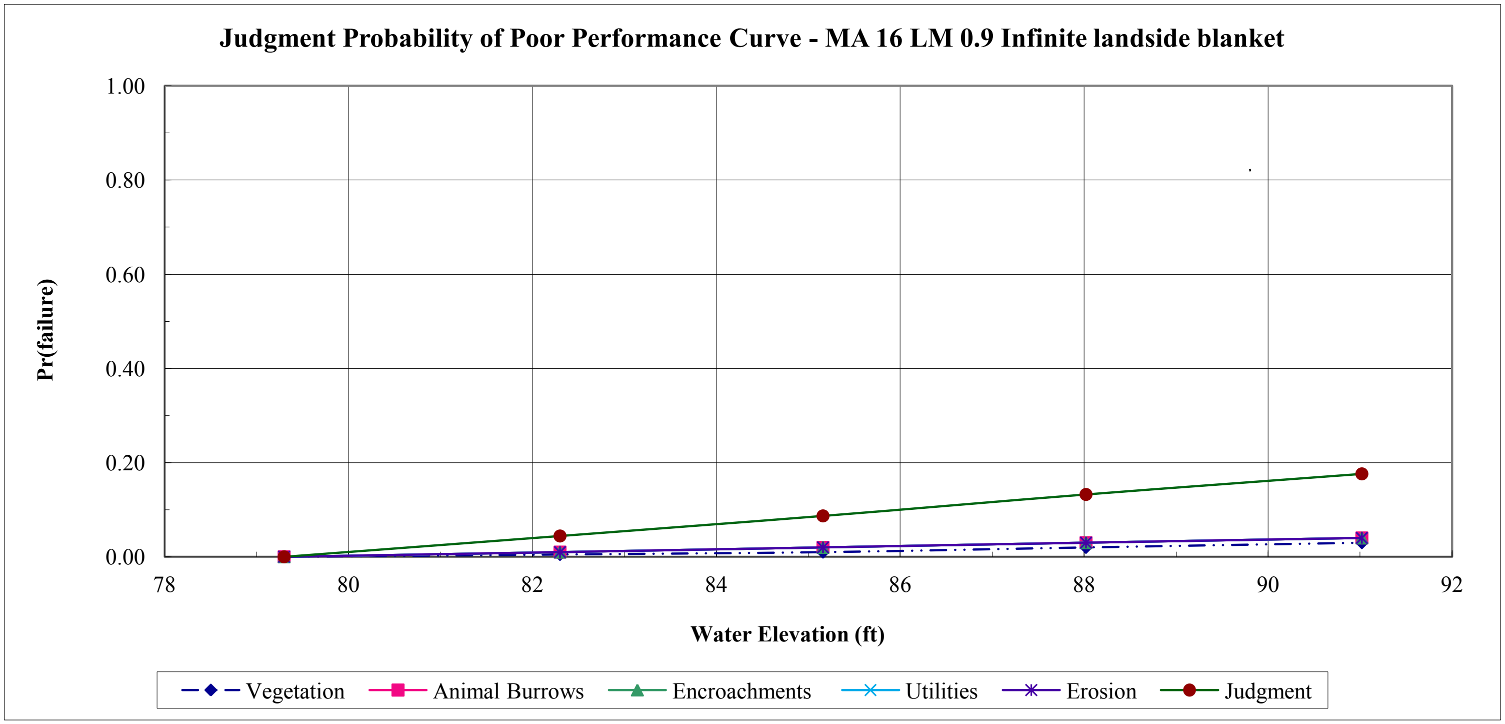
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 0.90
River Mile: 2224154.37 N
Analysis Case: Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/12/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
79.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
82.30	0.0050	0.9950	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0442	0.9558
85.16	0.0100	0.9900	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0200	0.9800	0.0869	0.9131
88.02	0.0200	0.9800	0.0300	0.9700	0.0300	0.9700	0.0300	0.9700	0.0300	0.9700	0.1324	0.8676
91.02	0.0300	0.9700	0.0400	0.9600	0.0400	0.9600	0.0400	0.9600	0.0400	0.9600	0.1761	0.8239



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

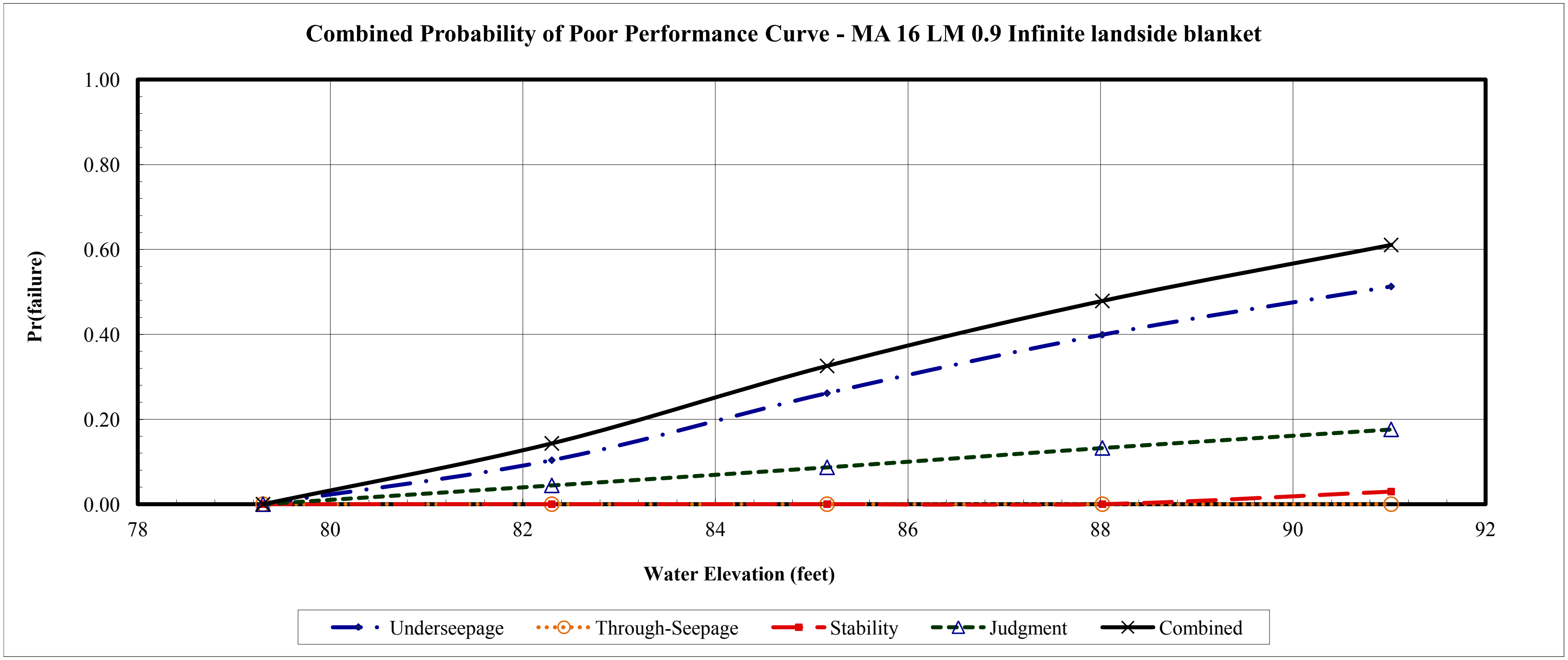
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 0.90
River Mile: 2224154.37 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 91.02
L/S Toe Elev.: 79.30
W/S Toe Elev.: 77.30

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/12/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
79.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
82.30	0.1036	0.8964	0.0000	1.0000	0.0000	1.0000	0.0442	0.9558	0.1432	0.8568
85.16	0.2614	0.7386	0.0000	1.0000	0.0000	1.0000	0.0869	0.9131	0.3256	0.6744
88.02	0.3990	0.6010	0.0000	1.0000	0.0000	1.0000	0.1324	0.8676	0.4786	0.5214
91.02	0.5127	0.4873	0.0000	1.0000	0.0297	0.9703	0.1761	0.8239	0.6105	0.3895



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River North
Basin and Reach: MA 16

Levee Mile: 2.90
Coordinates: 2233626.25 N; 6664328.54 E
Analysis Case Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material	Aquifer Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WM0016_011B	9	13	5	26	41	10	12	5	29	45	CL-ML	0.28	SP	28	100	1053	1603	2569602	98
WM0016_014B	20					18					CL-ML	0.0071	SW, SM, SC	14	1972				
WM0016_018B	10					12					SC, SM	0.14	SP,SW,SM	28	200				
WM0016_019S	18					18					CL, ML	0.0071	SP	28	3944				
WM0016_020B	10					10					CL, ML	0.28	SW-SM	14	50				
WM0016_022S	8					4					CL	0.28	SP-SM	14	50				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)		Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)			Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WM0016_011B	CL-ML	9	0.28				9	9	SP	10	28							28
WM0016_014B	CL-ML	20	0.0071				20	20	SW, SM, SC	18	14							14
WM0016_018B	SC, SM	10	0.14				10	10	SP,SW,SM	12	28							28
WM0016_019S	CL, ML	18	0.0071				18	18	SP	18	28							28
WM0016_020B	CL, ML	10	0.28				10	10	SW-SM	10	14							14
WM0016_022S	CL	8	0.28				10	10	SP-SM	3	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

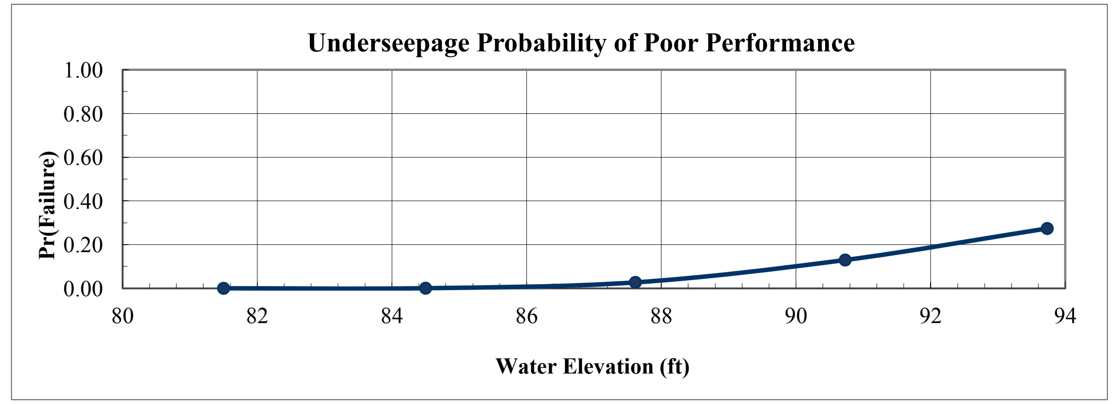
Levee Mile: 2.90
River Mile: 2233626.25 N; 6664328.54 E
Analysis Case Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	1053	1032	98
Blanket Thickness (z)	13	5	41
Aquifer Thickness (d)	12	5	45

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	81.50	0.0000
Toe+3ft	3.00	84.50	0.0005
Half Height	6.12	87.62	0.0271
Crest-3ft	9.23	90.73	0.1294
Crest	12.23	93.73	0.2738

Crest	Rh
Head = 12.23	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1053	12.50	12.00	49.74	397.35	0.0208	8.42	0.67		
2	2084	12.50	12.00	49.87	559.12	0.0162	9.25	0.74	0.062500	45.32
3	21	12.50	12.00	39.97	56.19	0.0531	3.04	0.24		
4	1053	17.63	12.00	49.81	471.87	0.0184	8.86	0.50	0.072900	52.86
5	1053	7.37	12.00	49.56	305.14	0.0248	7.70	1.04		
6	1053	12.50	17.37	49.82	478.01	0.0264	8.89	0.71	0.002500	1.81
7	1053	12.50	6.63	49.53	295.43	0.0140	7.61	0.61		
								Total	0.137900	100.00

E[I] = 0.670000
Var[I]= 0.137900
σ[I]= 0.371349
V(I) = 0.554252

E[ln I] = -0.534420
σ [ln I] = 0.517575

β =	-1.032545
F(z) =	0.726217
Pr(f) % =	27.378272

Ic=	0.80

ln(I crit) = -0.223144

Crest-3ft	Rh
Head = 9.23	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1053	12.50	12.00	49.74	397.35	0.0208	6.36	0.51		
2	2084	12.50	12.00	49.87	559.12	0.0162	6.98	0.56	0.036100	45.28
3	21	12.50	12.00	39.97	56.19	0.0531	2.29	0.18		
4	1053	17.63	12.00	49.81	471.87	0.0184	6.68	0.38	0.042025	52.71
5	1053	7.37	12.00	49.56	305.14	0.0248	5.81	0.79		
6	1053	12.50	17.37	49.82	478.01	0.0264	6.71	0.54	0.001600	2.01
7	1053	12.50	6.63	49.53	295.43	0.0140	5.74	0.46		
								Total	0.079725	100.00

E[I] = 0.510000
Var[I]= 0.079725
σ[I]= 0.282356
V(I) = 0.553640

E[ln I] = -0.807027
σ [ln I] = 0.517073

β =	-1.560760
F(z) =	0.870595
Pr(f) % =	12.940497

Ic=	0.80

ln(I crit) = -0.223144

Half Height	Rh
Head = 6.12	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1053	12.50	12.00	49.74	397.35	0.0208	4.21	0.34		
2	2084	12.50	12.00	49.87	559.12	0.0162	4.63	0.37	0.015625	44.96
3	21	12.50	12.00	39.97	56.19	0.0531	1.52	0.12		
4	1053	17.63	12.00	49.81	471.87	0.0184	4.43	0.25	0.018225	52.45
5	1053	7.37	12.00	49.56	305.14	0.0248	3.85	0.52		
6	1053	12.50	17.37	49.82	478.01	0.0264	4.44	0.36	0.000900	2.59
7	1053	12.50	6.63	49.53	295.43	0.0140	3.80	0.30		
								Total	0.034750	100.00

E[I] = 0.340000
Var[I]= 0.034750
σ[I]= 0.186414
V(I) = 0.548275

E[ln I] = -1.210225
σ [ln I] = 0.512669

β =	-2.360633
F(z) =	0.972909
Pr(f) % =	2.709121

Ic=	0.80

ln(I crit) = -0.223144

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1053	12.50	12.00	49.74	397.35	0.0208	2.07	0.17		
2	2084	12.50	12.00	49.87	559.12	0.0162	2.27	0.18	0.003600	41.86
3	21	12.50	12.00	39.97	56.19	0.0531	0.75	0.06		
4	1053	17.63	12.00	49.81	471.87	0.0184	2.17	0.12	0.004900	56.98
5	1053	7.37	12.00	49.56	305.14	0.0248	1.89	0.26		
6	1053	12.50	17.37	49.82	478.01	0.0264	2.18	0.17	0.000100	1.16
7	1053	12.50	6.63	49.53	295.43	0.0140	1.87	0.15		
								Total	0.008600	100.00

E[I] = 0.170000
Var[I]= 0.008600
σ[I]= 0.092736
V(I) = 0.545507

E[ln I] = -1.902207
σ [ln I] = 0.510391

β =	-3.726957
F(z) =	0.999499
Pr(f) % =	0.050137

Ic=	0.80

ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

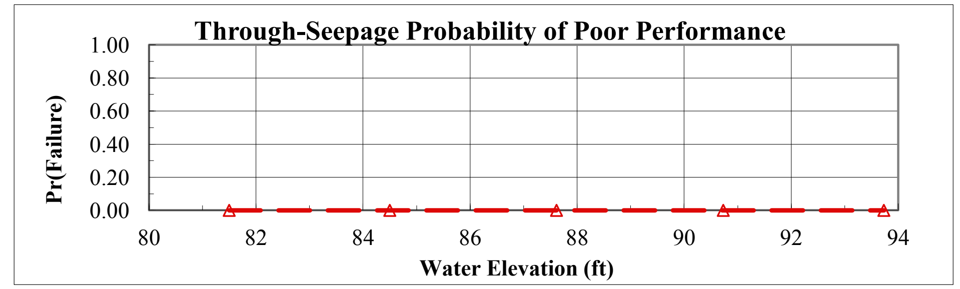
Levee Mile: 2.90
River Mile: 2233626.25 N; 6664328.54 E
Analysis Case Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	81.50	0.0000
Toe+3ft	3.00	84.50	0.000000
Half Height	6.12	87.62	0.000000
Crest-3ft	9.23	90.73	0.000000
Crest	12.23	93.73	0.000000

Crest | **Head =** 12.23 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 9.23 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height | **Head =** 6.12 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

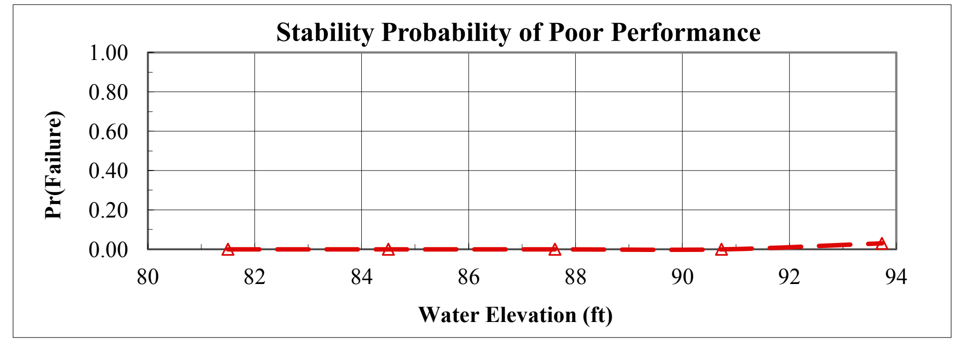
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 2.90
River Mile: 2233626.25 N; 6664328.54 E
Analysis Case Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/12/2012

Random Variables				
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %	
Levee ϕ'	29	1	5.00	
Levee γ	120	6	5.00	
Foundation c'	150	17	11.50	
Foundation γ	120	6	5.00	
Foundation ϕ'	31	4	11.50	



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	81.50	0.0000
Toe+3ft	3.00	84.50	0.000000
Half Height	6.12	87.62	0.000000
Crest-3ft	9.23	90.73	0.000000
Crest	12.23	93.73	0.029687

Crest | **Head =** 12.23 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.29		
2	28	120	150	120	31	1.22	0.000576	2.01
3	30	120	150	120	31	1.27		
4	29	114	150	120	31	1.30	0.003025	10.58
5	29	126	150	120	31	1.19		
6	29	120	133	120	31	1.23	0.000110	0.39
7	29	120	167	120	31	1.21		
8	29	120	150	114	31	1.33	0.013225	46.24
9	29	120	150	126	31	1.10		
10	29	120	150	120	27	1.36	0.011664	40.78
11	29	120	150	120	35	1.14		

E[FS] = 1.290000 | E[ln FS] = 0.246122 | Total 0.028600 100.00
Var[FS]= 0.028600
σ[FS]= 0.169116 | σ[ln FS]= 0.130540 | β = 1.885419
V(FS) = 0.131098 | F(z) = 0.029687
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 2.968665

Crest-3ft | **Head =** 9.23 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.25		
2	28	120	150	120	31	1.35	0.009604	90.64
3	30	120	150	120	31	1.16		
4	29	114	150	120	31	1.24	0.000144	1.36
5	29	126	150	120	31	1.26		
6	29	120	133	120	31	1.25	0.000784	7.40
7	29	120	167	120	31	1.19		
8	29	120	150	114	31	1.25	0.000064	0.60
9	29	120	150	126	31	1.23		
10	29	120	150	120	27	1.25	0.000000	0.00
11	29	120	150	120	35	1.25		

E[FS] = | E[ln FS] = | Total 0.010596 100.00
Var[FS]= 0.010596
σ[FS]= 0.102937 | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Half Height | **Head =** 6.12 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | β =
V(FS) = | F(z) =
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

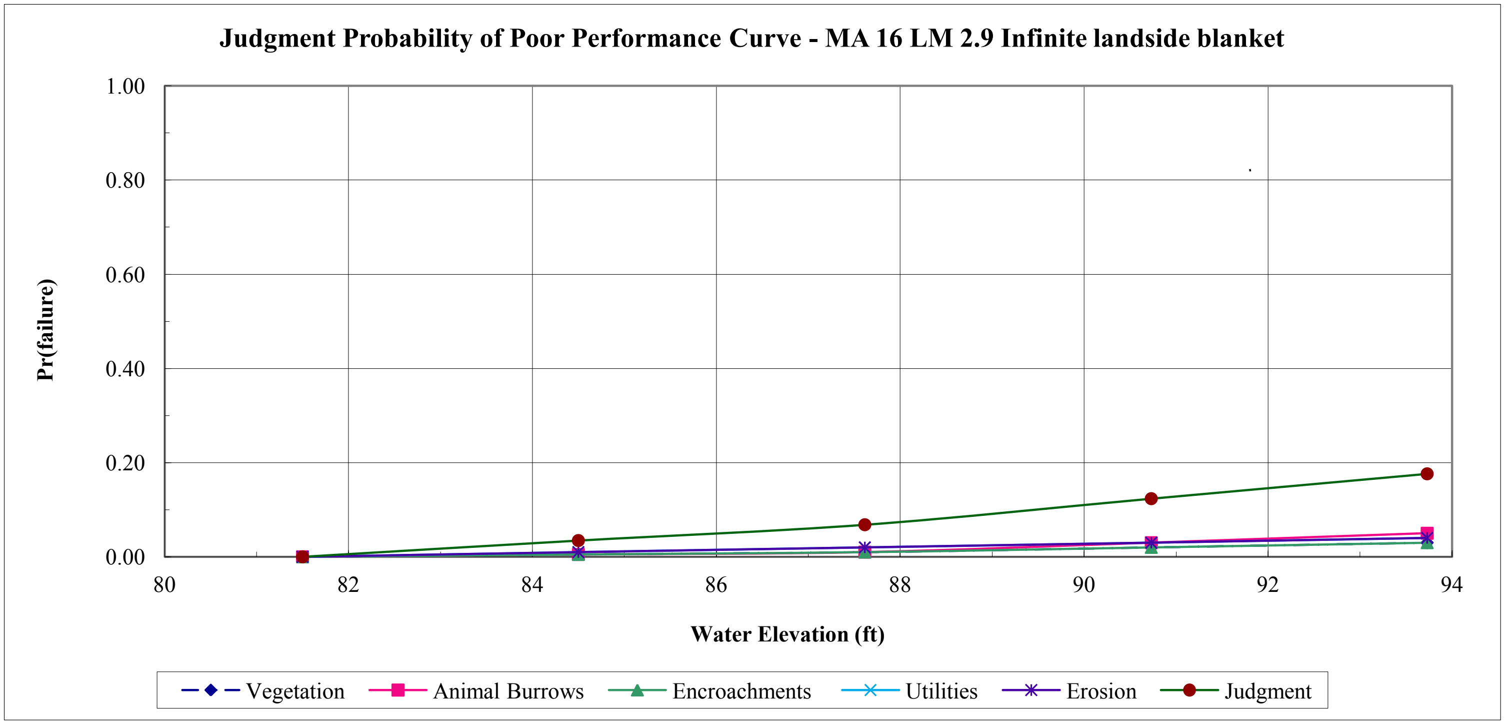
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 2.90
River Mile: 2233626.25 N
Analysis Case: Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/12/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
81.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
84.50	0.0050	0.9950	0.0050	0.9950	0.0050	0.9950	0.0100	0.9900	0.0100	0.9900	0.0345	0.9655
87.62	0.0100	0.9900	0.0100	0.9900	0.0100	0.9900	0.0200	0.9800	0.0200	0.9800	0.0681	0.9319
90.73	0.0200	0.9800	0.0300	0.9700	0.0200	0.9800	0.0300	0.9700	0.0300	0.9700	0.1235	0.8765
93.73	0.0300	0.9700	0.0500	0.9500	0.0300	0.9700	0.0400	0.9600	0.0400	0.9600	0.1762	0.8238



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

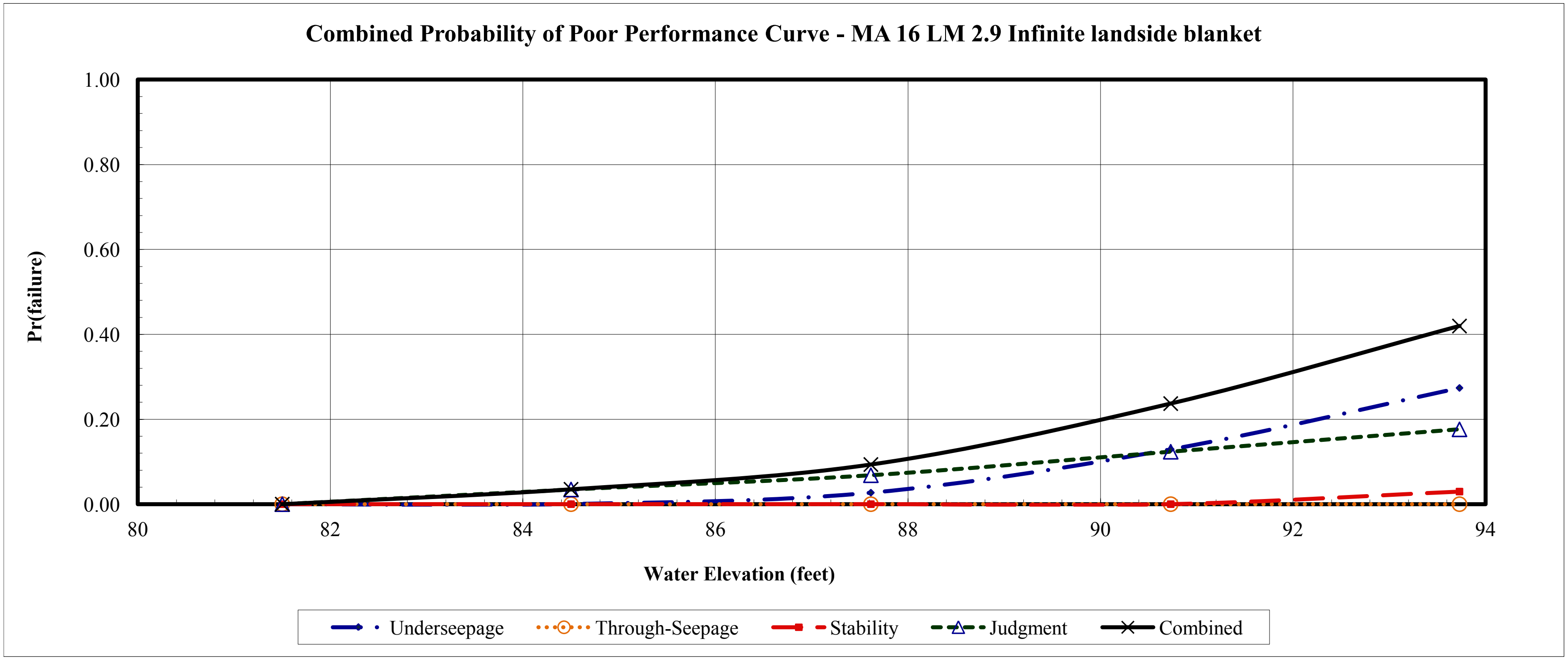
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: MA 16

Levee Mile: 2.90
River Mile: 2233626.25 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 93.73
L/S Toe Elev.: 81.50
W/S Toe Elev.: 79.40

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/12/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
81.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
84.50	0.0005	0.9995	0.0000	1.0000	0.0000	1.0000	0.0345	0.9655	0.0350	0.9650
87.62	0.0271	0.9729	0.0000	1.0000	0.0000	1.0000	0.0681	0.9319	0.0934	0.9066
90.73	0.1294	0.8706	0.0000	1.0000	0.0000	1.0000	0.1235	0.8765	0.2369	0.7631
93.73	0.2738	0.7262	0.0000	1.0000	0.0297	0.9703	0.1762	0.8238	0.4195	0.5805



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Feather River North
Basin and Reach: Hamilton Bend (MA 7)

Levee Mile: 0.51
Coordinates: 2288660.96 N; 6662820.24 E
Analysis Case Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WM0007_069B	10	12	3	10	26	31.5	30	21	482	70	sML	0.14	GP-GM, GP,GV	560	4000	11742	20103	306195806	98
WM0007_068S	9.02					49					L, CL-MLs/SP-	0.007	M,GW, GP-GM	70	10000				
WM0007_013S	8.71					60.5					CL-ML,sCL/MI	0.01	GW,GP,GC,SM	560	56000				
WM0007_067B/S	15					10					sML/SM	0.14	GP-GM	35	250				
WM0007_66S	16					13					ML, SC/CL-MI	0.14	SP	14	100				
WM0007_65B/S	13.3					15.5					sML/SM	0.14	W, SP-SM,GP-	14	100				

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Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WM0007_069B	sML	10	0.14				10	GP-GM, GP,GV	31.5	560							560
WM0007_068S	sML, CL-MLs	9	0.007	SP-SM	2	0.7	9.02	M,GW, GP-GM	49	70							70
WM0007_013S	CL-ML,sCL	8.5	0.01	ML	3	0.14	8.71	GW,GP,GC,SM	60.5	560							560
WM0007_067B/S	sML	11	0.14	SM	8	0.28	15	GP-GM	10	35							35
WM0007_66S	sML, SC	9	0.14	CL-ML	7	0.14	16	SP	13	14							14
WM0007_65B/S	sML	12	0.14	SM	6.5	0.7	13.3	W, SP-SM,GP-	15.5	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: Hamilton Bend (MA 7)

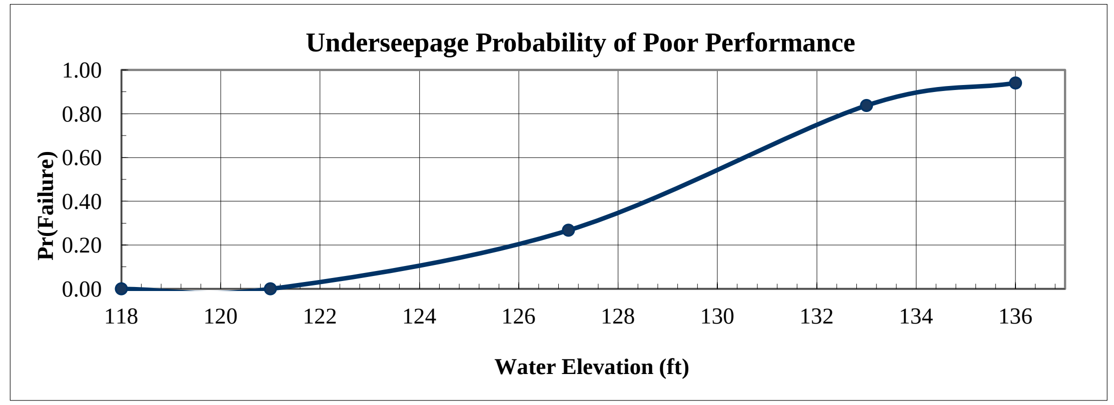
Levee Mile: 0.51
River Mile: 2288660.96 N; 6662820.24 E
Analysis Case Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	11742	11507	98
Blanket Thickness (z)	12	3	26
Aquifer Thickness (d)	30	21	70

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	50	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	118.00	0.0000
Toe+3ft	3.00	121.00	0.0000
Half Height	9.00	127.00	0.2678
Crest-3ft	15.00	133.00	0.8376
Crest	18.00	136.00	0.9405

Crest	Rh
Head = 18.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	11742	12.01	29.92	49.99	2053.54	0.0134	16.55	1.38		
2	23249	12.01	29.92	50.00	2889.58	0.0097	16.94	1.41	0.057600	28.54
3	235	12.01	29.92	49.51	290.41	0.0637	11.12	0.93		
4	11742	15.18	29.92	49.99	2309.16	0.0120	16.70	1.10	0.140625	69.68
5	11742	8.83	29.92	49.99	1761.20	0.0154	16.33	1.85		
6	11742	12.01	50.86	49.99	2677.48	0.0178	16.87	1.41	0.003600	1.78
7	11742	12.01	8.98	49.97	1124.79	0.0069	15.52	1.29		
								Total	0.201825	100.00

E[I] = 1.380000
Var[I]= 0.201825
σ[I]= 0.449249
V(I) = 0.325543

E[ln I] = 0.271718
σ [ln I] = 0.317380

β = 0.856129
F(z) = 0.059474
Pr(f) % = 94.052644

Ic= 0.80
ln(I crit) = -0.223144

Crest-3ft	Rh
Head = 15.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	11742	12.01	29.92	49.99	2053.54	0.0134	13.79	1.15		
2	23249	12.01	29.92	50.00	2889.58	0.0097	14.12	1.18	0.042025	29.99
3	235	12.01	29.92	49.51	290.41	0.0637	9.27	0.77		
4	11742	15.18	29.92	49.99	2309.16	0.0120	13.92	0.92	0.096100	68.57
5	11742	8.83	29.92	49.99	1761.20	0.0154	13.61	1.54		
6	11742	12.01	50.86	49.99	2677.48	0.0178	14.06	1.17	0.002025	1.44
7	11742	12.01	8.98	49.97	1124.79	0.0069	12.93	1.08		
								Total	0.140150	100.00

E[I] = 1.150000
Var[I]= 0.140150
σ[I]= 0.374366
V(I) = 0.325536

E[ln I] = 0.089399
σ [ln I] = 0.317374

β = 0.281684
F(z) = 0.162367
Pr(f) % = 83.763344

Ic= 0.80
ln(I crit) = -0.223144

Half Height	Rh
Head = 9.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	11742	12.01	29.92	49.99	2053.54	0.0134	8.27	0.69		
2	23249	12.01	29.92	50.00	2889.58	0.0097	8.47	0.71	0.015625	29.85
3	235	12.01	29.92	49.51	290.41	0.0637	5.56	0.46		
4	11742	15.18	29.92	49.99	2309.16	0.0120	8.35	0.55	0.036100	68.96
5	11742	8.83	29.92	49.99	1761.20	0.0154	8.17	0.93		
6	11742	12.01	50.86	49.99	2677.48	0.0178	8.43	0.70	0.000625	1.19
7	11742	12.01	8.98	49.97	1124.79	0.0069	7.76	0.65		
								Total	0.052350	100.00

E[I] = 0.690000
Var[I]= 0.052350
σ[I]= 0.228801
V(I) = 0.331596

E[ln I] = -0.423224
σ [ln I] = 0.322986

β = -1.310345
F(z) = 0.732196
Pr(f) % = 26.780357

Ic= 0.80
ln(I crit) = -0.223144

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	11742	12.01	29.92	49.99	2053.54	0.0134	2.76	0.23		
2	23249	12.01	29.92	50.00	2889.58	0.0097	2.82	0.23	0.001600	27.35
3	235	12.01	29.92	49.51	290.41	0.0637	1.85	0.15		
4	11742	15.18	29.92	49.99	2309.16	0.0120	2.78	0.18	0.004225	72.22
5	11742	8.83	29.92	49.99	1761.20	0.0154	2.72	0.31		
6	11742	12.01	50.86	49.99	2677.48	0.0178	2.81	0.23	0.000025	0.43
7	11742	12.01	8.98	49.97	1124.79	0.0069	2.59	0.22		
								Total	0.005850	100.00

E[I] = 0.230000
Var[I]= 0.005850
σ[I]= 0.076485
V(I) = 0.332545

E[ln I] = -1.522120
σ [ln I] = 0.323864

β = -4.699875
F(z) = 0.999970
Pr(f) % = 0.003025

Ic= 0.80
ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

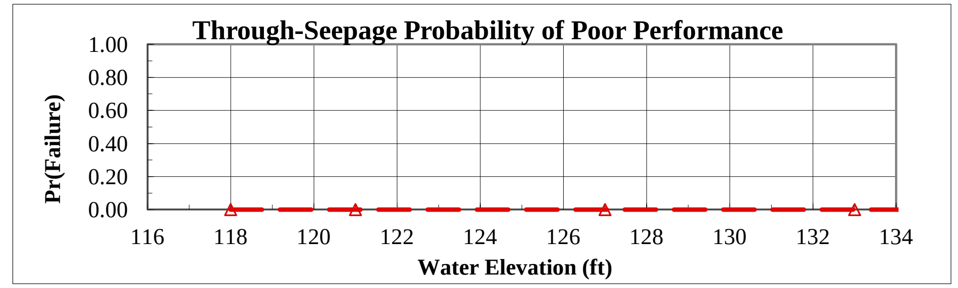
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: Hamilton Bend (MA 7)

Levee Mile: 0.51
River Mile: 2288660.96 N; 6662820.24 E
Analysis Case Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	118.00	0.0000
Toe+3ft	3.00	121.00	0.000000
Half Height	9.00	127.00	0.000000
Crest-3ft	15.00	133.00	0.000000
Crest	18.00	136.00	0.000000

Pr(f)=0
YES

Crest	Head =	18.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft	Head =	15.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height	Head =	9.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft	Head =	3.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

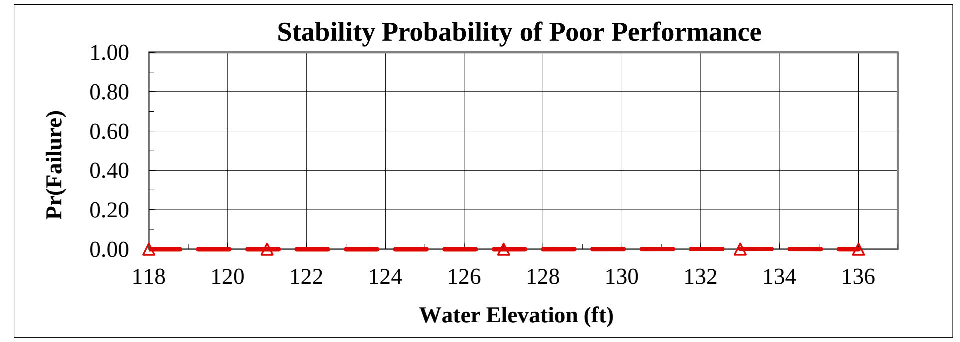
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: Hamilton Bend (MA 7)

Levee Mile: 0.51
River Mile: 2288660.96 N; 6662820.24 E
Analysis Case Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 2/21/2013

Random Variables				
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**	
Levee ϕ'	29	1	5.00	
Levee γ	120	6	5.00	
Foundation c'	150	17	11.50	
Foundation γ	120	6	5.00	
Foundation ϕ'	31	4	11.50	



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	118.00	0.0000
Toe+3ft	3.00	121.00	0.000000
Half Height	9.00	127.00	0.000000
Crest-3ft	15.00	133.00	0.000322
Crest	18.00	136.00	0.000000

Crest | **Head =** 18.00 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.67		
2	28	120	150	120	31	1.71	0.001892	10.54
3	30	120	150	120	31	1.62		
4	29	114	150	120	31	1.69	0.000240	1.34
5	29	126	150	120	31	1.66		
6	29	120	133	120	31	1.73	0.015006	83.59
7	29	120	167	120	31	1.48		
8	29	120	150	114	31	1.70	0.000812	4.52
9	29	120	150	126	31	1.64		
10	29	120	150	120	27	1.74	0.000002	0.01
11	29	120	150	120	35	1.75		

E[FS] = 1.671000   E[ln FS] = 0.510218   Total 0.017953 100.00
Var[FS]= 0.017953
σ[FS]= 0.133990   σ[ln FS]= 0.080057   β = 6.373187
V(FS) = 0.080185   F(z) = 0.000000
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 15.00 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31	1.80		
2	28	120	150	120	31	1.75	0.001936	2.11
3	30	120	150	120	31	1.84		
4	29	114	150	120	31	1.83	0.000812	0.88
5	29	126	150	120	31	1.77		
6	29	120	133	120	31	1.38	0.078400	85.41
7	29	120	167	120	31	1.94		
8	29	120	150	114	31	1.73	0.004556	4.96
9	29	120	150	126	31	1.86		
10	29	120	150	120	27	1.72	0.006084	6.63
11	29	120	150	120	35	1.87		

E[FS] = 1.796000   E[ln FS] = 0.571533   Total 0.091789 100.00
Var[FS]= 0.091789
σ[FS]= 0.302966   σ[ln FS]= 0.167507   β = 3.411982
V(FS) = 0.168689   F(z) = 0.000322
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.032246

Half Height | **Head =** 9.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	29	120	150	120	31			
2	28	120	150	120	31			
3	30	120	150	120	31			
4	29	114	150	120	31			
5	29	126	150	120	31			
6	29	120	133	120	31			
7	29	120	167	120	31			
8	29	120	150	114	31			
9	29	120	150	126	31			
10	29	120	150	120	27			
11	29	120	150	120	35			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   β =
V(FS) =   F(z) =
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

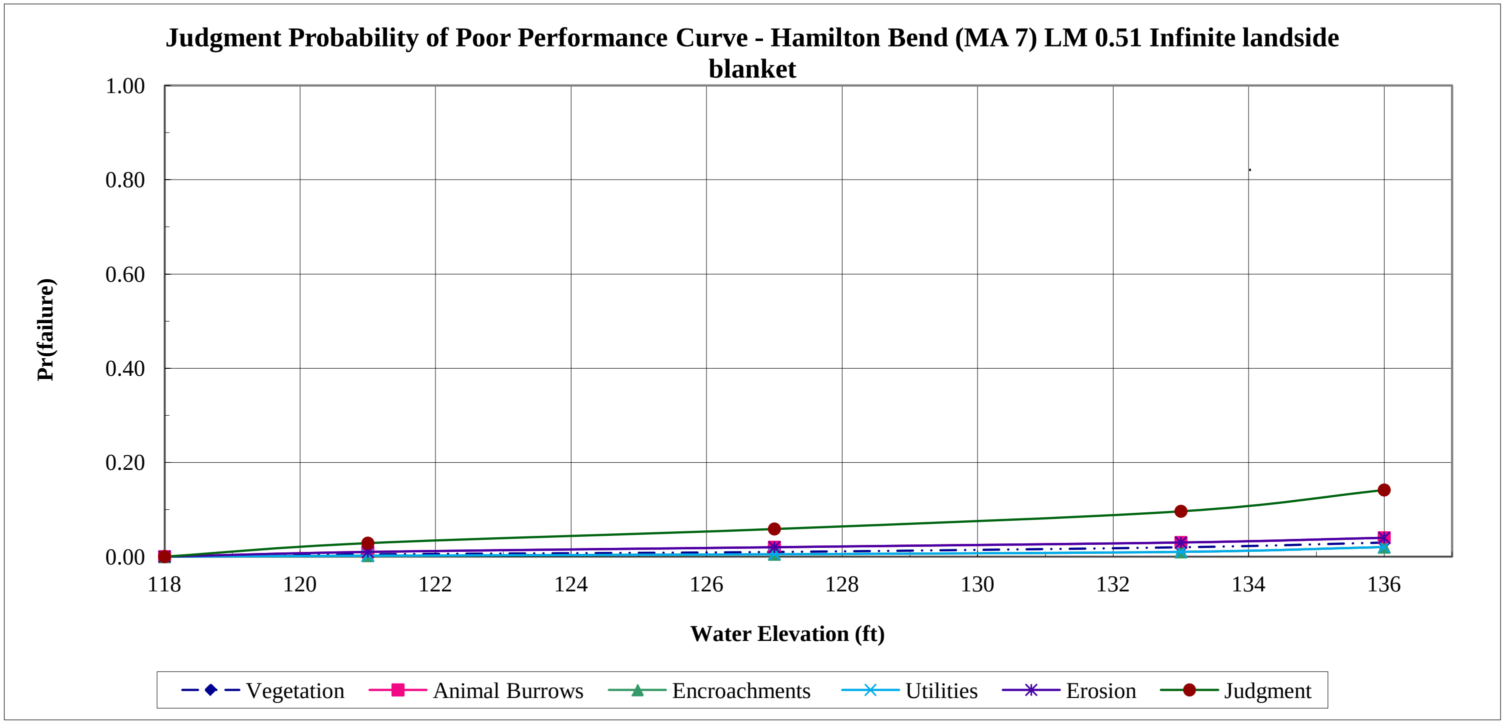
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: Hamilton Bend (MA 7)

Levee Mile: 0.51
River Mile: 2288660.96 N
Analysis Case: Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 2/21/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
118.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
121.00	0.0050	0.9950	0.0100	0.9900	0.0020	0.9980	0.0020	0.9980	0.0100	0.9900	0.0287	0.9713
127.00	0.0100	0.9900	0.0200	0.9800	0.0050	0.9950	0.0050	0.9950	0.0200	0.9800	0.0587	0.9413
133.00	0.0200	0.9800	0.0300	0.9700	0.0100	0.9900	0.0100	0.9900	0.0300	0.9700	0.0963	0.9037
136.00	0.0300	0.9700	0.0400	0.9600	0.0200	0.9800	0.0200	0.9800	0.0400	0.9600	0.1414	0.8586



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

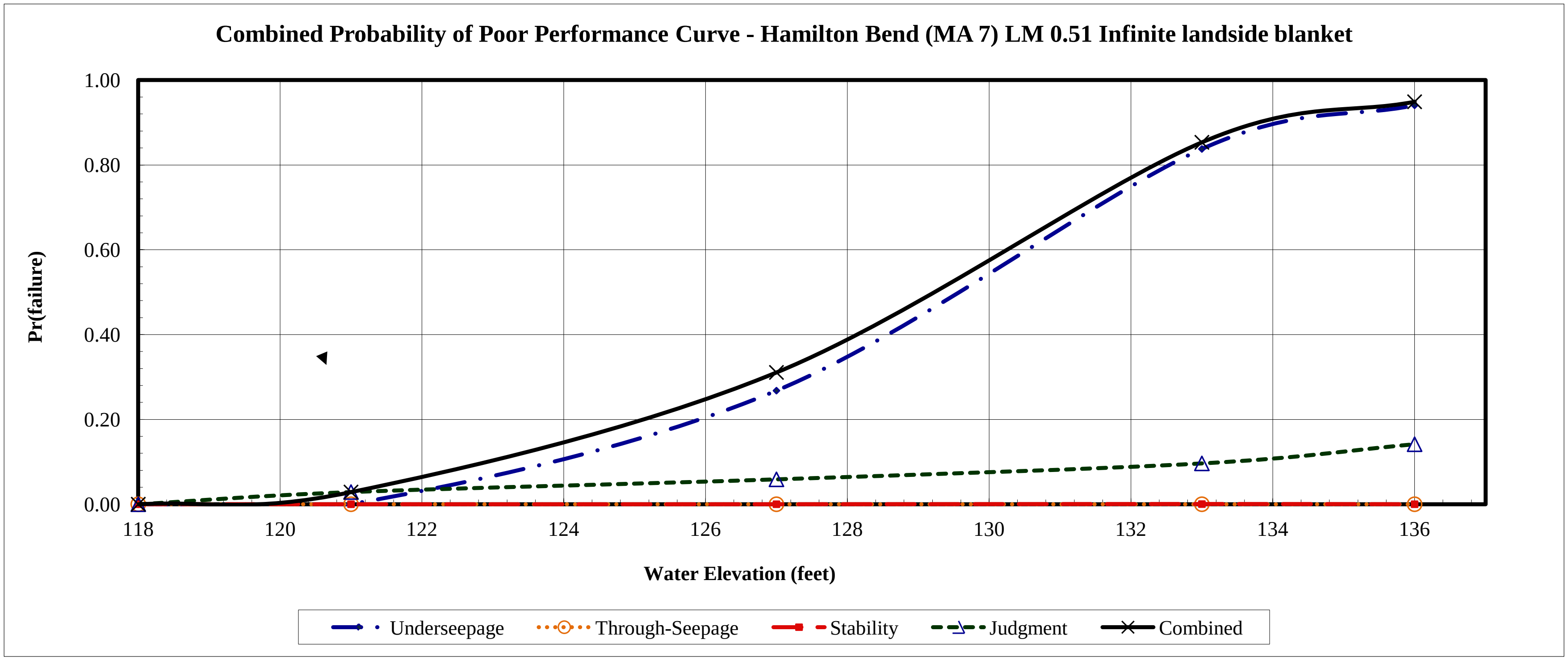
Project: Sutter Feasibility Study
Study Area: Feather River North
River Section: Hamilton Bend (MA 7)

Levee Mile: 0.51
River Mile: 2288660.96 N; (
Analysis Case: Infinite landside blanket

Crest Elev.: 136.00
L/S Toe Elev.: 118.00
W/S Toe Elev.: 118.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 2/21/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
118.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
121.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0287	0.9713	0.0287	0.9713
127.00	0.2678	0.7322	0.0000	1.0000	0.0000	1.0000	0.0587	0.9413	0.3108	0.6892
133.00	0.8376	0.1624	0.0000	1.0000	0.0003	0.9997	0.0963	0.9037	0.8533	0.1467
136.00	0.9405	0.0595	0.0000	1.0000	0.0000	1.0000	0.1414	0.8586	0.9489	0.0511



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Cherokee Canal
Basin and Reach: East Levee

Levee Mile: 9.50
Coordinates: 2301045.948 N: 6637006.261 E
Analysis Case Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/12

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)									
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
2F-08-34	7					9					CL-ML	0.01	SM, GC	3	300				
2F-08-35	3.2					8.3					CL-ML	0.01	SM	1.12	112				
2F-08-36	19					9.5					SC/CL	0.007	SW	10	1429				
2F-08-37	11					10.3					ML/SC/ML	0.1	SW	10	100				
2F-08-38	5	8	5	27	63	15.5	9	5	31	56	CL	0.007	SW	10	1429	417	578	313877	98
2F-08-39	10					13.3					ML/SM/ML	0.14	SM	10	71				
2F-08-40	5					1.5					MH	0.01	SC	1	100				
2F-08-41	9.5					1.5					CL	0.007	SC	1	143				
2F-08-42	6					14.5					sML	0.14	SW, SWg, ML	10	71				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
2F-08-34	CL-ML	7	0.01				7	SM, GC	9	3							3
2F-08-35	CL-ML	3.2	0.01				3.2	SM	8.3	1.12							1.12
2F-08-36	SC	10	0.007	CL	9	0.007	19	SW	9.5	10							10
2F-08-37	ML/SC/ML	11	0.1				11	SW	10.3	10							10
2F-08-38	CL	5	0.007				5	SW	15.5	10							10
2F-08-39	ML	8	0.14	SM/ML	8	0.56	10	SM	13.3	10							10
2F-08-40	MH	5	0.01				5	SC	1.5	1							1
2F-08-41	CL	9.5	0.007				9.5	SC	1.5	1							1
2F-08-42	sML	6	0.14				6	SW, SWg, ML	14.5	10							10

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Cherokee Canal
River Section: East Levee

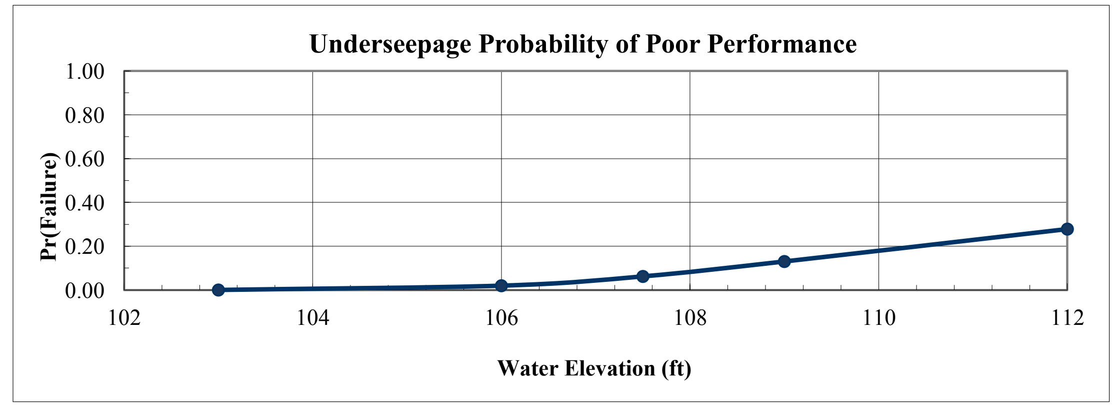
Levee Mile: 9.50
River Mile: 2301045.948 N: 6637006.261 E
Analysis Case Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/12

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	417	409	98
Blanket Thickness (z)	8	5	63
Aquifer Thickness (d)	9	5	56

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	49	65	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	103.00	0.0000
Toe+3ft	3.00	106.00	0.0195
Half Height	4.50	107.50	0.0620
Crest-3ft	6.00	109.00	0.1300
Crest	9.00	112.00	0.2780

Crest	Rh
Head = 9.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	417	8.00	9.00	47.73	173.27	0.0315	5.45	0.68		
2	826	8.00	9.00	48.35	243.82	0.0252	6.14	0.77	0.070225	21.12
3	8	8.00	9.00	23.62	24.50	0.0796	1.95	0.24		
4	417	13.00	9.00	48.21	220.88	0.0269	5.95	0.46	0.255025	76.71
5	417	3.00	9.00	45.79	106.11	0.0415	4.40	1.47		
6	417	8.00	14.00	48.18	216.11	0.0425	5.91	0.74	0.007225	2.17
7	417	8.00	4.00	46.26	115.52	0.0176	4.58	0.57		
								Total	0.332475	100.00

E[I] = 0.680000
Var[I]= 0.332475
σ[I]= 0.576606
V(I) = 0.847951

E[ln I] = -0.656540
σ [ln I] = 0.736040
ln(I crit) = -0.223144

Ic= 0.80

β =	-0.891990
F(z) =	0.722010
Pr(f) % =	27.799044

Crest-3ft	Rh
Head = 6.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	417	8.00	9.00	47.73	173.27	0.0315	3.64	0.46		
2	826	8.00	9.00	48.35	243.82	0.0252	4.10	0.51	0.030625	20.99
3	8	8.00	9.00	23.62	24.50	0.0796	1.30	0.16		
4	417	13.00	9.00	48.21	220.88	0.0269	3.97	0.31	0.112225	76.93
5	417	3.00	9.00	45.79	106.11	0.0415	2.94	0.98		
6	417	8.00	14.00	48.18	216.11	0.0425	3.94	0.49	0.003025	2.07
7	417	8.00	4.00	46.26	115.52	0.0176	3.06	0.38		
								Total	0.145875	100.00

E[I] = 0.460000
Var[I]= 0.145875
σ[I]= 0.381936
V(I) = 0.830295

E[ln I] = -1.038713
σ [ln I] = 0.724132
ln(I crit) = -0.223144

Ic= 0.80

β =	-1.434424
F(z) =	0.869975
Pr(f) % =	13.002548

Half Height	Rh
Head = 4.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	417	8.00	9.00	47.73	173.27	0.0315	2.73	0.34		
2	826	8.00	9.00	48.35	243.82	0.0252	3.07	0.38	0.016900	20.86
3	8	8.00	9.00	23.62	24.50	0.0796	0.97	0.12		
4	417	13.00	9.00	48.21	220.88	0.0269	2.98	0.23	0.062500	77.16
5	417	3.00	9.00	45.79	106.11	0.0415	2.20	0.73		
6	417	8.00	14.00	48.18	216.11	0.0425	2.95	0.37	0.001600	1.98
7	417	8.00	4.00	46.26	115.52	0.0176	2.29	0.29		
								Total	0.081000	100.00

E[I] = 0.340000
Var[I]= 0.081000
σ[I]= 0.284605
V(I) = 0.837073

E[ln I] = -1.344327
σ [ln I] = 0.728722
ln(I crit) = -0.223144

Ic= 0.80

β =	-1.844775
F(z) =	0.938044
Pr(f) % =	6.195558

Toe+3ft	Rh
Head = 3.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	417	8.00	9.00	47.73	173.27	0.0315	1.82	0.23		
2	826	8.00	9.00	48.35	243.82	0.0252	2.05	0.26	0.008100	21.37
3	8	8.00	9.00	23.62	24.50	0.0796	0.65	0.08		
4	417	13.00	9.00	48.21	220.88	0.0269	1.98	0.15	0.028900	76.25
5	417	3.00	9.00	45.79	106.11	0.0415	1.47	0.49		
6	417	8.00	14.00	48.18	216.11	0.0425	1.97	0.25	0.000900	2.37
7	417	8.00	4.00	46.26	115.52	0.0176	1.53	0.19		
								Total	0.037900	100.00

E[I] = 0.230000
Var[I]= 0.037900
σ[I]= 0.194679
V(I) = 0.846431

E[ln I] = -1.739804
σ [ln I] = 0.735021
ln(I crit) = -0.223144

Ic= 0.80

β =	-2.367012
F(z) =	0.980464
Pr(f) % =	1.953616

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

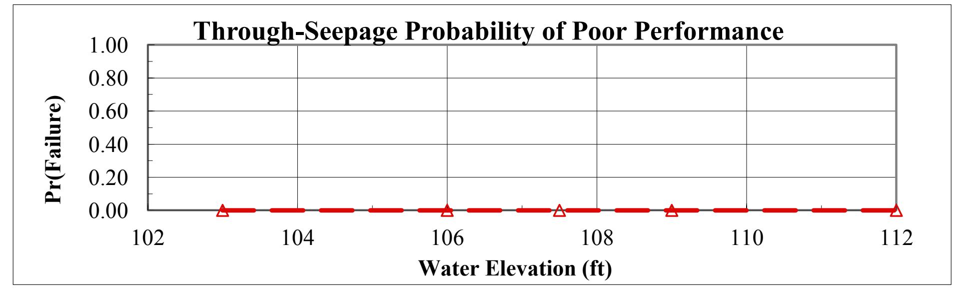
Project: Sutter Feasibility Study
Study Area: Cherokee Canal
River Section: East Levee

Levee Mile: 9.50
River Mile: 2301045.948 N: 6637006.261 E
Analysis Case Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/12

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	103.00	0.0000
Toe+3ft	3.00	106.00	0.000000
Half Height	4.50	107.50	0.000000
Crest-3ft	6.00	109.00	0.000000
Crest	9.00	112.00	0.000000

Pr(f)=0
YES

Crest Head = 9.00 **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Crest-3ft Head = 6.00 **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Half Height Head = 4.50 **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Toe+3ft Head = 3.00 **Horizontal Gradient (Ix) =**

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

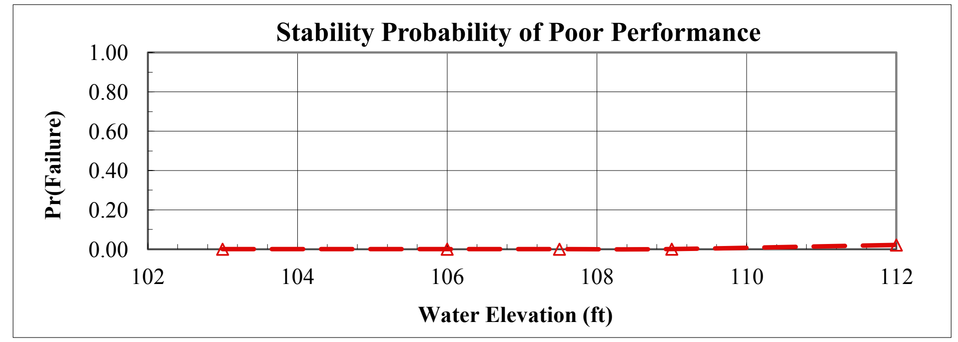
Project: Sutter Feasibility Study
Study Area: Cherokee Canal
River Section: East Levee

Levee Mile: 9.50
River Mile: 2301045.948 N: 6637006.261 E
Analysis Case Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/13/12

Random Variables				
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**	
Levee ϕ'	28	1	5.00	
Levee γ	120	6	5.00	
LEvee c'	150	17	11.50	
Foundation γ	115	6	5.00	
Foundation ϕ'	29	3	11.50	



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	103.00	0.0000
Toe+3ft	3.00	106.00	0.000000
Half Height	4.50	107.50	0.000000
Crest-3ft	6.00	109.00	0.000322
Crest	9.00	112.00	0.021343

Crest	Head =	9.00	Pr(f)=0	NO

Run	Levee φ'	Levee γ	LEvee c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	120	150	115	29	1.50		
2	27	120	150	115	29	1.48	0.001225	1.44
3	29	120	150	115	29	1.55		
4	28	114	150	115	29	1.52	0.000169	0.20
5	28	126	150	115	29	1.50		
6	28	120	133	115	29	1.11	0.074529	87.51
7	28	120	167	115	29	1.66		
8	28	120	150	109	29	1.44	0.005402	6.34
9	28	120	150	121	29	1.58		
10	28	120	150	115	26	1.45	0.003844	4.51
11	28	120	150	115	32	1.57		

E[FS] =	1.504000	E[ln FS] =	0.389648	Total 0.085169 100.00
Var[FS]=	0.085169			
σ[FS]=	0.291838	σ[ln FS]=	0.192251	β = 2.026769
V(FS) =	0.194041			F(z) = 0.021343
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % = 2.134304

Crest-3ft	Head =	6.00	Pr(f)=0	NO

Run	Levee φ'	Levee γ	LEvee c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	120	150	115	29	1.80		
2	27	120	150	115	29	1.75	0.001936	2.11
3	29	120	150	115	29	1.84		
4	28	114	150	115	29	1.83	0.000812	0.88
5	28	126	150	115	29	1.77		
6	28	120	133	115	29	1.38	0.078400	85.41
7	28	120	167	115	29	1.94		
8	28	120	150	109	29	1.73	0.004556	4.96
9	28	120	150	121	29	1.86		
10	28	120	150	115	26	1.72	0.006084	6.63
11	28	120	150	115	32	1.87		

E[FS] =	1.796000	E[ln FS] =	0.571533	Total 0.091789 100.00
Var[FS]=	0.091789			
σ[FS]=	0.302966	σ[ln FS]=	0.167507	β = 3.411982
V(FS) =	0.168689			F(z) = 0.000322
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % = 0.032246

Half Height	Head =	4.50	Pr(f)=0	YES

Run	Levee φ'	Levee γ	LEvee c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	120	150	115	29			
2	27	120	150	115	29			
3	29	120	150	115	29			
4	28	114	150	115	29			
5	28	126	150	115	29			
6	28	120	133	115	29			
7	28	120	167	115	29			
8	28	120	150	109	29			
9	28	120	150	121	29			
10	28	120	150	115	26			
11	28	120	150	115	32			

E[FS] =		E[ln FS] =		Total
Var[FS]=				
σ[FS]=		σ[ln FS]=		β =
V(FS) =				F(z) =
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % = 0.000000

Toe+3ft	Head =	3.00	Pr(f)=0	YES

Run	Levee φ'	Levee γ	LEvee c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	120	150	115	29			
2	27	120	150	115	29			
3	29	120	150	115	29			
4	28	114	150	115	29			
5	28	126	150	115	29			
6	28	120	133	115	29			
7	28	120	167	115	29			
8	28	120	150	109	29			
9	28	120	150	121	29			
10	28	120	150	115	26			
11	28	120	150	115	32			

E[FS] =		E[ln FS] =		Total
Var[FS]=				
σ[FS]=		σ[ln FS]=		β =
V(FS) =				F(z) =
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

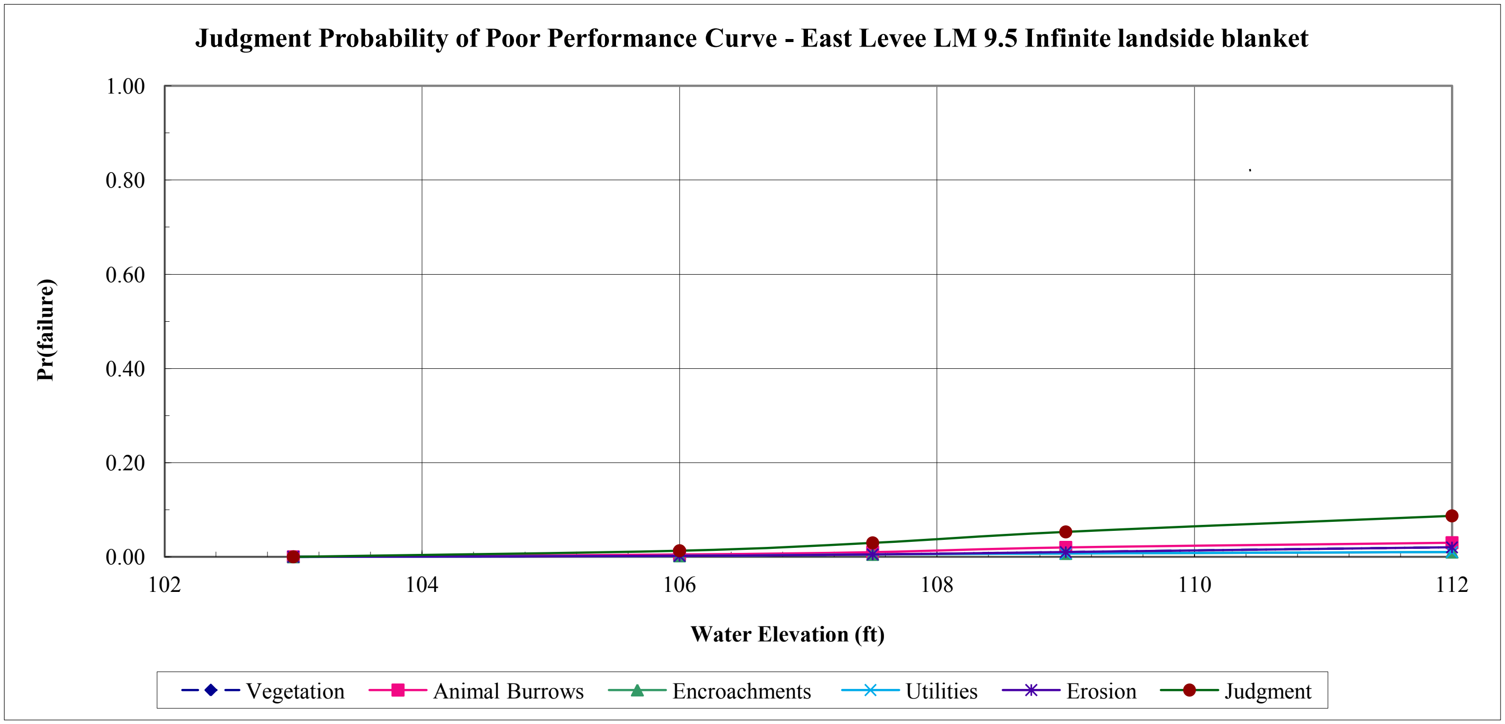
Project: Sutter Feasibility Study
Study Area: Cherokee Canal
River Section: East Levee

Levee Mile: 9.50
River Mile: 2301045.948
Analysis Case: Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/13/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
103.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
106.00	0.0020	0.9980	0.0050	0.9950	0.0020	0.9980	0.0020	0.9980	0.0020	0.9980	0.0129	0.9871
107.50	0.0050	0.9950	0.0100	0.9900	0.0050	0.9950	0.0050	0.9950	0.0050	0.9950	0.0297	0.9703
109.00	0.0100	0.9900	0.0200	0.9800	0.0070	0.9930	0.0070	0.9930	0.0100	0.9900	0.0529	0.9471
112.00	0.0200	0.9800	0.0300	0.9700	0.0100	0.9900	0.0100	0.9900	0.0200	0.9800	0.0870	0.9130



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

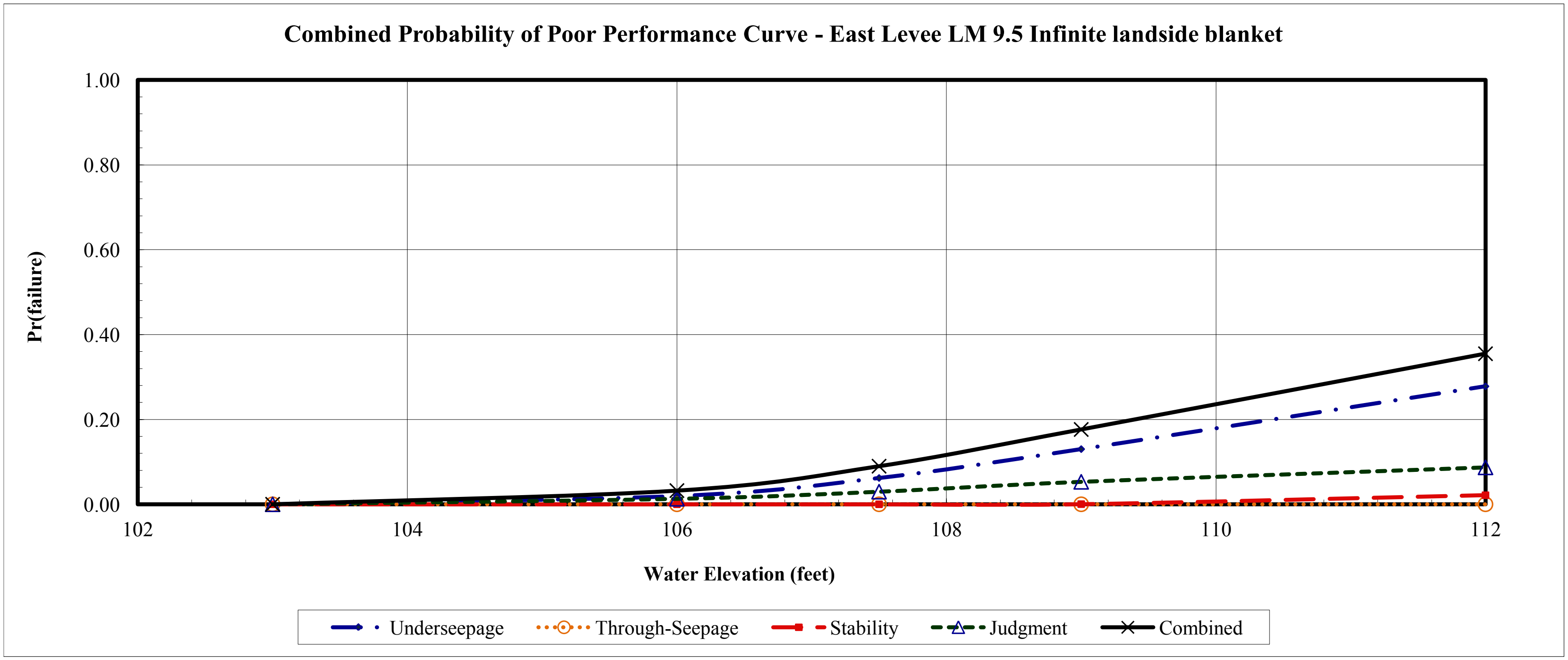
Project: Sutter Feasibility Study
Study Area: Cherokee Canal
River Section: East Levee

Levee Mile: 9.50
River Mile: 2301045.948 N:
Analysis Case: Infinite landside blanket

Crest Elev.: 112.00
L/S Toe Elev.: 103.00
W/S Toe Elev.: 104.00

Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/13/12

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
103.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
106.00	0.0195	0.9805	0.0000	1.0000	0.0000	1.0000	0.0129	0.9871	0.0322	0.9678
107.50	0.0620	0.9380	0.0000	1.0000	0.0000	1.0000	0.0297	0.9703	0.0898	0.9102
109.00	0.1300	0.8700	0.0000	1.0000	0.0003	0.9997	0.0529	0.9471	0.1763	0.8237
112.00	0.2780	0.7220	0.0000	1.0000	0.0213	0.9787	0.0870	0.9130	0.3548	0.6452



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Wadsworth Canal - Right Bank
Basin and Reach:

Levee Mile: 0.50
Coordinates: 2168750 N; 6627910 E
Analysis Case

Crest Elev.: 60.30
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: E.W. James
Checked By: J.M. Bolton
Date: Updated 09/14/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)									
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
2F-00-01	6	7	2	11	29	13	13	11	122	85	CL	0.0071	SP-SC	10	1408	1244	1249	1489425	98
2F-03-26	6					6					CL	0.0071	SP	28	3944				
2F-00-02	7.87					7					CL/ML/SC	0.0284	SP-SC	14	493				
2F-00-09	5.62					30					CL/ML	0.0284	SM/SP	2.8	99				
2F-00-10	8.41					30					CL/ML	0.0071	SP-SM/SP	10.27	1446				
2F-03-28	4.77					2.5					CL/SM	0.0284	SP	28	986				
WSEWWC_001A	10.5					11					CL, CH	0.0071	SM/SP-SM	11.14	1569				
WSEWWC_029B	4					7					CL	0.28	SP-SM	2.8	10				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
2F-00-01	CL	6	0.0071				6.0	SP-SC	13	10							10
2F-03-26	CL	6	0.0071				6.0	SP	6	28							28
2F-00-02	CL	3	0.0284	ML/SC	6	0.035	7.9	SP-SC	7	14							14
2F-00-09	CL	4	0.0284	ML	2	0.035	5.6	SM	2	2.8	SP	28					2.8
2F-00-10	CL	8	0.0071	ML	2	0.035	8.4	SP-SM	2	14	SP	28	10				10.27
2F-03-28	CL	4	0.0284	SM	9.5	0.35	4.8	SP	2.5	28							28
WSEWWC_001A	CL, CH	10.5	0.0071				10.5	SM	2.5	1.4	SP-SM	8.5	14				11.14
WSEWWC_029B	CL	4	0.28				4	SP-SM	7	2.8							2.8

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Wadsworth Canal - Right Bank
River Section:

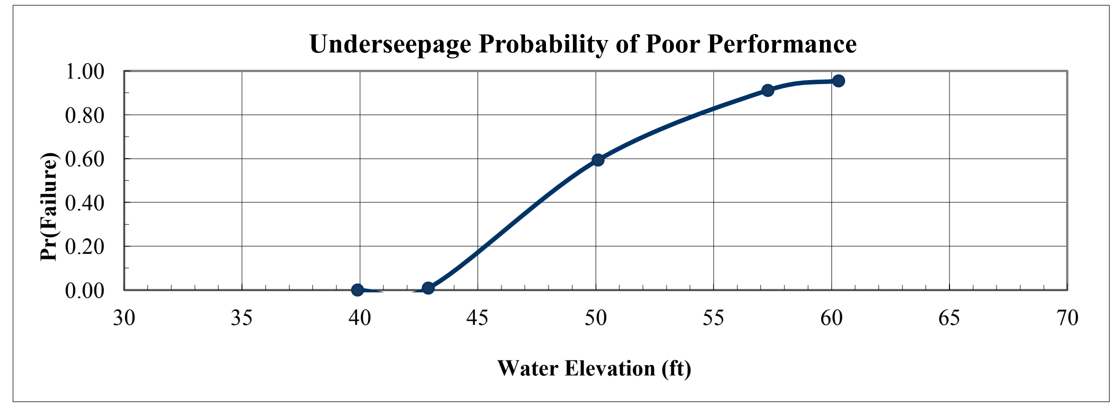
Levee Mile: 0.50
River Mile: 2168750 N; 6627910 E
Analysis Case

Crest Elev.: 60.30
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: E.W. James
Checked By: J.M. Bolton
Date: Updated 09/14/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	1244	1219	98
Blanket Thickness (z)	7	2	29
Aquifer Thickness (d)	13	11	85

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	25	130	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.0088
Half Height	10.20	50.10	0.5935
Crest-3ft	17.40	57.30	0.9112
Crest	20.40	60.30	0.9547

Crest	Rh
Head =	20.40

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1244	7.00	13.00	24.95	336.46	0.0265	13.97	2.00		
2	2463	7.00	13.00	24.98	473.44	0.0207	15.37	2.20	0.570025	56.35
3	25	7.00	13.00	22.93	47.58	0.0648	4.84	0.69		
4	1244	9.00	13.00	24.96	381.51	0.0242	14.51	1.61	0.265225	26.22
5	1244	5.00	13.00	24.94	284.36	0.0296	13.20	2.64		
6	1244	7.00	24.00	24.98	457.16	0.0392	15.24	2.18	0.176400	17.44
7	1244	7.00	2.00	24.71	131.97	0.0070	9.39	1.34		
								Total	1.011650	100.00

E[I] = 2.000000 E[ln I] = 0.580412
Var[I]= 1.011650
σ[I]= 1.005808 σ [ln I] = 0.474838
V(I) = 0.502904

β =	1.222337
F(z) =	0.045297
Pr(f) % =	95.470309

Ic= 0.80 ln(I crit) = -0.223144

Crest-3ft	Rh
Head =	17.40

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1244	7.00	13.00	24.95	336.46	0.0265	11.91	1.70		
2	2463	7.00	13.00	24.98	473.44	0.0207	13.11	1.87	0.409600	55.90
3	25	7.00	13.00	22.93	47.58	0.0648	4.13	0.59		
4	1244	9.00	13.00	24.96	381.51	0.0242	12.37	1.37	0.193600	26.42
5	1244	5.00	13.00	24.94	284.36	0.0296	11.26	2.25		
6	1244	7.00	24.00	24.98	457.16	0.0392	12.99	1.86	0.129600	17.69
7	1244	7.00	2.00	24.71	131.97	0.0070	8.01	1.14		
								Total	0.732800	100.00

E[I] = 1.700000 E[ln I] = 0.417633
Var[I]= 0.732800
σ[I]= 0.856037 σ [ln I] = 0.475385
V(I) = 0.503551

β =	0.878515
F(z) =	0.088843
Pr(f) % =	91.115651

Ic= 0.80 ln(I crit) = -0.223144

Half Height	Rh
Head =	10.20

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1244	7.00	13.00	24.95	336.46	0.0265	6.98	1.00		
2	2463	7.00	13.00	24.98	473.44	0.0207	7.68	1.10	0.140625	56.31
3	25	7.00	13.00	22.93	47.58	0.0648	2.42	0.35		
4	1244	9.00	13.00	24.96	381.51	0.0242	7.25	0.81	0.065025	26.04
5	1244	5.00	13.00	24.94	284.36	0.0296	6.60	1.32		
6	1244	7.00	24.00	24.98	457.16	0.0392	7.62	1.09	0.044100	17.66
7	1244	7.00	2.00	24.71	131.97	0.0070	4.70	0.67		
								Total	0.249750	100.00

E[I] = 1.000000 E[ln I] = -0.111472
Var[I]= 0.249750
σ[I]= 0.499750 σ [ln I] = 0.472169
V(I) = 0.499750

β =	-0.236084
F(z) =	0.406519
Pr(f) % =	59.348079

Ic= 0.80 ln(I crit) = -0.223144

Toe+3ft	Rh
Head =	3.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	1244	7.00	13.00	24.95	336.46	0.0265	2.05	0.29		
2	2463	7.00	13.00	24.98	473.44	0.0207	2.26	0.32	0.012100	56.74
3	25	7.00	13.00	22.93	47.58	0.0648	0.71	0.10		
4	1244	9.00	13.00	24.96	381.51	0.0242	2.13	0.24	0.005625	26.38
5	1244	5.00	13.00	24.94	284.36	0.0296	1.94	0.39		
6	1244	7.00	24.00	24.98	457.16	0.0392	2.24	0.32	0.003600	16.88
7	1244	7.00	2.00	24.71	131.97	0.0070	1.38	0.20		
								Total	0.021325	100.00

E[I] = 0.290000 E[ln I] = -1.350871
Var[I]= 0.021325
σ[I]= 0.146031 σ [ln I] = 0.475387
V(I) = 0.503555

β =	-2.841621
F(z) =	0.991159
Pr(f) % =	0.884059

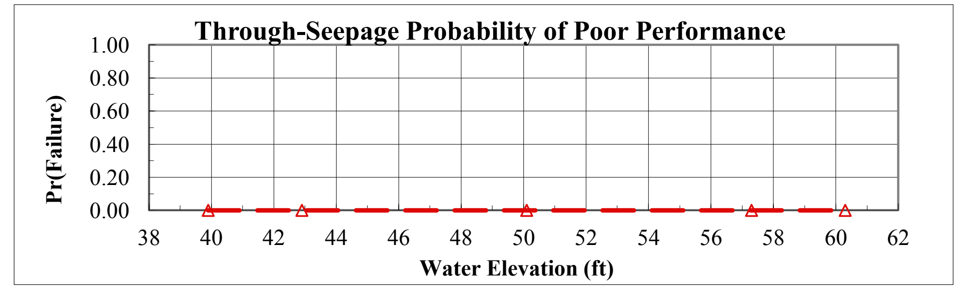
Ic= 0.80 ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project:	Sutter Feasibility Study	**Levee Mile:**	0.50	**Crest Elev.:**	60.30	**Analysis By:**	E.W. James
Study Area:	Wadsworth Canal - Right Bank	**River Mile:**	2168750 N; 6627910 E	**L/S Toe Elev.:**	39.90	**Checked By:**	J.M. Bolton
River Section:		**Analysis Case**		**W/S Toe Elev.:**	41.50	**Date:**	Updated 09/14/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.000000
Half Height	10.20	50.10	0.000000
Crest-3ft	17.40	57.30	0.000000
Crest	20.40	60.30	0.000000

Crest | **Head =** 20.40 | **Horizontal Gradient (Ix) =**

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 17.40 | **Horizontal Gradient (Ix) =**

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Half Height | **Head =** 10.20 | **Horizontal Gradient (Ix) =**

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Horizontal Gradient (Ix) =**

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

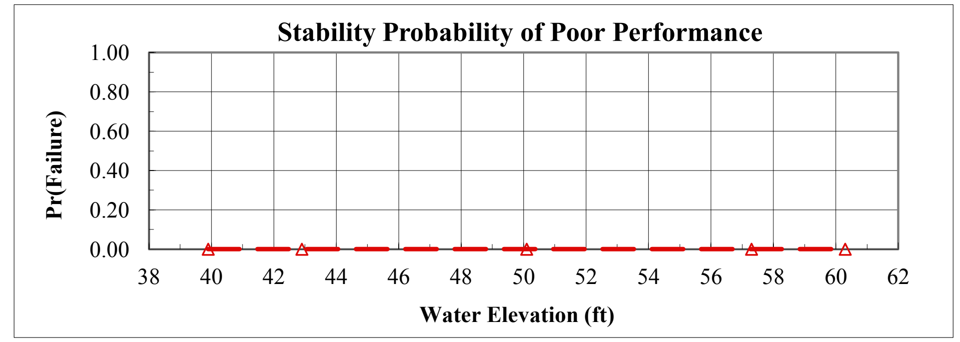
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal - Right Bank
River Section:

Levee Mile: 0.50
River Mile: 2168750 N; 6627910 E
Analysis Case

Crest Elev.: 60.30
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: E.W. James
Checked By: J.M. Bolton
Date: Updated 09/14/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	32	4	13.00
Levee Cohesion	10	4	40.00
Levee γ	125	9	7.00
Foundation Φ	28	4	13.00
Foundation Cohesion	100	40	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.000000
Half Height	10.20	50.10	0.000000
Crest-3ft	17.40	57.30	0.000000
Crest	20.40	60.30	0.000026

Crest | **Head =** 20.40 | **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	32	10	125	28	100	1.49		
2	28	10	125	28	100	1.46	0.000930	4.37
3	36	10	125	28	100	1.52		
4	32	6	125	28	100	1.49	0.000020	0.10
5	32	14	125	28	100	1.50		
6	32	10	116	28	100	1.51	0.000342	1.61
7	32	10	134	28	100	1.47		
8	32	10	125	24	100	1.43	0.004489	21.09
9	32	10	125	32	100	1.56		
10	32	10	125	28	60	1.35	0.015500	72.83
11	32	10	125	28	140	1.60		

E[FS] = 1.491000 | E[ln FS] = 0.394683 | Total 0.021282 100.00
Var[FS]= 0.021282
σ[FS]= 0.145884 | σ[ln FS]= 0.097610 | **β =** 4.043480
V(FS) = 0.097843 | **F(z) =** 0.000026
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.002633

Crest-3ft | **Head =** 17.40 | **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	32	10	125	28	100			
2	28	10	125	28	100			
3	36	10	125	28	100			
4	32	6	125	28	100			
5	32	14	125	28	100			
6	32	10	116	28	100			
7	32	10	134	28	100			
8	32	10	125	24	100			
9	32	10	125	32	100			
10	32	10	125	28	60			
11	32	10	125	28	140			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | **β =**
V(FS) = | **F(z) =**
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Half Height | **Head =** 10.20 | **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	32	10	125	28	100			
2	28	10	125	28	100			
3	36	10	125	28	100			
4	32	6	125	28	100			
5	32	14	125	28	100			
6	32	10	116	28	100			
7	32	10	134	28	100			
8	32	10	125	24	100			
9	32	10	125	32	100			
10	32	10	125	28	60			
11	32	10	125	28	140			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | **β =**
V(FS) = | **F(z) =**
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	32	10	125	28	100			
2	28	10	125	28	100			
3	36	10	125	28	100			
4	32	6	125	28	100			
5	32	14	125	28	100			
6	32	10	116	28	100			
7	32	10	134	28	100			
8	32	10	125	24	100			
9	32	10	125	32	100			
10	32	10	125	28	60			
11	32	10	125	28	140			

E[FS] = | E[ln FS] = | Total
Var[FS]=
σ[FS]= | σ[ln FS]= | **β =**
V(FS) = | **F(z) =**
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

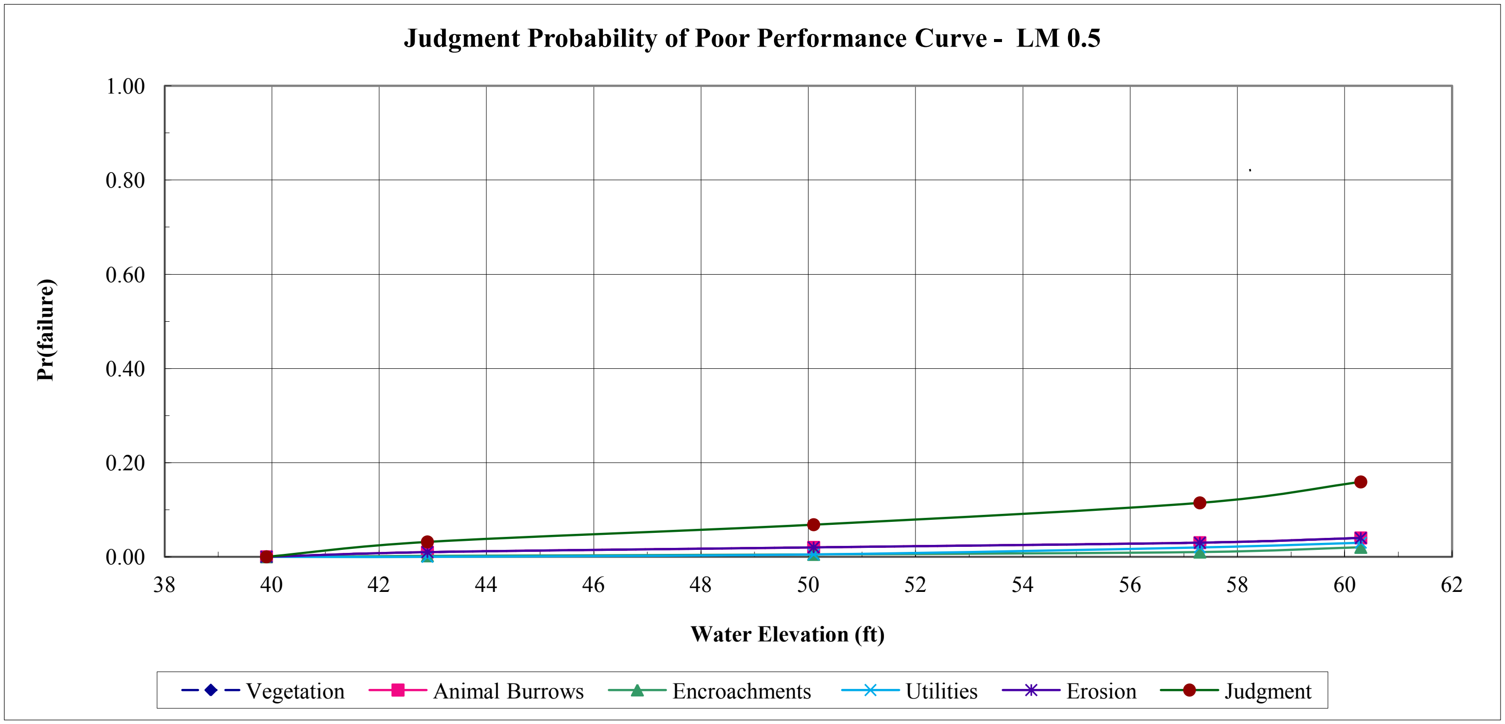
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal - Right Bank
River Section:

Levee Mile: 0.50
River Mile: 2168750 N; 6
Analysis Case:

Crest Elev.: 60.30
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: E.W. James
Checked By: J.M. Bolton
Date: Updated 09/1

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
39.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
42.90	0.0100	0.9900	0.0100	0.9900	0.0020	0.9980	0.0000	1.0000	0.0100	0.9900	0.0316	0.9684
50.10	0.0200	0.9800	0.0200	0.9800	0.0050	0.9950	0.0050	0.9950	0.0200	0.9800	0.0682	0.9318
57.30	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0300	0.9700	0.1145	0.8855
60.30	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

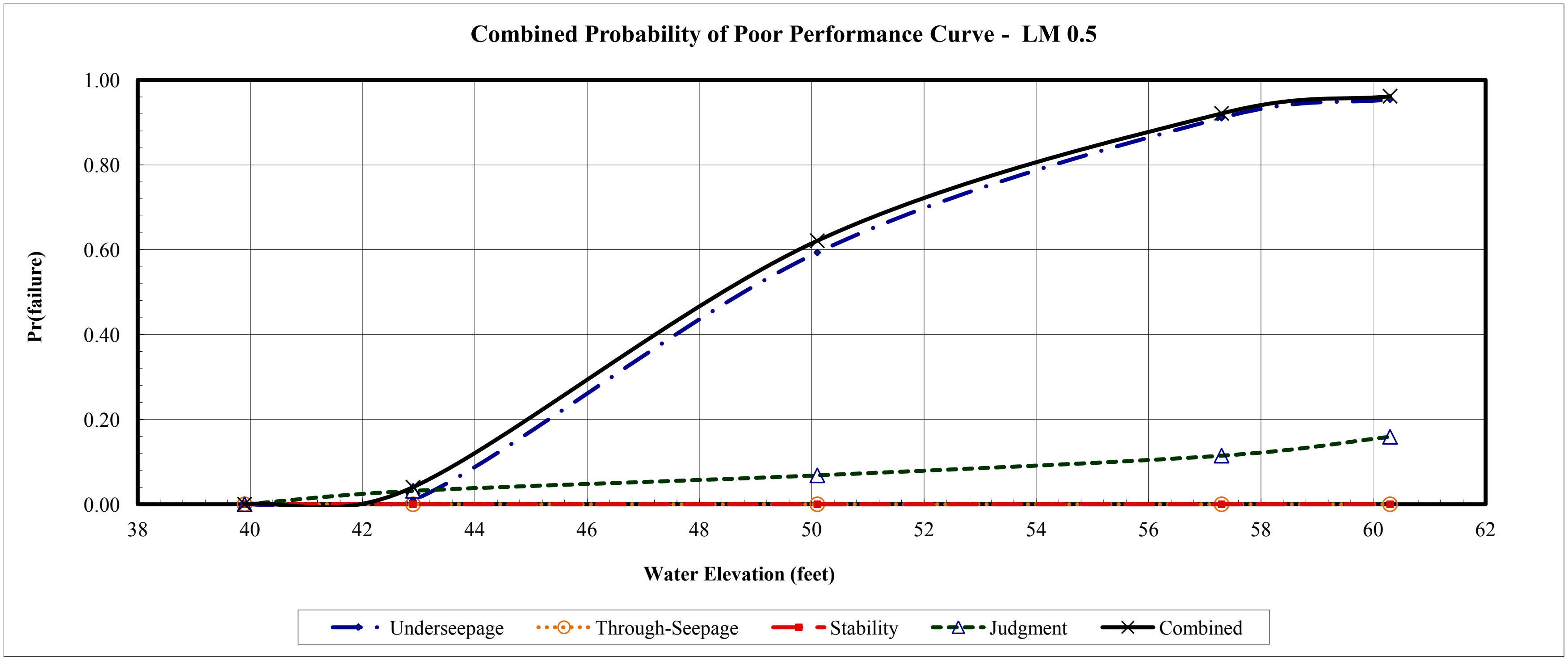
Project: Sutter Feasibility Study
Study Area: Wadsworth Canal - Right Bank
River Section:

Levee Mile: 0.50
River Mile: 2168750 N; 662
Analysis Case:

Crest Elev.: 60.30
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: E.W. James
Checked By: J.M. Bolton
Date: Updated 09/14/:

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
39.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
42.90	0.0088	0.9912	0.0000	1.0000	0.0000	1.0000	0.0316	0.9684	0.0402	0.9598
50.10	0.5935	0.4065	0.0000	1.0000	0.0000	1.0000	0.0682	0.9318	0.6212	0.3788
57.30	0.9112	0.0888	0.0000	1.0000	0.0000	1.0000	0.1145	0.8855	0.9213	0.0787
60.30	0.9547	0.0453	0.0000	1.0000	0.0000	1.0000	0.1590	0.8410	0.9619	0.0381



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Sutter Feasibility Study
Channel: Sutter Bypass
Basin and Reach: Left Levee

Levee Mile: 4.00
Coordinates: 2168110 N; 6626590 E
Analysis Case Infinite waterside/landside blanket

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/14/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WSESBP_011B	14					9					sCL/CH	0.007	SP-SM	10	1429				
WSESBP_015B	9					16.5					CLs,CL,ML	0.1	SM	3	30				
WSESBP_016B	10.5					9					SM/CL	0.007	SP-SM	10	1429				
WSESBP_017B	8	10	4	25	40	2	10	6	43	60	CL,SM	0.3	SP-SM	10	33	725	650	427548	90
WSESBP_018B	14					8					CL,SM/CL	0.007	SP-SM,ML,sMI	3	429				
WSESBP_019B	4.8					17.5					CL/ML	0.01	SP-SM	10	1000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WSESBP_011B	sCL	6	0.007	CH	8	0.007	14	SP-SM	9	10							10
WSESBP_015B	CLs,CL,ML	9	0.1				9	SM	16.5	3							3
WSESBP_016B	SM	2	0.007	CL	8.5	0.007	10.5	SP-SM	9	10							10
WSESBP_017B	CL,SM	8	0.3				8	SP-SM	2	10							10
WSESBP_018B	CL,SM	2.5	0.007	CL	11.5	0.007	14	SP-SM,ML,sMI	8	3							3
WSESBP_019B	CL	4.5	0.01	ML	3	0.1	4.8	SP-SM	17.5	10							10

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: Left Levee

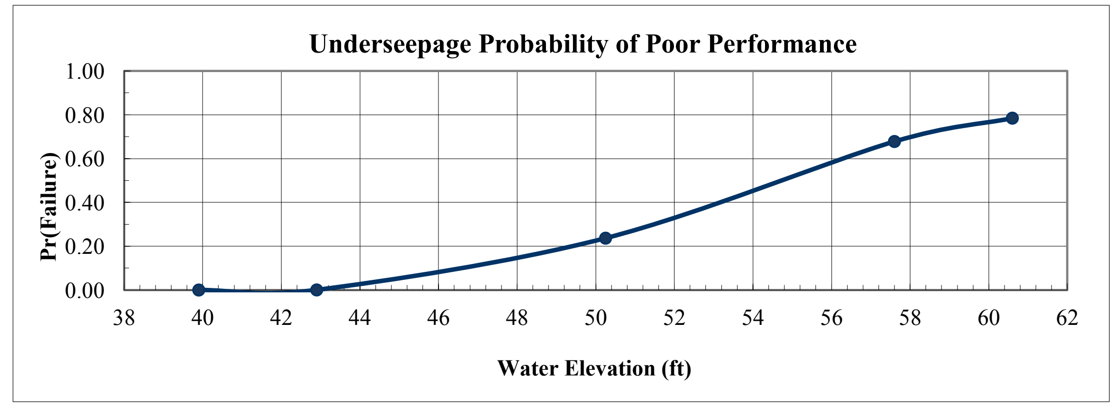
Levee Mile: 4.00
River Mile: 2168110 N; 6626590 E
Analysis Case Infinite waterside/landside blanket

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/14/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Permaebility Ratio	725	650	90
Blanket Thickness (z)	10	4	40
Aquifer Thickness (d)	10	6	60

Blanket Theory Analysis Inputs					
Pr(f)=0	BTA Case No.	L1	L2	L3	γ Blanket
NO	7A	15	150	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.0004
Half Height	10.35	50.25	0.2366
Crest-3ft	17.70	57.60	0.6780
Crest	20.70	60.60	0.7846

Crest	Rh
Head =	20.70

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	12.84	1.28		
2	1375	10.00	10.00	14.99	370.81	0.0187	14.33	1.43	0.129600	33.32
3	75	10.00	10.00	14.85	86.60	0.0398	7.13	0.71		
4	725	14.00	10.00	14.99	318.59	0.0207	13.64	0.97	0.230400	59.24
5	725	6.00	10.00	14.97	208.57	0.0268	11.56	1.93		
6	725	10.00	16.00	14.99	340.59	0.0316	13.94	1.39	0.028900	7.43
7	725	10.00	4.00	14.96	170.29	0.0119	10.51	1.05		
								Total	0.388900	100.00

E[I] = 1.280000
Var[I]= 0.388900
σ[I]= 0.623618
V(I) = 0.487202

E[ln I] = 0.140368
σ [ln I] = 0.461503

β =	0.304154
F(z) =	0.215445
Pr(f) % =	78.455479

Ic= 0.80

ln(I crit) = -0.223144

Crest-3ft	Rh
Head =	17.70

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	10.98	1.10		
2	1375	10.00	10.00	14.99	370.81	0.0187	12.25	1.23	0.096100	33.69
3	75	10.00	10.00	14.85	86.60	0.0398	6.10	0.61		
4	725	14.00	10.00	14.99	318.59	0.0207	11.66	0.83	0.168100	58.94
5	725	6.00	10.00	14.97	208.57	0.0268	9.88	1.65		
6	725	10.00	16.00	14.99	340.59	0.0316	11.92	1.19	0.021025	7.37
7	725	10.00	4.00	14.96	170.29	0.0119	8.99	0.90		
								Total	0.285225	100.00

E[I] = 1.100000
Var[I]= 0.285225
σ[I]= 0.534065
V(I) = 0.485513

E[ln I] = -0.010518
σ [ln I] = 0.460061

β =	-0.022862
F(z) =	0.321980
Pr(f) % =	67.801956

Ic= 0.80

ln(I crit) = -0.223144

Half Height	Rh
Head =	10.35

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	6.42	0.64		
2	1375	10.00	10.00	14.99	370.81	0.0187	7.16	0.72	0.032400	34.16
3	75	10.00	10.00	14.85	86.60	0.0398	3.56	0.36		
4	725	14.00	10.00	14.99	318.59	0.0207	6.82	0.49	0.055225	58.22
5	725	6.00	10.00	14.97	208.57	0.0268	5.78	0.96		
6	725	10.00	16.00	14.99	340.59	0.0316	6.97	0.70	0.007225	7.62
7	725	10.00	4.00	14.96	170.29	0.0119	5.26	0.53		
								Total	0.094850	100.00

E[I] = 0.640000
Var[I]= 0.094850
σ[I]= 0.307977
V(I) = 0.481214

E[ln I] = -0.550431
σ [ln I] = 0.456385

β =	-1.206066
F(z) =	0.763353
Pr(f) % =	23.664715

Ic= 0.80

ln(I crit) = -0.223144

Toe+3ft	Rh
Head =	3.00

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	725	10.00	10.00	14.98	269.26	0.0230	1.86	0.19		
2	1375	10.00	10.00	14.99	370.81	0.0187	2.08	0.21	0.003025	35.38
3	75	10.00	10.00	14.85	86.60	0.0398	1.03	0.10		
4	725	14.00	10.00	14.99	318.59	0.0207	1.98	0.14	0.004900	57.31
5	725	6.00	10.00	14.97	208.57	0.0268	1.68	0.28		
6	725	10.00	16.00	14.99	340.59	0.0316	2.02	0.20	0.000625	7.31
7	725	10.00	4.00	14.96	170.29	0.0119	1.52	0.15		
								Total	0.008550	100.00

E[I] = 0.190000
Var[I]= 0.008550
σ[I]= 0.092466
V(I) = 0.486664

E[ln I] = -1.767012
σ [ln I] = 0.461044

β =	-3.832633
F(z) =	0.999594
Pr(f) % =	0.040605

Ic= 0.80

ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: Left Levee

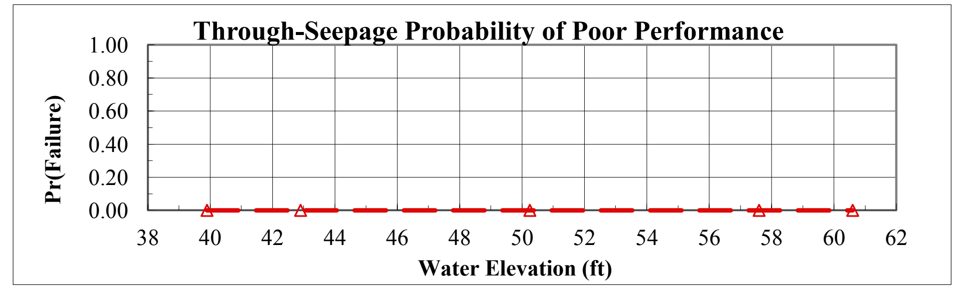
Levee Mile: 4.00
River Mile: 2168110 N; 6626590 E
Analysis Case Infinite waterside/landside blanket

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/14/2012

Random Variables			
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Tractive Stress (Tc)			10.00
Initial Porosity (n)			10.00
Initial Permeability (Ko)			30.00

Pr(f)=0
YES



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.000000
Half Height	10.35	50.25	0.000000
Crest-3ft	17.70	57.60	0.000000
Crest	20.70	60.60	0.000000

Crest	Head =	20.70	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Crest-3ft	Head =	17.70	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Half Height	Head =	10.35	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Toe+3ft	Head =	3.00	Horizontal Gradient (Ix) =	

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)							
2							
3							
4							
5							
6							
7							

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

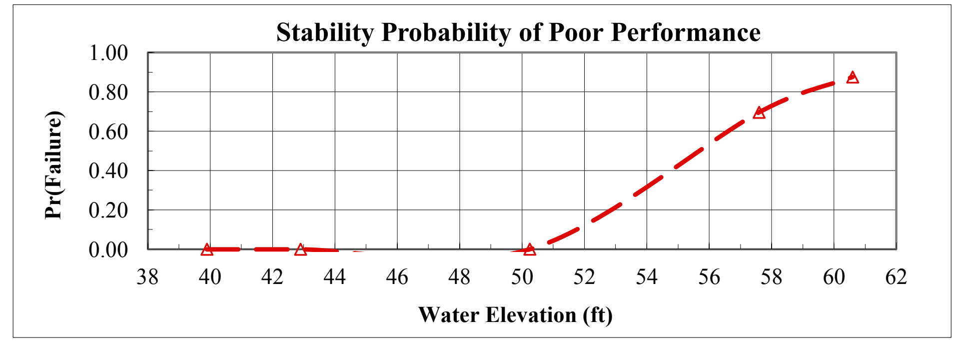
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: Left Levee

Levee Mile: 4.00
River Mile: 2168110 N; 6626590 E
Analysis Case Infinite waterside/landside blanket

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: T. Huynh
Checked By: E.W. James/J.M. Bolton
Date: Updated 9/14/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee ϕ'	28	3	12.00
Levee γ	115	6	5.00
Foundation c'	200	66	33.00
Foundation γ	115	6	5.00
Foundation ϕ'	28	3	12.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	39.90	0.0000
Toe+3ft	3.00	42.90	0.000000
Half Height	10.35	50.25	0.000457
Crest-3ft	17.70	57.60	0.695853
Crest	20.70	60.60	0.875362

Crest | **Head =** 20.70 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	115	200	115	28	0.88		
2	25	115	200	115	28	0.79	0.008930	83.83
3	31	115	200	115	28	0.98		
4	28	109	200	115	28	0.84	0.001722	16.17
5	28	121	200	115	28	0.92		
6	28	115	134	115	28	0.88	0.000000	0.00
7	28	115	266	115	28	0.88		
8	28	115	200	109	28	0.88	0.000000	0.00
9	28	115	200	121	28	0.88		
10	28	115	200	115	25	0.88	0.000000	0.00
11	28	115	200	115	31	0.88		

E[FS] =	0.880000	E[ln FS] =	-0.134664	Total	0.010653		100.00
Var[FS]=	0.010653						
σ[FS]=	0.103211	σ[ln FS]=	0.116885			β =	-1.152112
V(FS) =	0.117285					F(z) =	0.875362
FS req'd =	1.00	ln(FS req'd) =	0.000000			Pr(f) % =	87.536249

Crest-3ft | **Head =** 17.70 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	115	200	115	28	0.95		
2	25	115	200	115	28	0.85	0.009604	70.29
3	31	115	200	115	28	1.05		
4	28	109	200	115	28	0.90	0.001560	11.42
5	28	121	200	115	28	0.98		
6	28	115	134	115	28	1.05	0.002500	18.30
7	28	115	266	115	28	0.95		
8	28	115	200	109	28	0.95	0.000000	0.00
9	28	115	200	121	28	0.95		
10	28	115	200	115	25	0.95	0.000000	0.00
11	28	115	200	115	31	0.95		

E[FS] =	0.946000	E[ln FS] =	-0.063089	Total	0.013664		100.00
Var[FS]=	0.013664						
σ[FS]=	0.116894	σ[ln FS]=	0.123099			β =	-0.512509
V(FS) =	0.123567					F(z) =	0.695853
FS req'd =	1.00	ln(FS req'd) =	0.000000			Pr(f) % =	69.585275

Half Height | **Head =** 10.35 | **Pr(f)=0** NO

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	115	200	115	28	1.48		
2	25	115	200	115	28	1.31	0.030102	99.21
3	31	115	200	115	28	1.66		
4	28	109	200	115	28	1.47	0.000240	0.79
5	28	121	200	115	28	1.50		
6	28	115	134	115	28	1.48	0.000000	0.00
7	28	115	266	115	28	1.48		
8	28	115	200	109	28	1.48	0.000000	0.00
9	28	115	200	121	28	1.48		
10	28	115	200	115	25	1.48	0.000000	0.00
11	28	115	200	115	31	1.48		

E[FS] =	1.484000	E[ln FS] =	0.387899	Total	0.030343		100.00
Var[FS]=	0.030343						
σ[FS]=	0.174191	σ[ln FS]=	0.116978			β =	3.316000
V(FS) =	0.117379					F(z) =	0.000457
FS req'd =	1.00	ln(FS req'd) =	0.000000			Pr(f) % =	0.045658

Toe+3ft | **Head =** 3.00 | **Pr(f)=0** YES

Run	Levee φ'	Levee γ	Foundation c'	Foundation γ	Foundation φ'	FS	Variance Component	% Variance
1 (Mean)	28	115	200	115	28			
2	25	115	200	115	28			
3	31	115	200	115	28			
4	28	109	200	115	28			
5	28	121	200	115	28			
6	28	115	134	115	28			
7	28	115	266	115	28			
8	28	115	200	109	28			
9	28	115	200	121	28			
10	28	115	200	115	25			
11	28	115	200	115	31			

E[FS] =		E[ln FS] =		Total			
Var[FS]=							
σ[FS]=		σ[ln FS]=				β =	
V(FS) =						F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000			Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

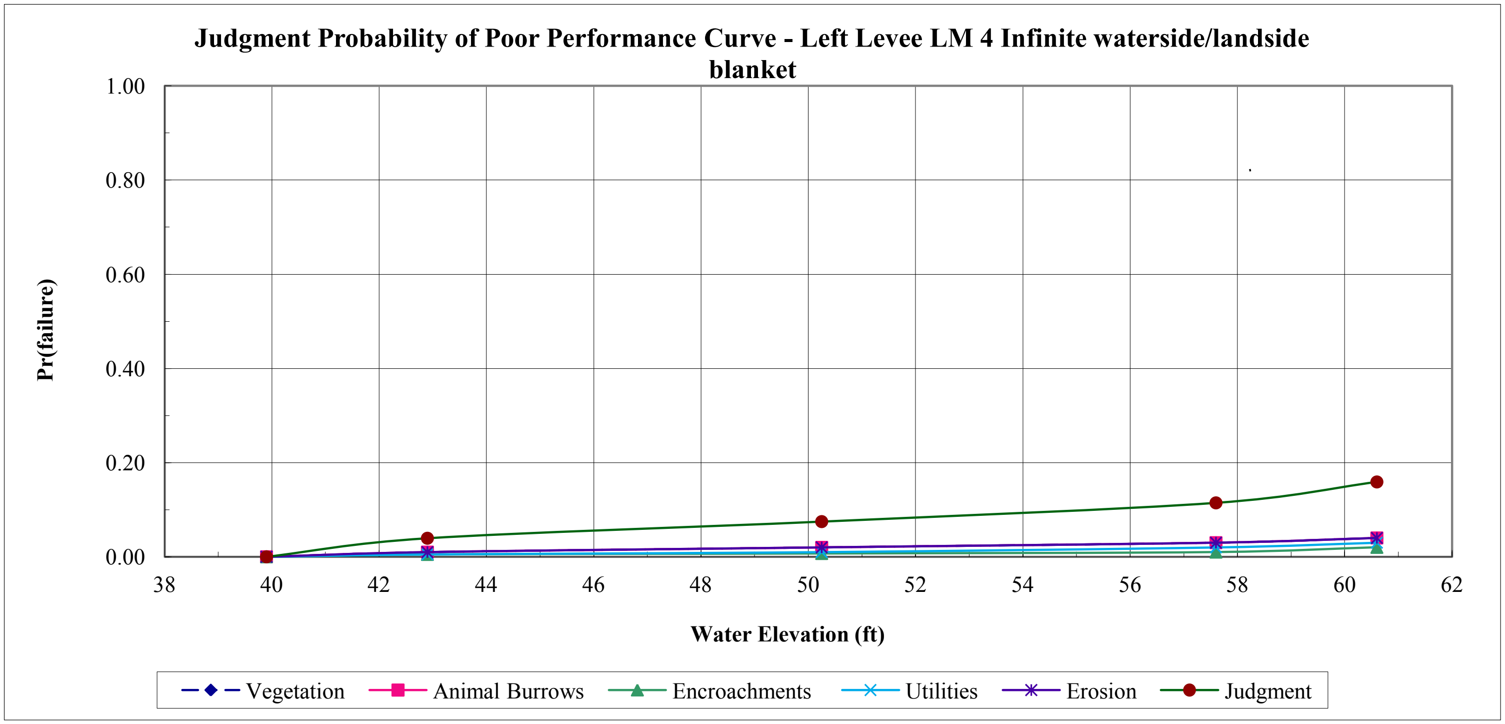
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: Left Levee

Levee Mile: 4.00
River Mile: 2168110 N; 6
Analysis Case: Infinite waterside/landside bl

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

Analysis By: T. Huynh
Checked By: E.W. James/J.
Date: Updated 9/14/

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
39.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
42.90	0.0100	0.9900	0.0100	0.9900	0.0050	0.9950	0.0050	0.9950	0.0100	0.9900	0.0394	0.9606
50.25	0.0200	0.9800	0.0200	0.9800	0.0070	0.9930	0.0100	0.9900	0.0200	0.9800	0.0747	0.9253
57.60	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.0300	0.9700	0.1145	0.8855
60.60	0.0400	0.9600	0.0400	0.9600	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.1590	0.8410



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

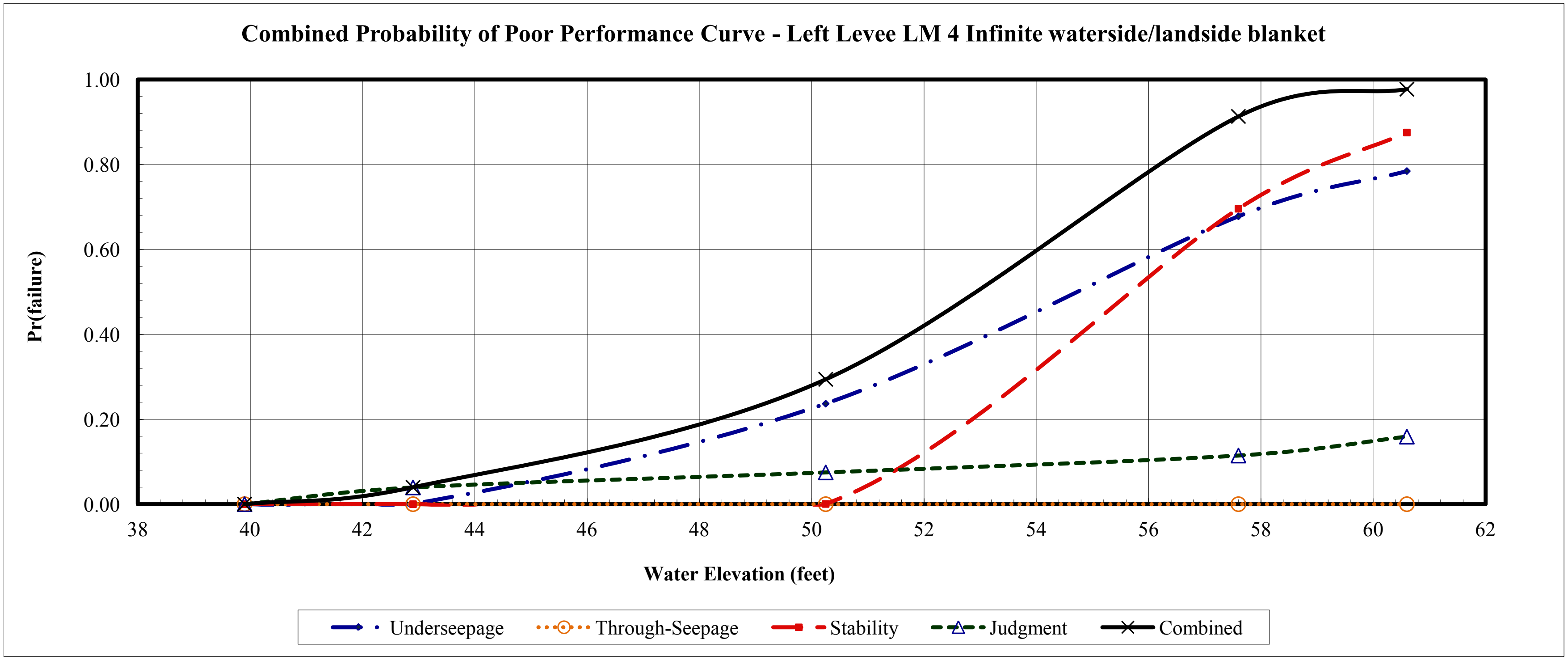
Project: Sutter Feasibility Study
Study Area: Sutter Bypass
River Section: Left Levee

Levee Mile: 4.00
River Mile: 2168110 N; 662
Analysis Case: Infinite waterside/landside blanket

Crest Elev.: 60.60
L/S Toe Elev.: 39.90
W/S Toe Elev.: 41.50

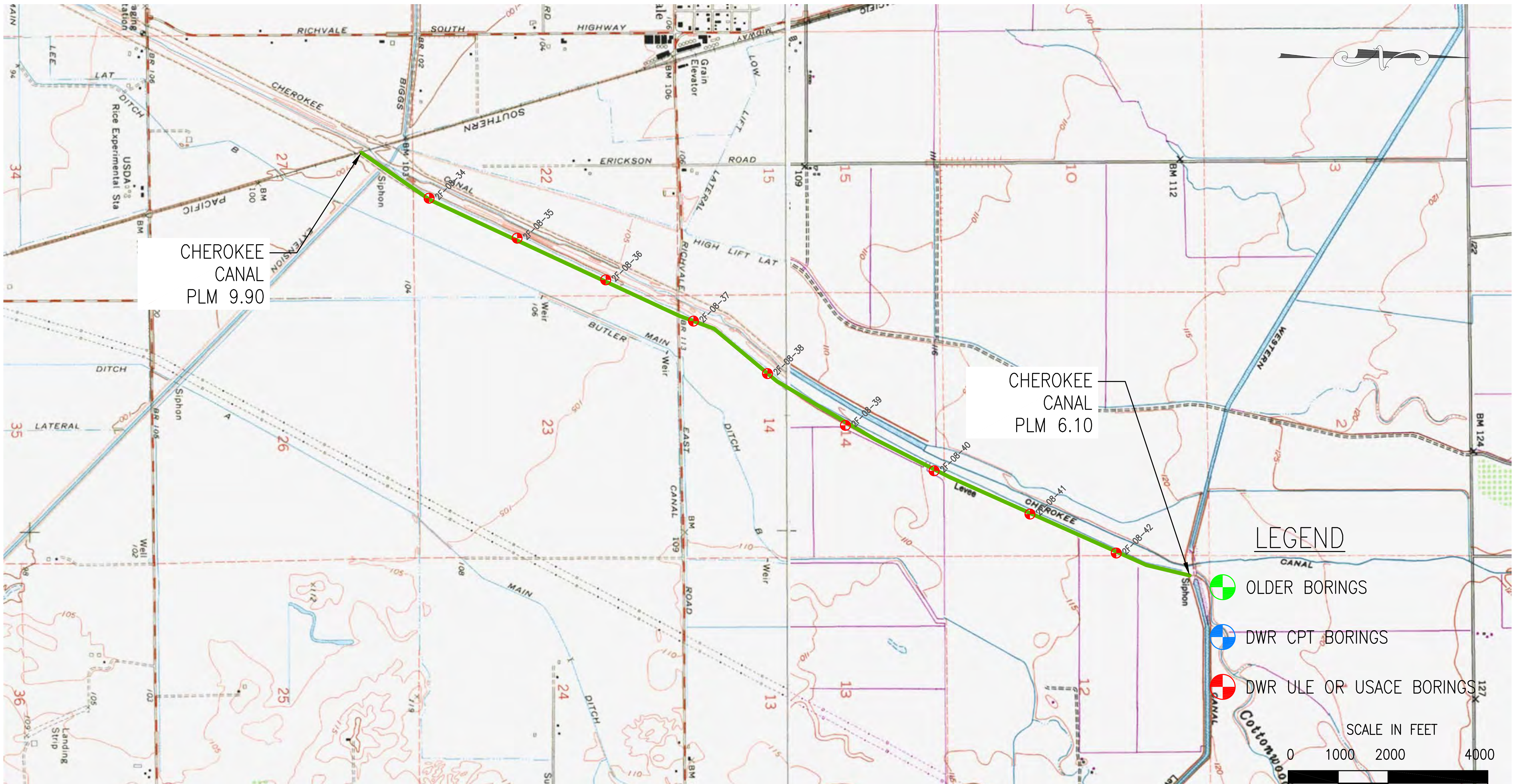
Analysis By: T. Huynh
Checked By: E.W. James/J.M.
Date: Updated 9/14/20

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
39.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
42.90	0.0004	0.9996	0.0000	1.0000	0.0000	1.0000	0.0394	0.9606	0.0398	0.9602
50.25	0.2366	0.7634	0.0000	1.0000	0.0005	0.9995	0.0747	0.9253	0.2940	0.7060
57.60	0.6780	0.3220	0.0000	1.0000	0.6959	0.3041	0.1145	0.8855	0.9133	0.0867
60.60	0.7846	0.2154	0.0000	1.0000	0.8754	0.1246	0.1590	0.8410	0.9774	0.0226



ENCLOSURE F

CHEROKEE CANAL LEFT LEVEE BORING LOGS



SUTTER BASIN FEASIBILITY STUDY – CHEROKEE CANAL

STUDY MAP SHOWING EXPLORATORY BORINGS

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER AND BUTTE COUNTY CALIFORNIA

SUTTER FEASIBILITY STUDY

GEOTECHNICAL ENGINEERING APPENDIX
STUDY MAP SHOWING
GEOTECHNICAL EXPLORATIONS
CHEROKEE CANAL

DATE: 7 DEC 2012
SCALE: 1" = 2000'

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS
1. PROJECT Sutter Basin Feasibility Study Richvale, CA		9. COORDINATE SYSTEM State Plane CA Zone 2 ; HORIZONTAL NAD83 ; VERTICAL NAVD88	
		10. SIZE AND TYPE OF BIT 8-inch HSA	
2. HOLE NUMBER 2F-08-34	LOCATION COORDINATES N 2,301,046.0 E 6,637,006.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer	
3. DRILLING AGENCY R&M Environmental		12. TOTAL SAMPLES ; DISTURBED 3 ; UNDISTURBED 11	
4. NAME OF DRILLER Kelin Jensen		13. TOTAL NUMBER CORE BOXES	
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED	DEG FROM VERTICAL --- ; BEARING	14. ELEVATION GROUND WATER INITIAL 87.0 12/2/08 ; STATIC	
		15. DATE STARTED 12/2/08 ; DATE COMPLETED 12/2/08	
6. THICKNESS OF OVERBURDEN 41.5		16. ELEVATION TOP OF BORING 112.0	
7. DEPTH DRILLED INTO ROCK 0.0		17. TOTAL CORE RECOVERY FOR BORING N/A	
8. TOTAL DEPTH OF BORING 41.5		18. LOGGED BY C. Payton ; CHECKED BY E. James	

ELEV	DEPTH	SAMPLE	Blows/0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							LEVEE FILL LEAN CLAY (CL); moist; dark grayish brown (10YR 4/2); trace sand; low plasticity fines; firm; PP = 2.25 tsf.										
108.0	4.0		5 6 7	13	9			83									
106.5	5.5		3 6 8	14	11		LEVEE FILL Silty CLAY (CL-ML); moist; yellowish brown (10YR 5/6); low plasticity fines; firm; PP = 2.0 tsf.	50	L1	0	30	70	41	25		CL	
			2 3 4	7	7		LEVEE FILL FAT CLAY with Sand (CH); moist; dark grayish brown (10YR 4/2); trace sand; low plasticity fines; firm; PP = 1.25 tsf.	60									
			4 7 7	14	9		Very dark gray (10YR 3/1); PP = 1.75 tsf.	60	L2						26		
			3 5 8	13	10		Very dark gray (5Y 3/1); PP = 1.25 tsf.	33	L3				49	35			
			4 6 8	14	14		Dark gray (10YR 4/1); low plasticity fines; hard; PP = 1.25 to 1.5 tsf.	67									
97.0	15.0							96	S-1	0	23	77	50	33		CH	
			7 14 21	35	30		Silty CLAY (CL-ML); moist; brown (10YR 5/3); PP = 4 to 4.5 tsf.	93	L4						24		
			8 10 12	22	23			100									
			12 14 15	29	25		PP > 4.5 tsf.	100									
							PP > 4.5 tsf.										

0.0 2.5 5.0 7.5 10.0 12.5 15.0 17.5 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION	Sacramento District
SHEET	2 OF 2 SHEETS
PROJECT	Sutter Basin Feasibility Study Richvale, CA
COORDINATE SYSTEM	State Plane CA Zone 2
HORIZONTAL	NAD83
VERTICAL	NAVD88
LOCATION COORDINATES	N 2,301,046.0 E 6,637,006.0
ELEVATION TOP OF BORING	112.0

ELEV	DEPTH	Blows/ 0.5 ft	N_f	N_{60}	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
					Silty CLAY (CL-ML); moist; brown (10YR 5/3); PP = 4 to 4.5 tsf. *(continued)*										
90.0	22.0				SILTY SAND (SM); wet; brown (10YR 4/3); fine to coarse sand; 10% low plasticity fines; dense.										
		10 23 26	49	48		33	L5	0	75	26			23		
81.5	30.5	26			Moist; yellowish brown (10YR 5/6).										
81.0	31.0	50 50/5"	R	R	Clayey GRAVEL (GC); wet; yellowish brown (10YR 5/6); fine, subrounded gravel; 35% low plasticity fines; 10% fine to coarse sand; very dense.	33	L6						14		
					ELASTIC SILT (MH); moist; yellowish brown (10YR 5/4) light yellowish brown (10YR 6/4); low plasticity fines; hard; Well indurated. Mottled with iron oxide staining..										
		19 50/5"	R	R		67	L7				53	14			
73.7	38.3				SAND (SW); moist; dark yellowish brown (10YR 4/4); fine sand; dense; Silt lenses.										
70.5	41.5	7 20 26	46	45	Grayish brown (10YR 5/2); medium sand; 0% fines.	100	L8								

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88
	10. SIZE AND TYPE OF BIT 8-inch HSA
2. HOLE NUMBER 2F-08-35; LOCATION COORDINATES N 2,302,812.0 E 6,637,807.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY R&M Environmental	12. TOTAL SAMPLES; DISTURBED 3; UNDISTURBED 16
4. NAME OF DRILLER Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 97.5 12/3/08; STATIC 98.5 12/3/08
	15. DATE STARTED 12/3/08; DATE COMPLETED 12/3/08
6. THICKNESS OF OVERBURDEN 41.5	16. ELEVATION TOP OF BORING 113.0
7. DEPTH DRILLED INTO ROCK 0.0	17. TOTAL CORE RECOVERY FOR BORING N/A
8. TOTAL DEPTH OF BORING 41.5	18. LOGGED BY C. Payton; CHECKED BY E. James

ELEV	DEPTH	Blows/ 0.5 ft	N_f	N_{60}	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
					LEVEE FILL LEAN CLAY (CL); moist; dark brown (10YR 3/3) yellowish brown (10YR 5/4); low plasticity fines; firm; Road base gravel.										
		4, 6, 8	14	9	Yellowish brown (10YR 5/4); PP = 1.25 tsf.	100	L1				34	19			
108.3	4.7	2, 3, 5	8	6	LEVEE FILL SILTY CLAY (CL-ML); moist; dark brown (10YR 3/3); low plasticity fines; firm; PP = 1.5 tsf.	93									
		2, 4, 4	8	8		100									
105.5	7.5	2, 3, 4	7	5	LEVEE FILL LEAN CLAY (CL); very dark gray (10YR 3/1); firm; PP = 0.7 tsf.	100									
		3, 5, 7	12	10	Dark gray (10YR 4/1); PP = 1.25 tsf, TV = 0.37 kg/cm^2.	100									
		4, 6, 8	14	14	Dark gray (10YR 4/1).	100									
101.5	11.5				SILTY CLAY (CL-ML); moist; yellowish brown (10YR 5/4); very hard; PP = 1.25 tsf.	84	S-1								
98.3	14.7	8, 12, 16	28	24	SILTY SAND (SM); wet; brown (10YR 4/3); 60% fine to medium sand; 40% low plasticity fines; medium dense.	100									
		10, 16, 19	35	36		100	L2	0	69	32					
95.0	18.0				SANDY SILT (ML); moist; yellowish brown (10YR 5/4); dense.										
94.5	18.5	12, 22, 24	46	40	SANDY SILT with Gravel (SM); wet; dark yellowish brown (10YR 4/4); 30% fine sand; dense.	87									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION	SHEET 2 OF 2 SHEETS
Sacramento District	

PROJECT	COORDINATE SYSTEM	HORIZONTAL	VERTICAL
Sutter Basin Feasibility Study Richvale, CA	State Plane CA Zone 2	NAD83	NAVD88

LOCATION COORDINATES	ELEVATION TOP OF BORING
N 2,302,812.0 E 6,637,807.0	113.0

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
			26 36 40	76	74		SANDY SILT with Gravel (SM); wet; dark yellowish brown (10YR 4/4); 30% fine sand; dense. *(continued)* Very dense.	93	L3	2	86	12			14		
90.0	23.0						CLAYEY SAND (SC); moist; dark yellowish brown (10YR 4/4); 10% fine to medium sand; hard.										
			4 17 29	46	45			0									
			16 50	R	R			67	L4	0	51	49			25		
84.0	29.0						SANDY SILT (ML); yellowish brown (10YR 5/6); 40% fine sand; hard.										
			11 27 40	67	65			93	L5	0	61	39	38	19		SC	
77.2	35.8						SILTY SAND (SM); wet; yellowish brown (10YR 5/4); 40% fine to medium sand; 35% low plasticity fines; dense.										
			9 16 26	42	43			100	L6								
72.8	40.2		3 7 24	31	32		SILT (ML); moist; yellowish brown (10YR 5/8); trace sand; firm.	67	L7								
71.5	41.5																

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT: Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM: State Plane CA Zone 2; HORIZONTAL: NAD83; VERTICAL: NAVD88
	10. SIZE AND TYPE OF BIT: 8-inch HSA
2. HOLE NUMBER: 2F-08-36; LOCATION COORDINATES: N 2,304,588.0 E 6,638,644.0	11. MANUFACTURER'S DESIGNATION OF DRILL: Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY: R&M Environmental	12. TOTAL SAMPLES; DISTURBED: ; UNDISTURBED: 8
4. NAME OF DRILLER: Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING: ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL: ---; BEARING:	14. ELEVATION GROUND WATER: INITIAL 84.5 12/4/08; STATIC
	15. DATE STARTED: 12/4/08; DATE COMPLETED: 12/4/08
6. THICKNESS OF OVERBURDEN: 39.5	16. ELEVATION TOP OF BORING: 114.5
7. DEPTH DRILLED INTO ROCK: 0.0	17. TOTAL CORE RECOVERY FOR BORING: N/A
8. TOTAL DEPTH OF BORING: 39.5	18. LOGGED BY: C. Payton; CHECKED BY: E. James

ELEV	DEPTH	Blows/0.5 ft	N_f	N_{60}	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
					LEAN CLAY (CL); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
		3, 8, 9	17			100					50	37	21		
		9, 2, 8	10			80									
		3, 4, 6	10			67									
						100					52	37			
						100									
103.5	11.0				SILT (ML).	100									
						84									
						100		0	51	49	32	18		SC	
					SILTY CLAY with Sand (ML). SILT (ML).	100									
					SILTY CLAY (ML).	87									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION: Sacramento District		SHEET 2 OF 2 SHEETS
PROJECT: Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM: State Plane CA Zone 2	HORIZONTAL: NAD83 / VERTICAL: NAVD88
LOCATION COORDINATES: N 2,304,588.0 E 6,638,644.0	ELEVATION TOP OF BORING: 114.5	

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SILT (ML). *(continued)*	93					48	26			
								0									
								67		0	19	81	36	22		CL	
84.5	30.0						SAND (SW).										
								93									
								100									
76.5	38.0						SAND (SW).										
75.0	39.5																

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS
1. PROJECT Sutter Basin Feasibility Study Richvale, CA		9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88	
		10. SIZE AND TYPE OF BIT 8-inch HSA	
2. HOLE NUMBER 2F-08-37	LOCATION COORDINATES N 2,306,348.0 E 6,639,470.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer	
3. DRILLING AGENCY R&M Environmental		12. TOTAL SAMPLES; DISTURBED; UNDISTURBED 6	
4. NAME OF DRILLER Kelin Jensen		13. TOTAL NUMBER CORE BOXES	
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED	DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 96.5 12/4/08; STATIC 99.5 12/4/08	
		15. DATE STARTED 12/4/08; DATE COMPLETED 12/4/08	
6. THICKNESS OF OVERBURDEN 40.5		16. ELEVATION TOP OF BORING 117.5	
7. DEPTH DRILLED INTO ROCK 0.0		17. TOTAL CORE RECOVERY FOR BORING N/A	
8. TOTAL DEPTH OF BORING 40.5		18. LOGGED BY C. Payton; CHECKED BY E. James	

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							ELASTIC SILT (MH); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
								83		2	22	76	42	23		CL	
								50									
								60									
								60							23		
108.8	8.7						FAT CLAY (CL).										
								33									
								67									
105.5	12.0						SANDY SILT (ML).										
								96					45	23			
								93									
101.6	15.9						CLAYEY SAND with Gravel (SC).										
								100									
99.1	18.4						SANDY SILT (ML).										
								100									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)	INSTALLATION Sacramento District			SHEET 2 OF 2 SHEETS
PROJECT Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM State Plane CA Zone 2	HORIZONTAL NAD83	VERTICAL NAVD88	
LOCATION COORDINATES N 2,306,348.0 E 6,639,470.0	ELEVATION TOP OF BORING 117.5			

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SANDY SILT (ML). *(continued)*			0	45	55			29		
94.5	23.0						SAND (SW).										
								33									
								33									
84.2	33.3						SILT (ML).										
								67									
77.0	40.5																

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLING LOG	DIVISION: South Pacific	INSTALLATION: Sacramento District	SHEET 1 OF 2 SHEETS

Field	Value
1. PROJECT	Sutter Basin Feasibility Study Richvale, CA
2. HOLE NUMBER	2F-08-38
LOCATION COORDINATES	N 2,307,833.0 E 6,640,524.0
3. DRILLING AGENCY	R&M Environmental
4. NAME OF DRILLER	Kelin Jensen
5. DIRECTION OF BORING	☒ VERTICAL ☐ INCLINED
DEG FROM VERTICAL	---
BEARING	
6. THICKNESS OF OVERBURDEN	42.0
7. DEPTH DRILLED INTO ROCK	0.0
8. TOTAL DEPTH OF BORING	42.0
9. COORDINATE SYSTEM	State Plane CA Zone 2
HORIZONTAL	NAD83
VERTICAL	NAVD88
10. SIZE AND TYPE OF BIT	8-inch HSA
11. MANUFACTURER'S DESIGNATION OF DRILL	Mobile B-61 with 140-lb automatic trip slide hammer
12. TOTAL SAMPLES	DISTURBED: — UNDISTURBED: 9
13. TOTAL NUMBER CORE BOXES	
14. ELEVATION GROUND WATER	INITIAL 99.5 12/5/08 STATIC
15. DATE STARTED	12/5/08
DATE COMPLETED	12/5/08
16. ELEVATION TOP OF BORING	119.0
17. TOTAL CORE RECOVERY FOR BORING	N/A
18. LOGGED BY	C. Payton
CHECKED BY	E. James

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SILT (ML); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
116.0	3.0							83		0	47	53	40	23		CL	
115.5	3.5						LEAN CLAY (CL).										
							ELASTIC SILT with Sand (MH).										
114.5	4.5							50									
114.0	5.0						LEAN CLAY (CL).										
113.7	5.3						ELASTIC SILT with Sand (MH).										
							LEAN CLAY (CL).										
112.9	6.1						ELASTIC SILT with Sand (MH).	60									
								60		0	38	62			27		
110.3	8.7						LEAN CLAY (CL).										
								33									
								67									
								96									
105.0	14.0						SANDY SILT (ML).										
								93		0	28	72	34	14		CL	
								100									
								100									
100.0	19.0						SAND (SW).										

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)	INSTALLATION Sacramento District	SHEET 2 OF 2 SHEETS
PROJECT Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM State Plane CA Zone 2	HORIZONTAL NAD83 / VERTICAL NAVD88
LOCATION COORDINATES N 2,307,833.0 E 6,640,524.0	ELEVATION TOP OF BORING 119.0	

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SAND (SW). *(continued)*								24		
97.8	21.2						SANDY SILT (ML).										
95.7	23.3						SAND (SW).										
								33		0	73	27			25		
91.0	28.0						SANDY SILT (ML).										
								33					30	13			
87.9	31.1						SAND (SW).										
84.5	34.5						SILT (ML).										
								67									
										0	33	67			39		
								100									
77.0	42.0																

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88
	10. SIZE AND TYPE OF BIT 8-inch HSA
2. HOLE NUMBER 2F-08-39; LOCATION COORDINATES N 2,309,396.0 E 6,641,562.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY R&M Environmental	12. TOTAL SAMPLES; DISTURBED; UNDISTURBED 7
4. NAME OF DRILLER Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 100.0 12/8/09 STATIC 103.0 12/8/09
	15. DATE STARTED 12/8/09; DATE COMPLETED 12/8/09
6. THICKNESS OF OVERBURDEN 42.0	16. ELEVATION TOP OF BORING 121.0
7. DEPTH DRILLED INTO ROCK 0.0	17. TOTAL CORE RECOVERY FOR BORING N/A
8. TOTAL DEPTH OF BORING 42.0	18. LOGGED BY C. Payton; CHECKED BY E. James

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SANDY SILT with Gravel (ML); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
							SILT (ML).	83		0	35	65	35	22		CL	
117.3	3.7						ELASTIC SILT (MH).										
116.5	4.5						Silty CLAY (CL).	50									
115.0	6.0							60									
114.5	6.5						ELASTIC SILT (MH).										
							LEAN CLAY (CL).										
							Silty CLAY (CL).	60					52	28			
								33									
								67									
108.5	12.5						SILT (ML).	96		0	37	63	41	26	25	CL	
								93		0	32	68	NV	NP		ML	
								100									
								100									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION: Sacramento District			SHEET 2 OF 2 SHEETS
PROJECT: Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM: State Plane CA Zone 2	HORIZONTAL: NAD83	VERTICAL: NAVD88
LOCATION COORDINATES: N 2,309,396.0 E 6,641,562.0	ELEVATION TOP OF BORING: 121.0		

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS	Depth scale
							SILT (ML). *(continued)*											20.0
100.5	20.5						Silty SAND (SM).			0	72	28			24			
																		22.5
97.5	23.5						Sandy SILT (ML).											
								33										25.0
																		27.5
92.5	28.5						SAND (SW).											
								33										30.0
																		32.5
								67										35.0
																		37.5
								100										40.0
79.2	41.8						SILT (ML).											
79.0	42.0																	

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88
	10. SIZE AND TYPE OF BIT 8-inch HSA
2. HOLE NUMBER 2F-08-40; LOCATION COORDINATES N 2,311,175.0 E 6,642,474.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY R&M Environmental	12. TOTAL SAMPLES; DISTURBED; UNDISTURBED 5
4. NAME OF DRILLER Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 106.0 12/9/09; STATIC 107.5 12/9/09
	15. DATE STARTED 12/9/09; DATE COMPLETED 12/9/09
6. THICKNESS OF OVERBURDEN 40.0	16. ELEVATION TOP OF BORING 122.5
7. DEPTH DRILLED INTO ROCK 0.0	17. TOTAL CORE RECOVERY FOR BORING N/A
8. TOTAL DEPTH OF BORING 40.0	18. LOGGED BY C. Payton; CHECKED BY E. James

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							ELASTIC SILT (MH); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
119.7	2.8							83		7	25	68	46	33		CL	
119.4	3.1						Silty SAND (SM).										
119.2	3.3						Sandy SILT (ML).										
							LEAN CLAY (CL).										
								50									
								60									
							LEAN CLAY (CL).										
								60		0	38	62	34	19	34	CL	
113.5	9.0						ELASTIC SILT (MH).	33									
								67									
							ELASTIC SILT (MH).	96									
108.5	14.0						Clayey SAND (SC).										
107.0	15.5						ELASTIC SILT (MH).	93		8	41	51	43	16		ML	
								100									
								100									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION	Sacramento District
SHEET	2 OF 2 SHEETS
PROJECT	Sutter Basin Feasibility Study Richvale, CA
COORDINATE SYSTEM	State Plane CA Zone 2
HORIZONTAL	NAD83
VERTICAL	NAVD88
LOCATION COORDINATES	N 2,311,175.0 E 6,642,474.0
ELEVATION TOP OF BORING	122.5

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							ELASTIC SILT (MH). *(continued)*										
102.0	20.5						SAND (SP).										
101.3	21.2						Sandy SILT (ML).										
	25.0–26.5							33									
	30.0–31.5							33		0	27	73			24		
							SILT (ML).										
	35.0–36.5							67									
82.5	40.0																

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88
	10. SIZE AND TYPE OF BIT 8-inch HSA
2. HOLE NUMBER 2F-08-41; LOCATION COORDINATES N 2,313,098.0 E 6,643,342.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY R&M Environmental	12. TOTAL SAMPLES; DISTURBED; UNDISTURBED 7
4. NAME OF DRILLER Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 107.0 12/9/09 STATIC 110.0 12/9/09
	15. DATE STARTED 12/9/09; DATE COMPLETED 12/9/09
6. THICKNESS OF OVERBURDEN 40.5	16. ELEVATION TOP OF BORING 124.5
7. DEPTH DRILLED INTO ROCK 0.0	17. TOTAL CORE RECOVERY FOR BORING N/A
8. TOTAL DEPTH OF BORING 40.5	18. LOGGED BY C. Payton; CHECKED BY E. James

ELEV	DEPTH	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
	0.0	SANDY LEAN CLAY (CL); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
		LEAN CLAY (CL).	83		1	27	73	36	25		CL	
			50									
119.0	5.5	ELASTIC SILT (MH).	60									
118.0	6.5	LEAN CLAY (CL).										
			60					48	33			
			33									
			67									
			96									
			93									
108.5	16.0	Clayey SAND (SC).										
			100		0	32	68			32		
107.2	17.3	CLAYEY SAND (SC-SM).										
107.0	17.5	ELASTIC SILT (MH).										
			100									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

INSTALLATION: Sacramento District			SHEET 2 OF 2 SHEETS
PROJECT: Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM: State Plane CA Zone 2	HORIZONTAL: NAD83	VERTICAL: NAVD88
LOCATION COORDINATES: N 2,313,098.0 E 6,643,342.0	ELEVATION TOP OF BORING: 124.5		

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							ELASTIC SILT (MH). *(continued)*										
100.0	24.5						SILT (ML).						NV	NP			
								33									
								33									
							Sandy SILT (ML).										
								67							33		
							SILT (ML).										
84.0	40.5																

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLING LOG	DIVISION South Pacific	INSTALLATION Sacramento District	SHEET 1 OF 2 SHEETS

1. PROJECT Sutter Basin Feasibility Study Richvale, CA	9. COORDINATE SYSTEM State Plane CA Zone 2; HORIZONTAL NAD83; VERTICAL NAVD88
	10. SIZE AND TYPE OF BIT 8-inch HSA
2. HOLE NUMBER 2F-08-42; LOCATION COORDINATES N 2,314,828.0 E 6,644,123.0	11. MANUFACTURER'S DESIGNATION OF DRILL Mobile B-61 with 140-lb automatic trip slide hammer
3. DRILLING AGENCY R&M Environmental	12. TOTAL SAMPLES; DISTURBED; UNDISTURBED 9
4. NAME OF DRILLER Kelin Jensen	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED; DEG FROM VERTICAL ---; BEARING	14. ELEVATION GROUND WATER INITIAL 110.0 12/10/08 STATIC
	15. DATE STARTED 12/10/08; DATE COMPLETED 12/10/08
6. THICKNESS OF OVERBURDEN 41.6	16. ELEVATION TOP OF BORING 125.5
7. DEPTH DRILLED INTO ROCK 0.0	17. TOTAL CORE RECOVERY FOR BORING N/A
8. TOTAL DEPTH OF BORING 41.6	18. LOGGED BY C. Payton; CHECKED BY E. James

ELEV	DEPTH	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
		SILTY CLAY (CL); ASTM D2488 used for field classifications. Sample decriptions were corrected to reflect lab results and reclassified using ASTM D2487 where applicable. Contacts are dashed where approximate or inferred, and solid where observed..										
			83		0	21	80	49	33		CL	
		LEAN CLAY (CL).	50									
			60									
										24		
			60									
116.0	9.5	SANDY SILT (ML).	33									
			67									
			96									
110.0 ▽	15.5	SAND (SW).	93									
108.8	16.7	SAND with Gravel (SWG).	100		25	64	11			21		
108.2	17.3	SANDY SILT (ML).										
107.1	18.4	SAND (SW).	100									

Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLING LOG (Cont Sheet)

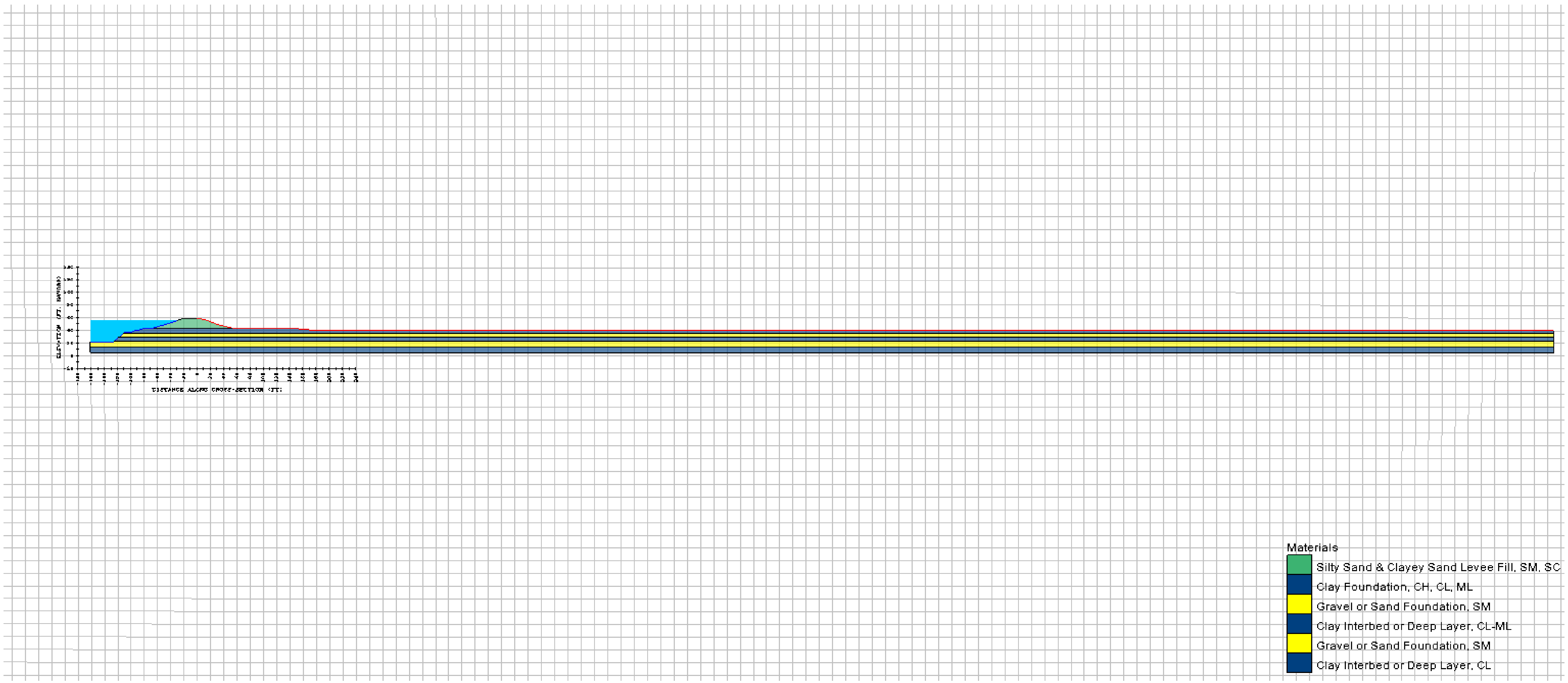
INSTALLATION	Sacramento District			SHEET 2 OF 2 SHEETS	
PROJECT	Sutter Basin Feasibility Study Richvale, CA	COORDINATE SYSTEM	State Plane CA Zone 2	HORIZONTAL NAD83	VERTICAL NAVD88
LOCATION COORDINATES	N 2,314,828.0 E 6,644,123.0	ELEVATION TOP OF BORING	125.5		

ELEV	DEPTH	SAMPLE	Blows/ 0.5 ft	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
							SAND (SW). *(continued)*			1	50	49	40	16		SC	
104.1	21.4						SILT (ML).										
102.0	23.5						SAND (SW).										
								33		0	95	6			32		
95.5	30.0						LEAN CLAY (CL).	33					78	51			
90.5	35.0						SILT (ML).	67							36		
								100									

Depth scale: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

ENCLOSURE G

DETERMINISTIC ANALYSIS



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Silty Sand & Clayey Sand Levee Fill, SM, SC	1.40	0.35	4	125	10	32
2	Clay Foundation, CH, CL, ML	0.028	0.028	1	115	100	28
3	Gravel or Sand Foundation, SM	2.800	0.710	4	125	0	34
4	Clay Interbed or Deep Layer, CL-ML	0.03	0.01	4	115	100	28
5	Gravel or Sand Foundation, SM	2.80	0.71	4	125	0	34
6	Clay Interbed or Deep Layer, CL	0.03	0.01	4	115	100	28

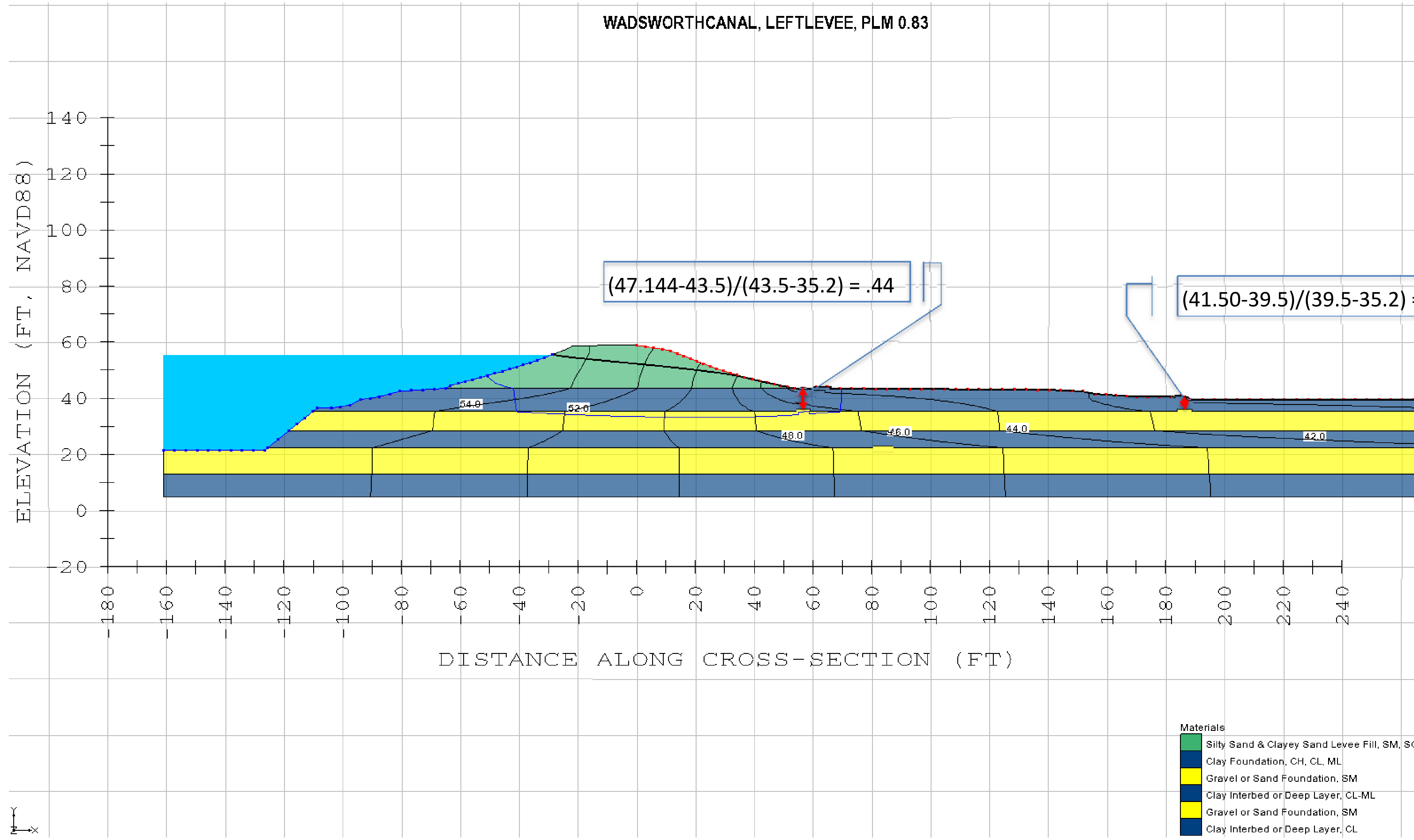
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_001B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 55.5 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, LEFT LEVEE, LM0.83
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Silty Sand & Clayey Sand Levee Fill, SM, SC	1.40	0.35	4	125	10	32
2	Clay Foundation, CH, CL, ML	0.028	0.028	1	115	100	28
3	Gravel or Sand Foundation, SM	2.800	0.710	4	125	0	34
4	Clay Interbed or Deep Layer, CL-ML	0.03	0.01	4	115	100	28
5	Gravel or Sand Foundation, SM	2.80	0.71	4	125	0	34
6	Clay Interbed or Deep Layer, CL	0.03	0.01	4	115	100	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_001B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 55.5 ft.

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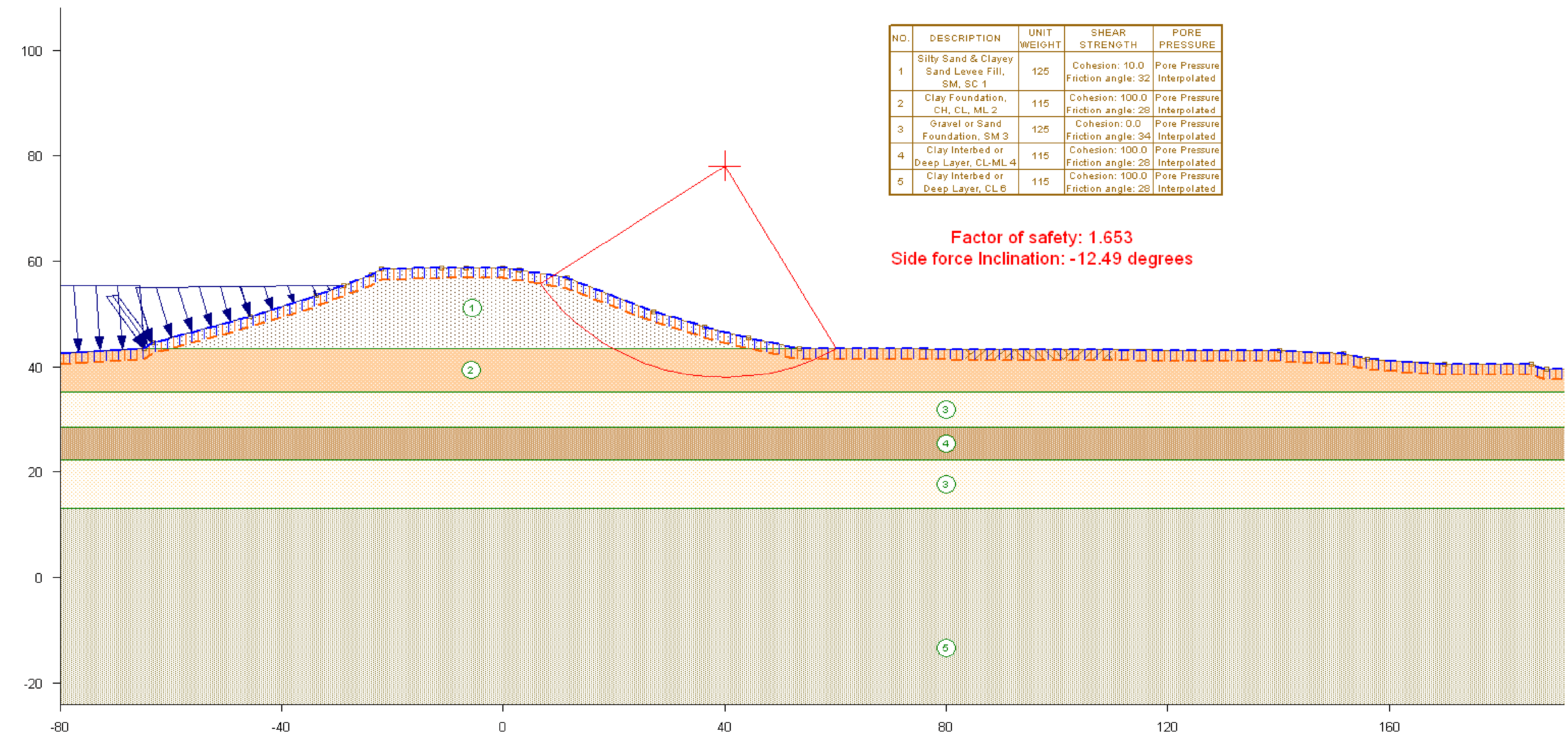
SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, LEFT LEVEE, LM0.83
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Silty Sand & Clayey Sand Levee Fill, SM, SC	1.40	0.35	4	125	10	32
2	Clay Foundation, CH, CL, ML	0.028	0.028	1	115	100	28
3	Gravel or Sand Foundation, SM	2.800	0.710	4	125	0	34
4	Clay Interbed or Deep Layer, CL-ML	0.03	0.01	4	115	100	28
5	Gravel or Sand Foundation, SM	2.80	0.71	4	125	0	34
6	Clay Interbed or Deep Layer, CL	0.03	0.01	4	115	100	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_001B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 55.5 ft.

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SACRAMENTO DISTRICT
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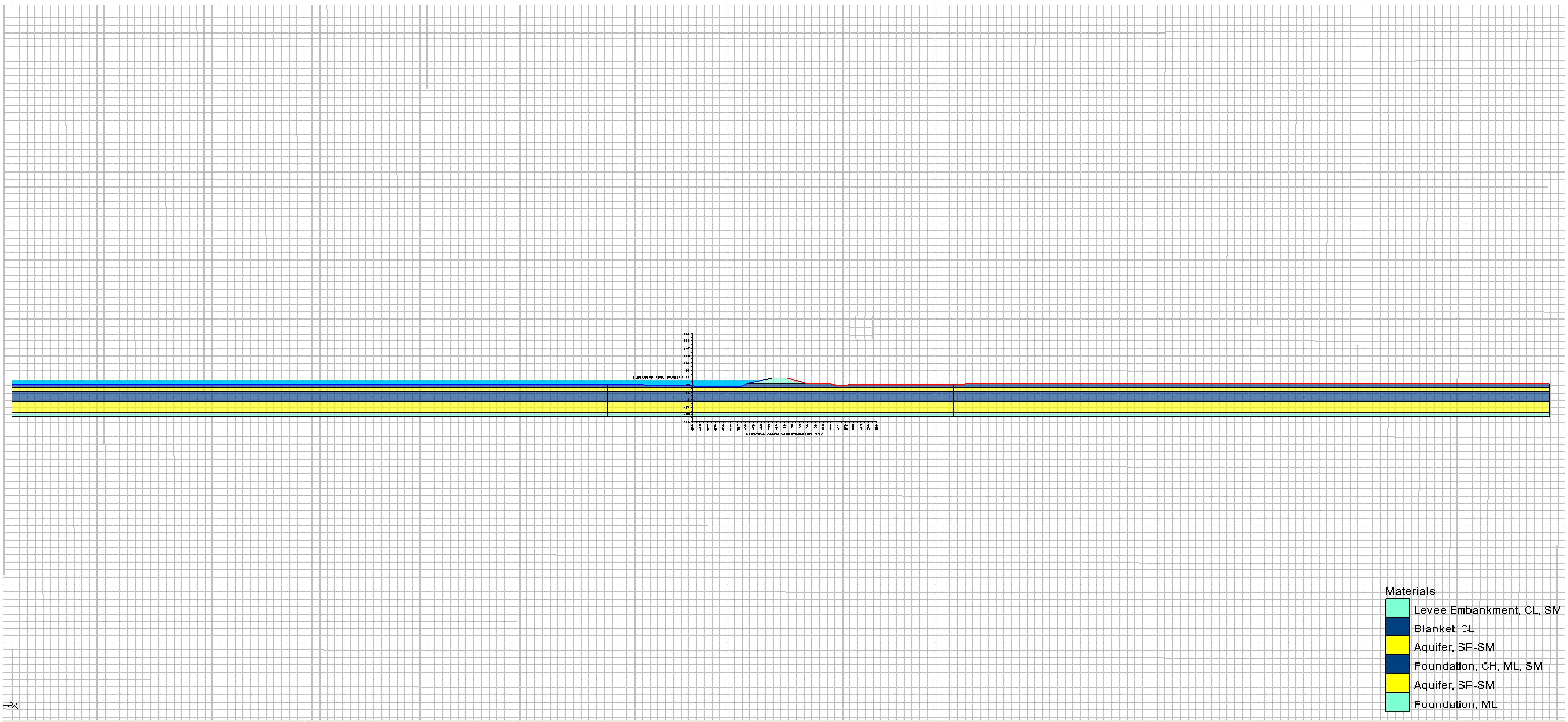
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, LEFT LEVEE, LM0.83
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, CL, SM	0.57	0.14	4	115	50	28
2	Blanket, CL	0.028	0.007	4	115	100	28
3	Aquifer, SP-SM	14.000	1.400	10	125	0	32
4	Foundation, CH, ML, SM	0.57	0.14	4	120	100	28
5	Aquifer, SP-SM	14.00	1.40	10	125	0	32
6	Foundation, ML	0.57	0.14	4	120	10	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_017B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 53.29 ft.

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SACRAMENTO DISTRICT
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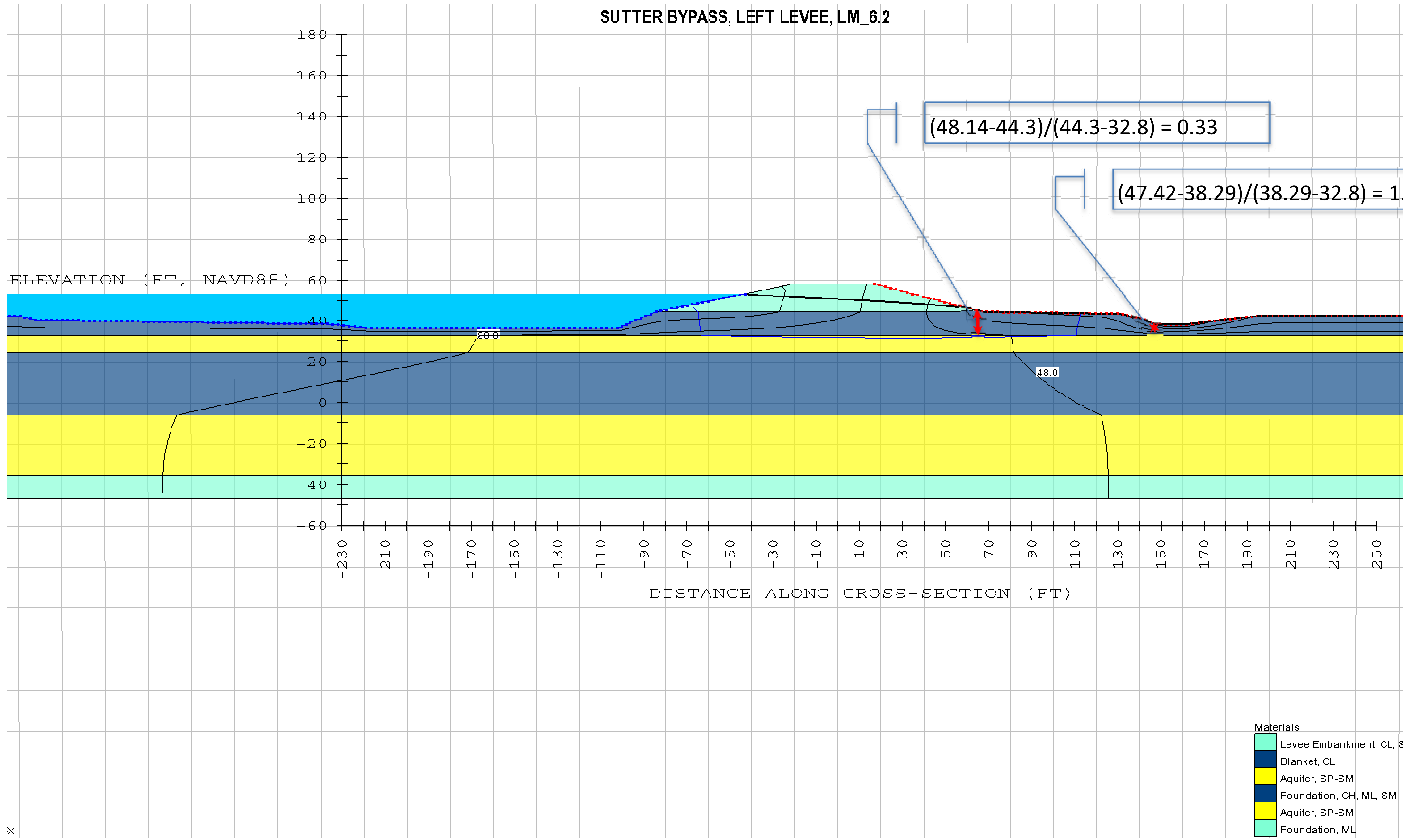
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_6.2, STATION 823+27
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, CL, SM	0.57	0.14	4	115	50	28
2	Blanket, CL	0.028	0.007	4	115	100	28
3	Aquifer, SP-SM	14.000	1.400	10	125	0	32
4	Foundation, CH, ML, SM	0.57	0.14	4	120	100	28
5	Aquifer, SP-SM	14.00	1.40	10	125	0	32
6	Foundation, ML	0.57	0.14	4	120	10	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_017B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 53.29 ft.

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SACRAMENTO DISTRICT
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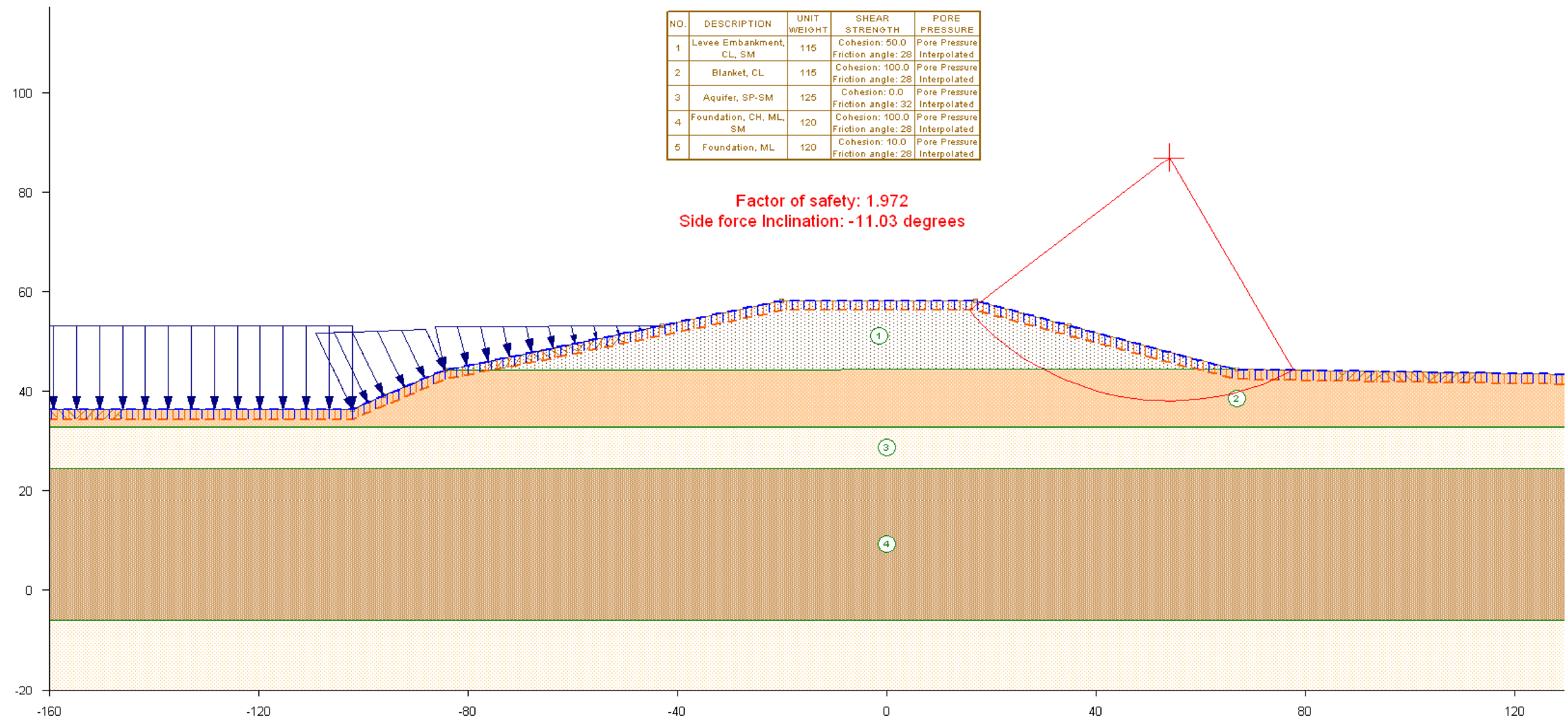
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_6.2, STATION 823+27
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012	SCALE: AS SHOWN	SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, CL, SM	115	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
2	Blanket, CL	115	Cohesion: 100.0 Friction angle: 28	Pore Pressure Interpolated
3	Aquifer, SP-SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
4	Foundation, CH, ML, SM	120	Cohesion: 100.0 Friction angle: 28	Pore Pressure Interpolated
5	Foundation, ML	120	Cohesion: 10.0 Friction angle: 28	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, CL, SM	0.57	0.14	4	115	50	28
2	Blanket, CL	0.028	0.007	4	115	100	28
3	Aquifer, SP-SM	14.000	1.400	10	125	0	32
4	Foundation, CH, ML, SM	0.57	0.14	4	120	100	28
5	Aquifer, SP-SM	14.00	1.40	10	125	0	32
6	Foundation, ML	0.57	0.14	4	120	10	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_017B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 53.29 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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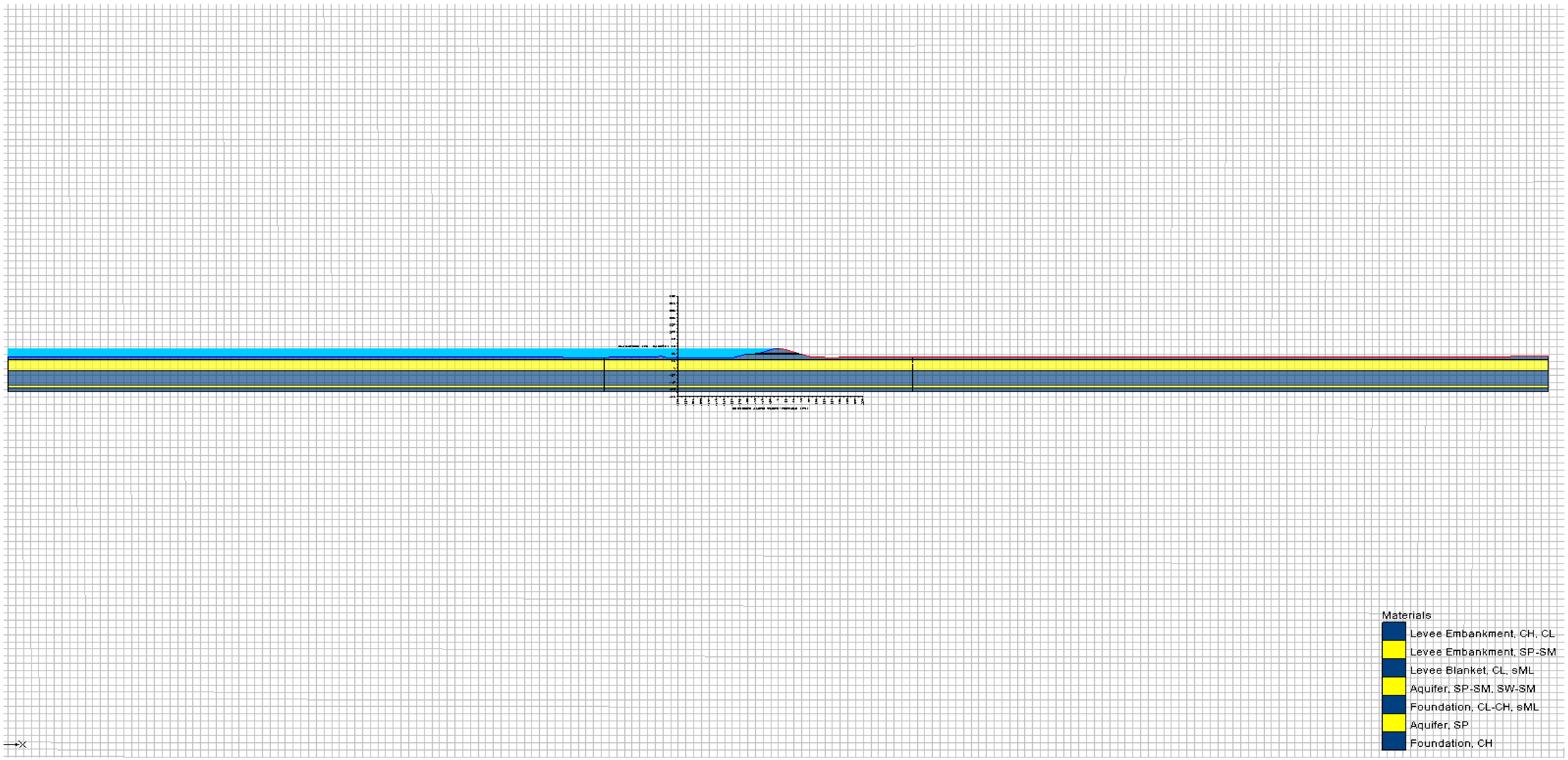
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_6.2, STATION 823+27
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	150	27
3	Blanket, CL, (CL)s	0.028	0.007	4	115	150	27
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_005B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 54.1 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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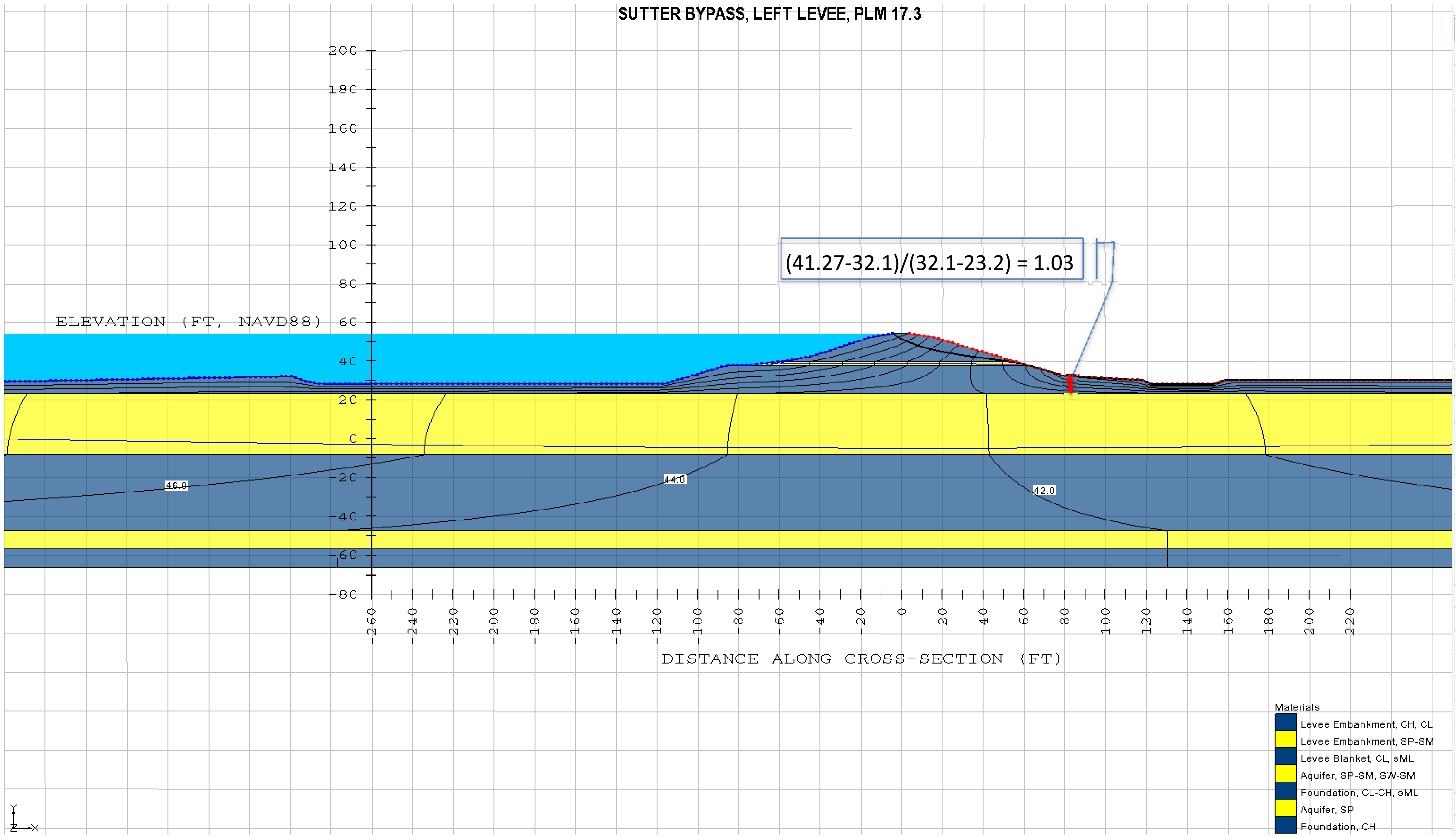
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_17.3 STATION 293+00
COMPLETE SUBSURFACE MODEL

28-Jun-2012	SCALE: AS SHOWN	SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	150	27
3	Blanket, CL, (CL)s	0.028	0.007	4	115	150	27
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

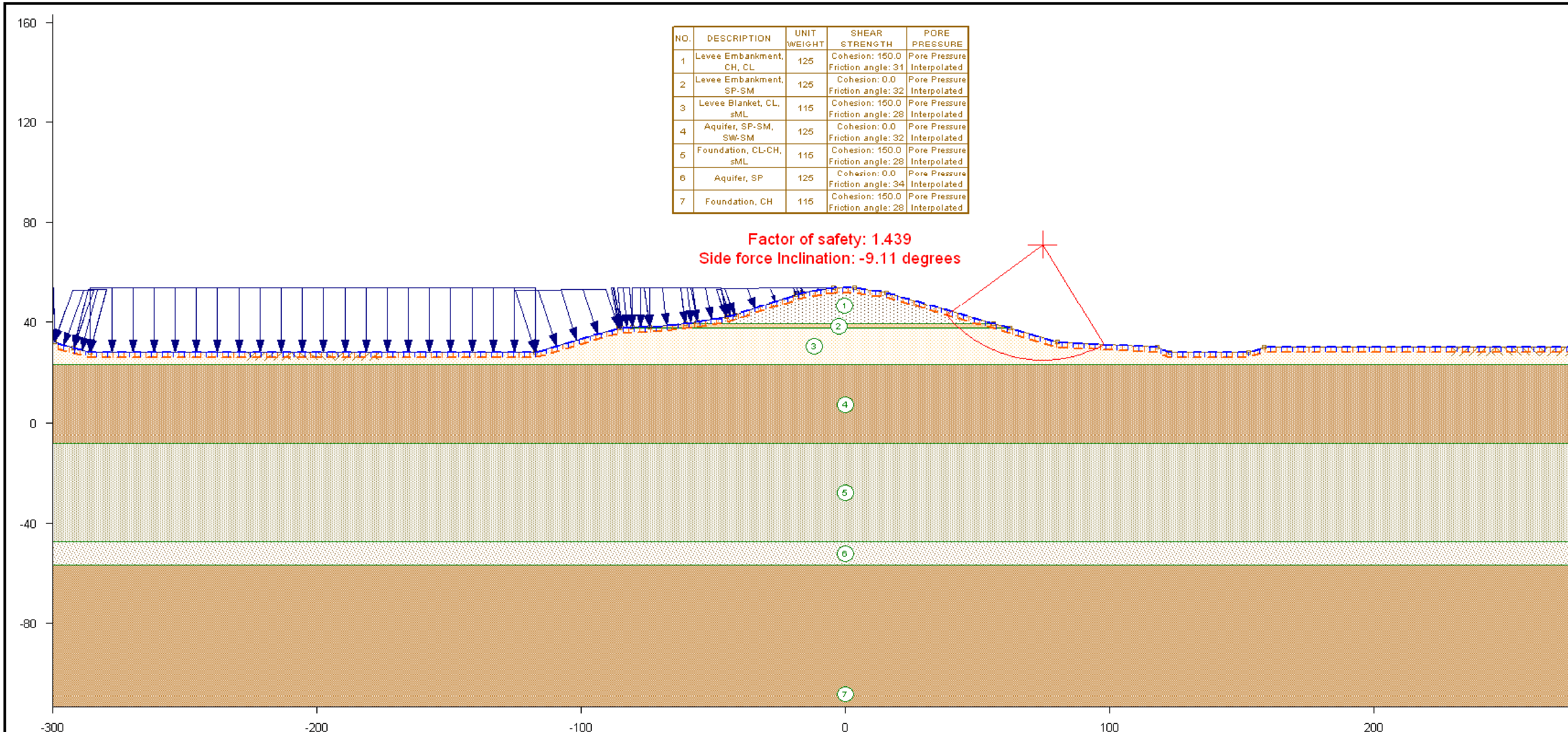
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_005B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 54.1 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_17.3 STATION 293+00
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, CH, CL	125	Cohesion: 150.0 Friction angle: 31	Pore Pressure Interpolated
2	Levee Embankment, SP-SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
3	Levee Blanket, CL, sML	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated
4	Aquifer, SP-SM, SW-SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
5	Foundation, CL-CH, sML	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated
6	Aquifer, SP	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
7	Foundation, CH	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	150	27
3	Blanket, CL, (CL)s	0.028	0.007	4	115	150	27
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

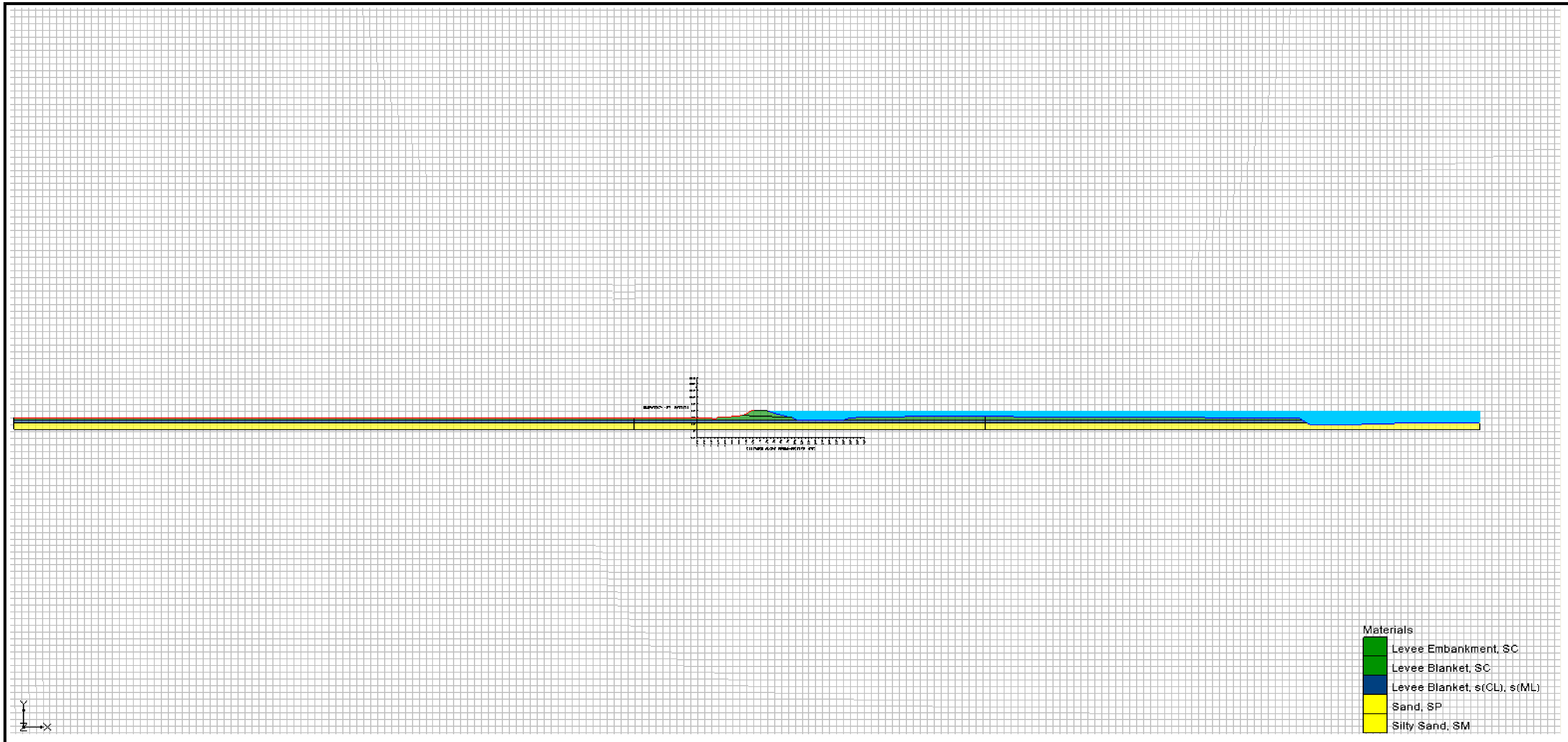
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_005B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 54.1 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS, LM_17.3 STATION 293+00
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, SC	0.28	0.07	4	115.00	10.00	27.00
2	Blanket, SC	0.28	0.07	4	115.00	10.00	27.00
3	Blanket, s(ML), s(CL)	0.57	0.14	4	120.00	10.00	28.00
4	Aquifer, SP	28.00	2.80	10	125.00	0.00	34.00
5	Aquifer, SM	28.00	2.80	10	125.00	0.00	32.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WM0003_002B, 2F-91-7, 5F-73-CD10 (1957 boring)
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 59 ft.

DEPARTMENT OF THE ARMY
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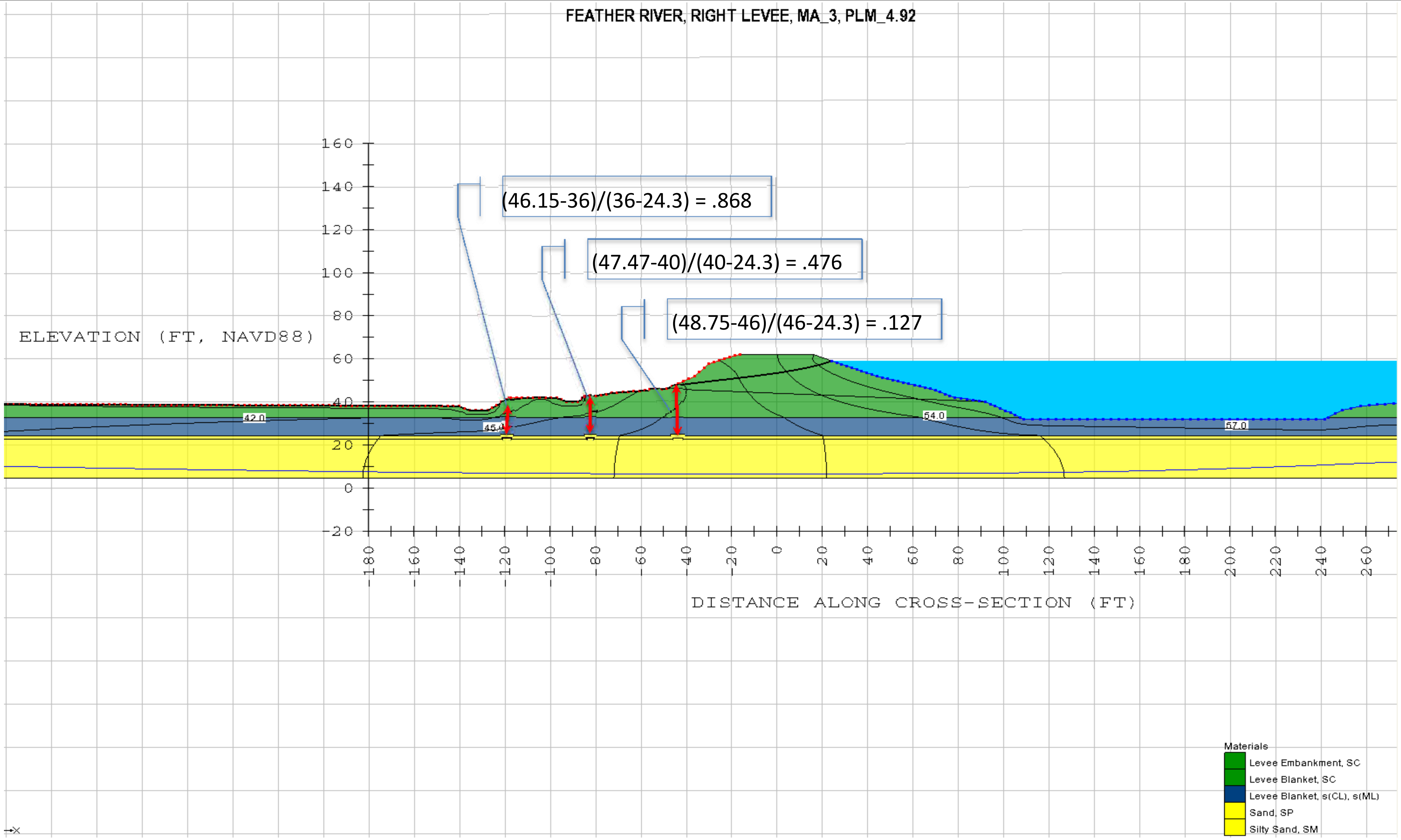
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_4.92, STATION 262+00
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, SC	0.28	0.07	4	115.00	10.00	27.00
2	Blanket, SC	0.28	0.07	4	115.00	10.00	27.00
3	Blanket, s(ML), s(CL)	0.57	0.14	4	120.00	10.00	28.00
4	Aquifer, SP	28.00	2.80	10	125.00	0.00	34.00
5	Aquifer, SM	28.00	2.80	10	125.00	0.00	32.00

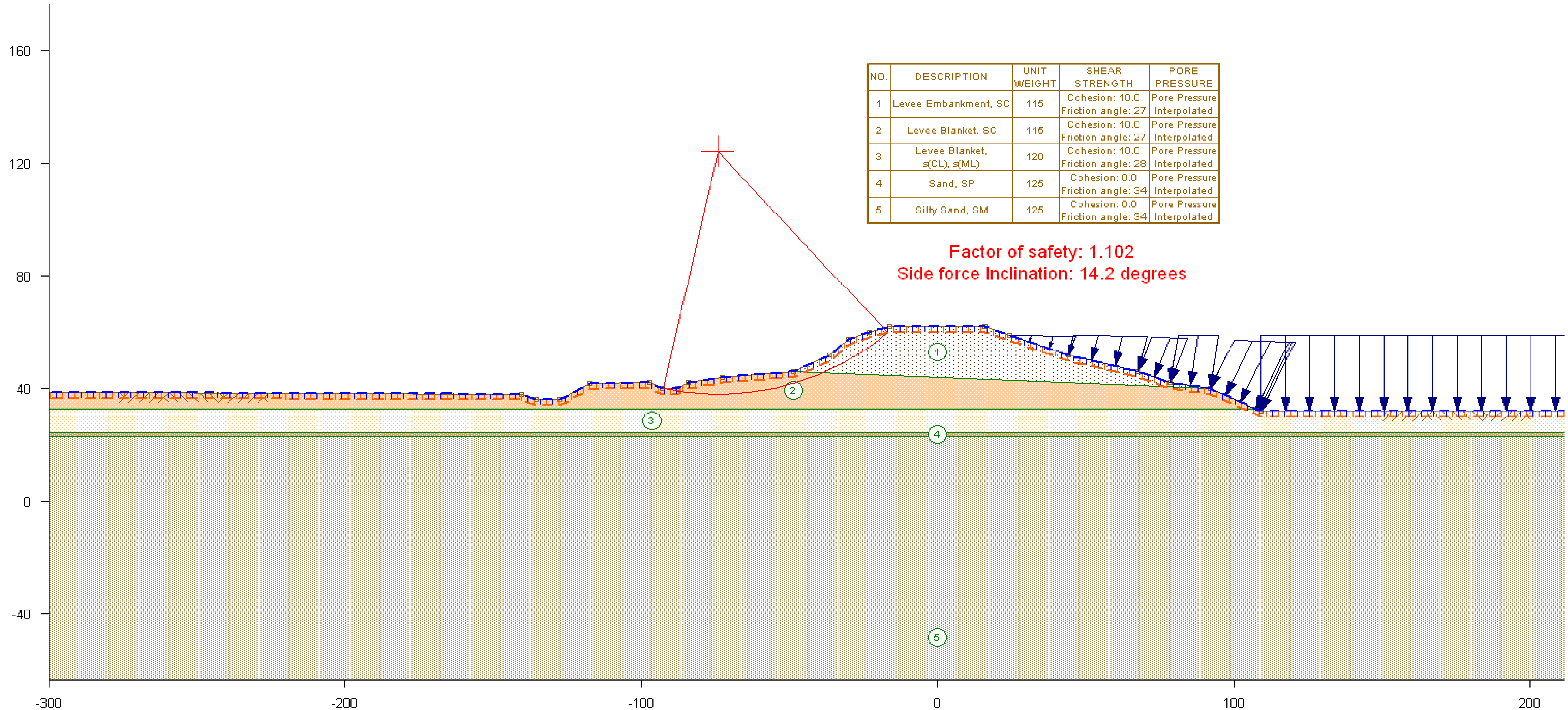
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WM0003_002B, 2F-91-7, 5F-73-CD10 (1957 boring)
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 59 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_4.92, STATION 262+00
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, SC	115	Cohesion: 10.0 Friction angle: 27	Pore Pressure Interpolated
2	Levee Blanket, SC	115	Cohesion: 10.0 Friction angle: 27	Pore Pressure Interpolated
3	Levee Blanket, s(CL), s(ML)	120	Cohesion: 10.0 Friction angle: 28	Pore Pressure Interpolated
4	Sand, SP	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
5	Silty Sand, SM	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, SC	0.28	0.07	4	115.00	10.00	27.00
2	Blanket, SC	0.28	0.07	4	115.00	10.00	27.00
3	Blanket, s(ML), s(CL)	0.57	0.14	4	120.00	10.00	28.00
4	Aquifer, SP	28.00	2.80	10	125.00	0.00	34.00
5	Aquifer, SM	28.00	2.80	10	125.00	0.00	32.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WM0003_002B, 2F-91-7, 5F-73-CD10 (1957 boring)
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 59 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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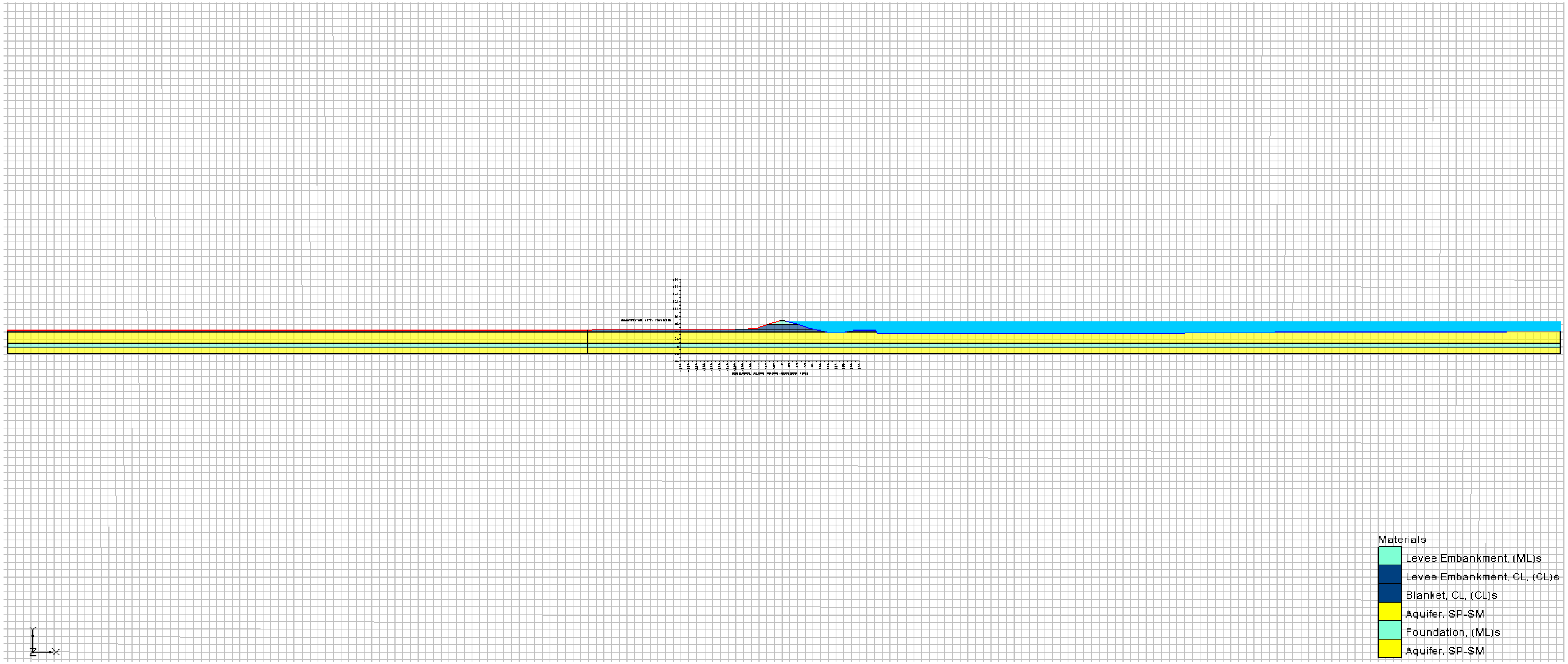
SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER, LM_4.92, STATION 262+00

EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters								
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	200	28
3	Blanket, CL, (CL)s	0.028	0.007	4	115	200	28
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0001_088B, WL0001_007S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 68 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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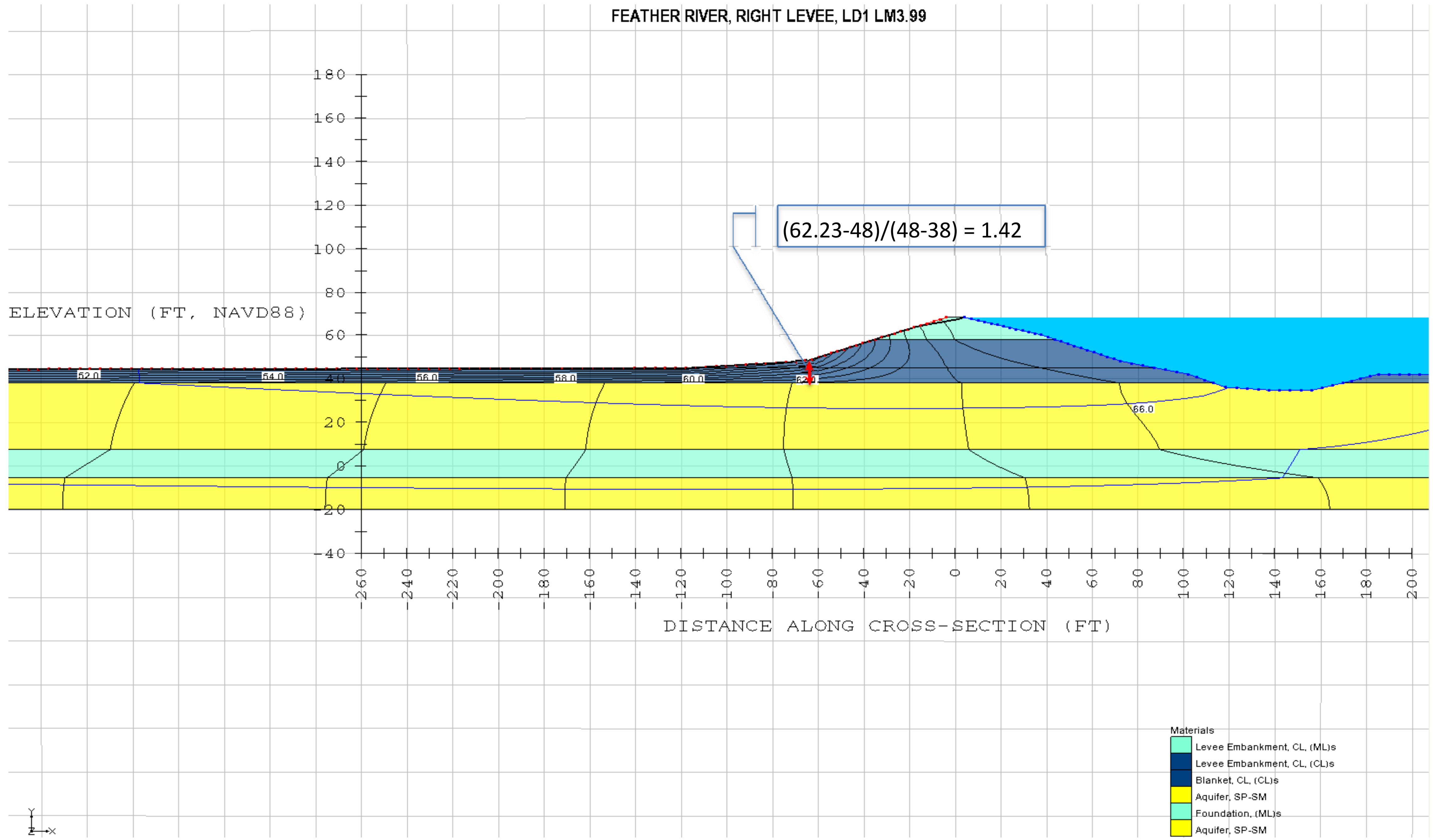
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER LM_3.99, STA 540+00
COMPLETE SUBSURFACE MODEL

28-Jun-2012	SCALE: AS SHOWN	SHEET 1 of 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	200	28
3	Blanket, CL, (CL)s	0.028	0.007	4	115	200	28
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0001_088B, WL0001_007S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 68 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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SACRAMENTO, CALIFORNIA

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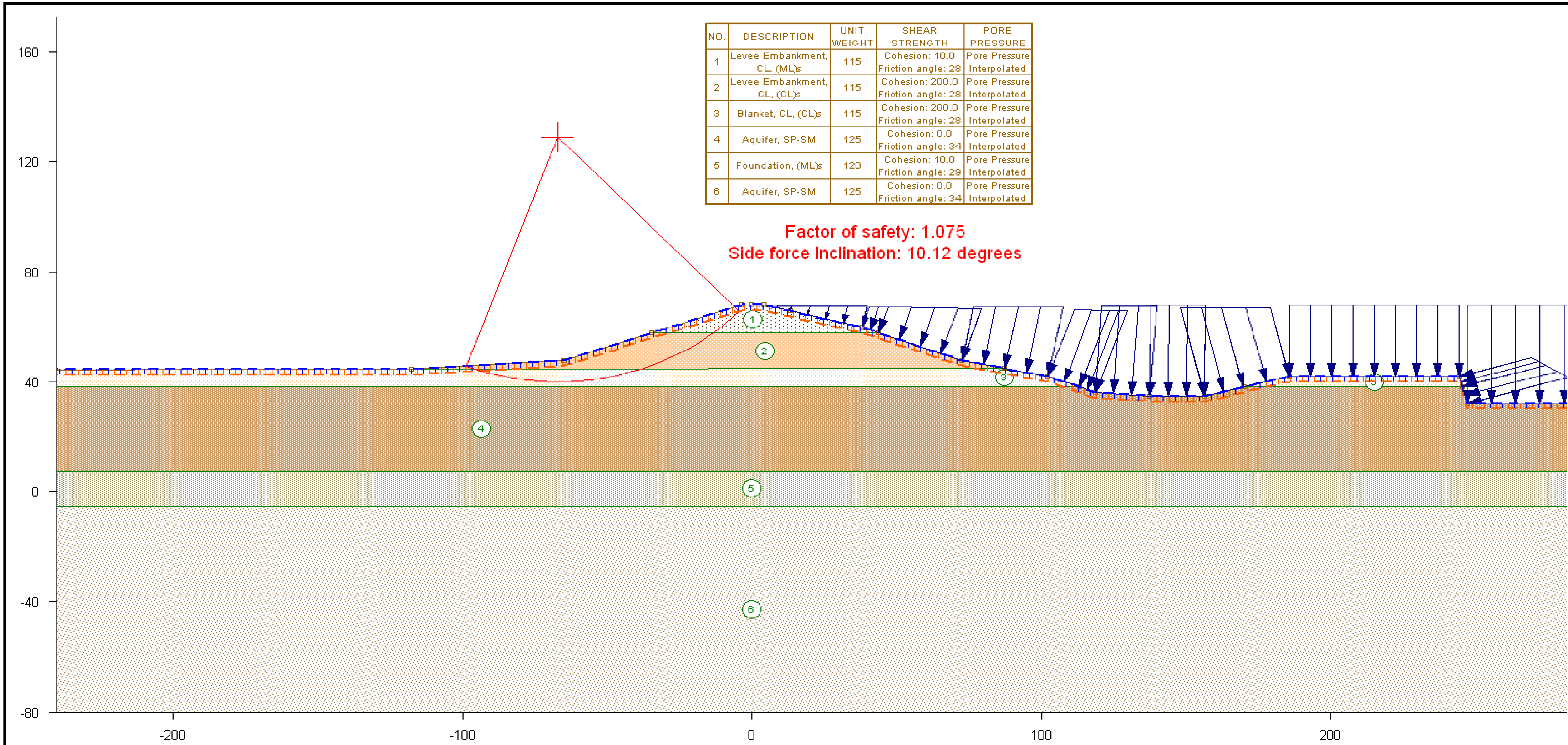
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER LM_3.99, STA 540+00

EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL, (CL)s	0.028	0.007	4	115	200	28
3	Blanket, CL, (CL)s	0.028	0.007	4	115	200	28
4	Aquifer, SP-SM	14.00	1.40	10	125	0	34
5	Foundation, (ML)s	0.57	0.14	4	120	10	28
6	Aquifer, SP-SM	14.00	1.40	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0001_088B, WL0001_007S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 68 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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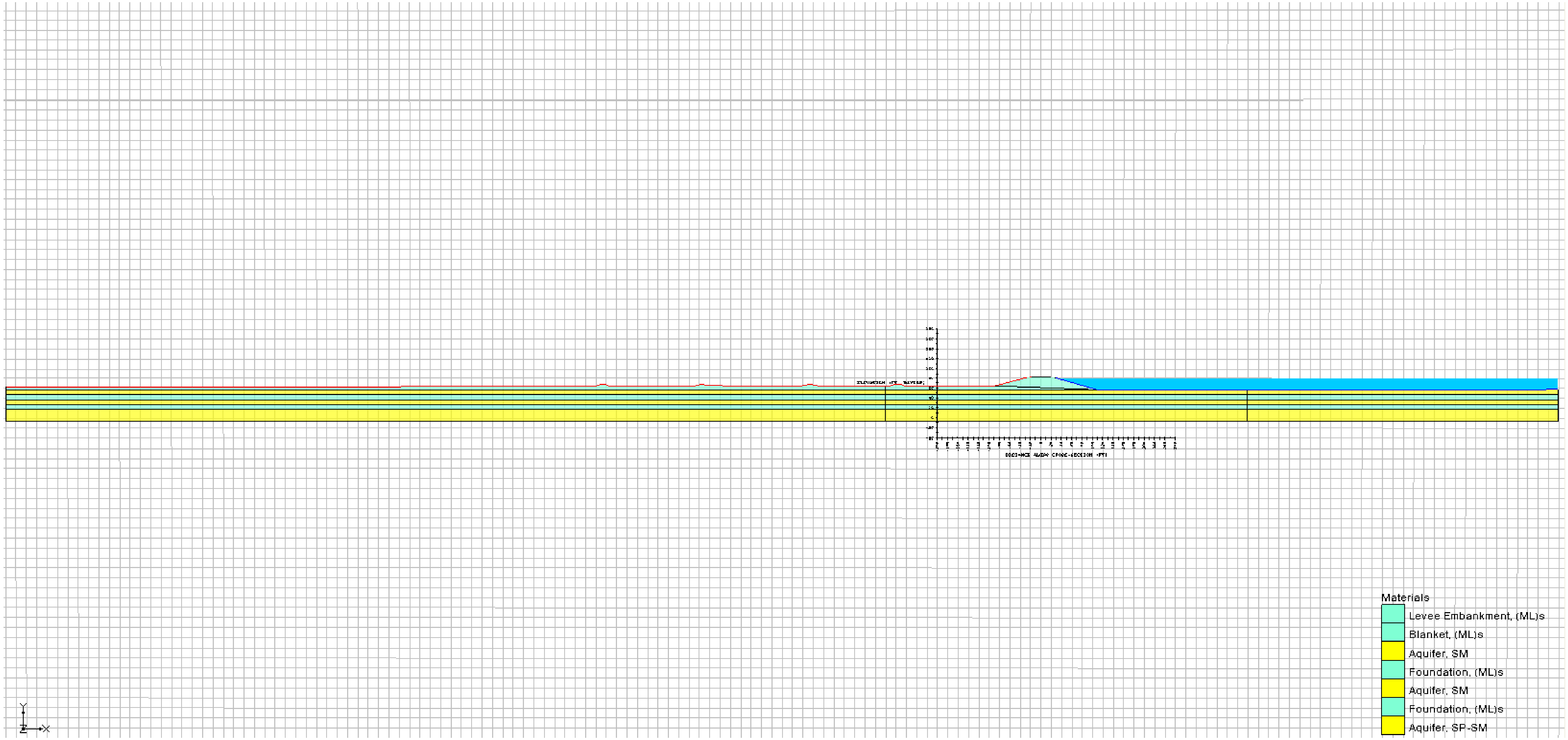
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER LM_3.99, STA 540+00
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	50	28
2	Blanket, (ML)s	0.570	0.140	4	115	50	28
3	Aquifer, SM	2.800	0.710	4	125	0	32
4	Foundation, (ML)s	0.57	0.14	4	120	50	28
5	Aquifer, SM	2.80	0.71	4	125	0	32
6	Foundation, (ML)s	0.57	0.14	4	120	50	28
7	Aquifer, SP-SM	14	1.4	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-07-05 & 2F-07-06
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 79.6 ft.

DEPARTMENT OF THE ARMY
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SACRAMENTO, CALIFORNIA

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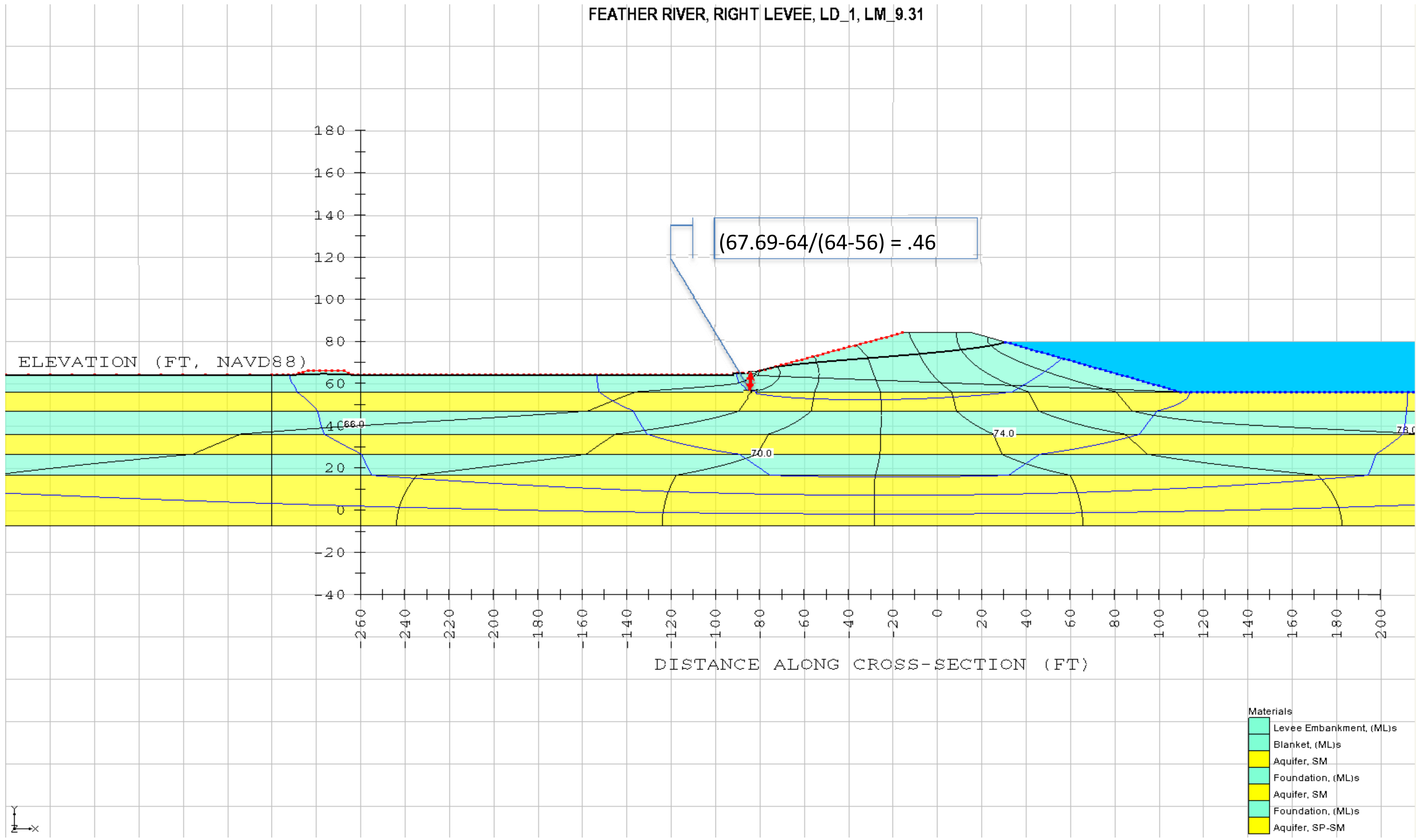
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_9.31, STA 1108+86
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	50	28
2	Blanket, (ML)s	0.570	0.140	4	115	50	28
3	Aquifer, SM	2.800	0.710	4	125	0	32
4	Foundation, (ML)s	0.57	0.14	4	120	50	28
5	Aquifer, SM	2.80	0.71	4	125	0	32
6	Foundation, (ML)s	0.57	0.14	4	120	50	28
7	Aquifer, SP-SM	14	1.4	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-07-05 & 2F-07-06
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 79.6 ft.

DEPARTMENT OF THE ARMY
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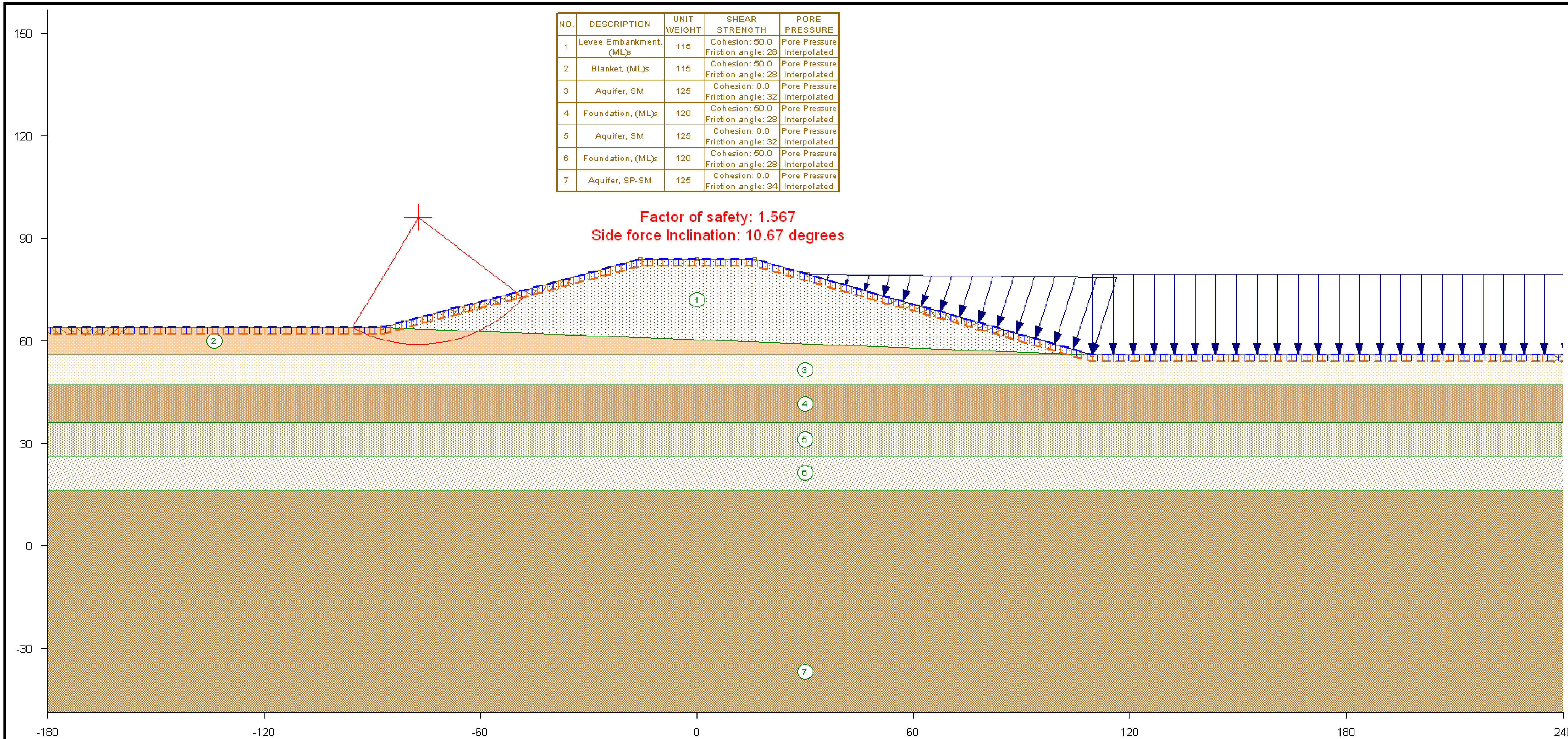
SACRAMENTO, CALIFORNIA

SUTTER BASIN — CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_9.31, STA 1108+86
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012	SCALE: AS SHOWN	SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, (ML)s	115	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
2	Blanket, (ML)s	115	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
3	Aquifer, SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
4	Foundation, (ML)s	120	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
5	Aquifer, SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
6	Foundation, (ML)s	120	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
7	Aquifer, SP-SM	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	50	28
2	Blanket, (ML)s	0.570	0.140	4	115	50	28
3	Aquifer, SM	2.800	0.710	4	125	0	32
4	Foundation, (ML)s	0.57	0.14	4	120	50	28
5	Aquifer, SM	2.80	0.71	4	125	0	32
6	Foundation, (ML)s	0.57	0.14	4	120	50	28
7	Aquifer, SP-SM	14	1.4	10	125	0	34

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-07-05 & 2F-07-06
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 79.6 ft.

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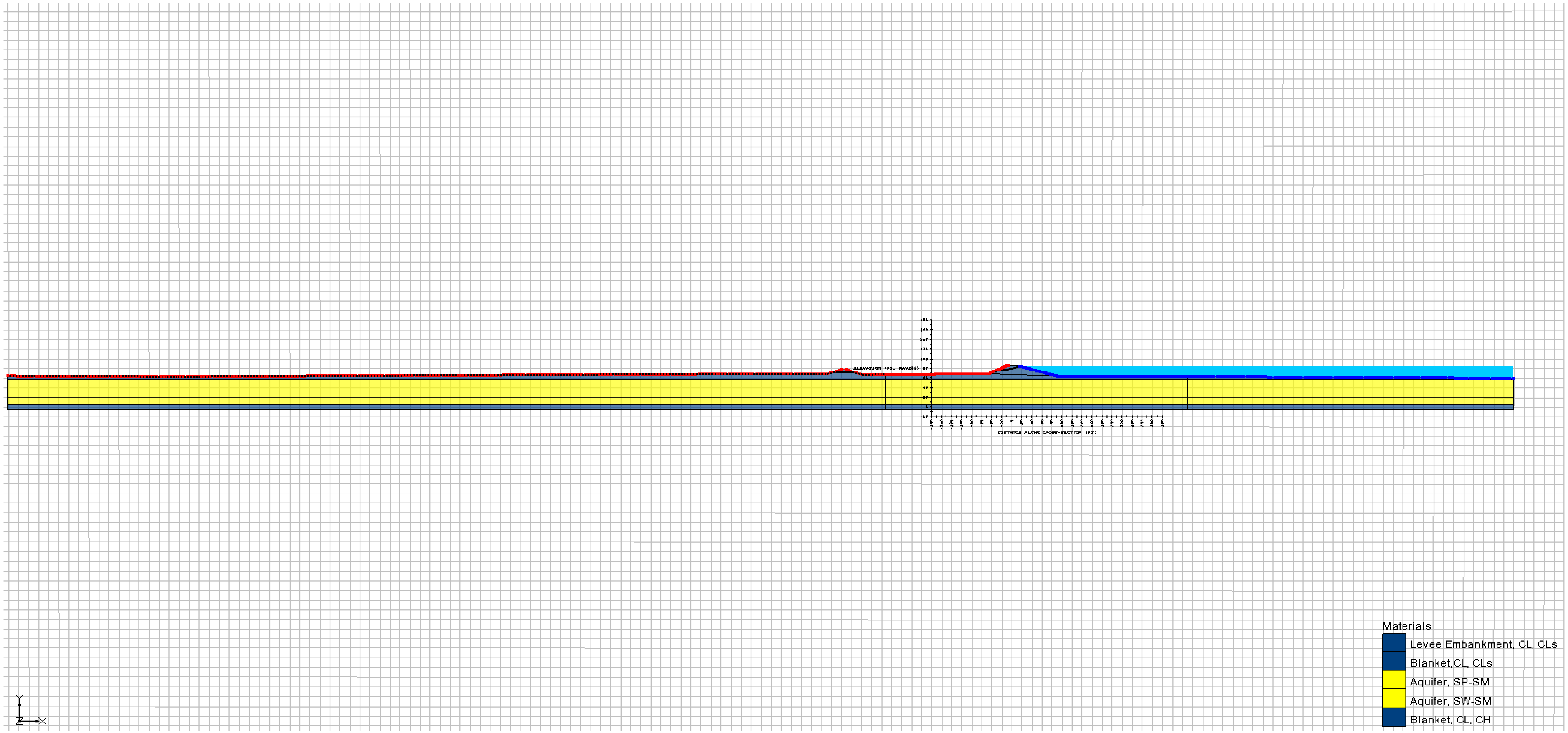
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER, LM_9.31, STA 1108+86

EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment CL CLs	0.028	0.007	4	115.00	150	28.00
2	Blanket CL CLs	0.028	0.007	4	115.00	150	28.00
3	Aquifer SP-SM	14.00	1.40	10	125.00	0	34.00
4	Aquifer SW-SM	14.00	1.40	10	125.00	0	34.00
5	Blanket CL CH	0.028	0.007	4	115.00	150	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0009_001S, WL0009_007A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 84 ft.

DEPARTMENT OF THE ARMY
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SACRAMENTO, CALIFORNIA

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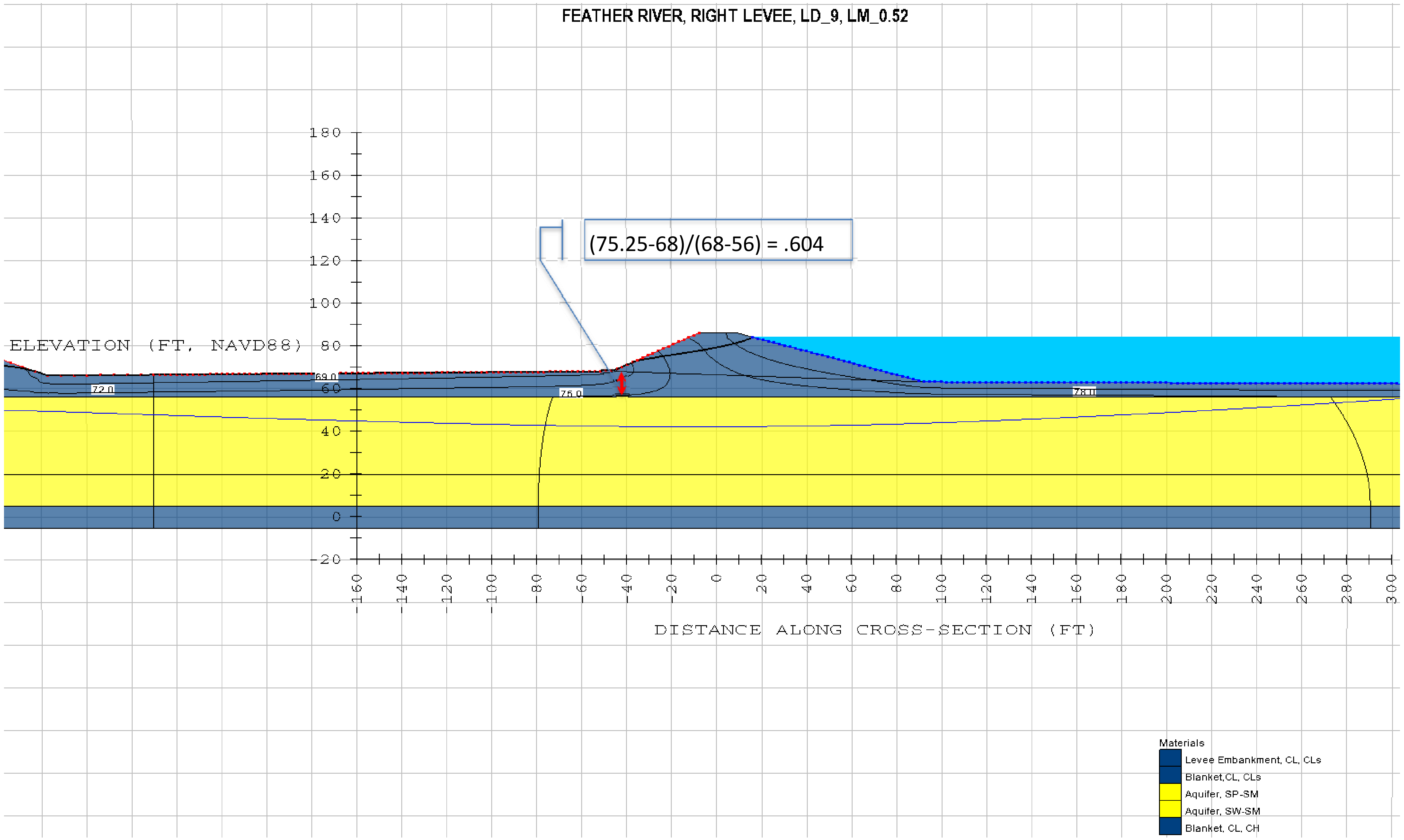
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_9, STATION 1138+68
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment CL CLs	0.028	0.007	4	115.00	150	28.00
2	Blanket CL CLs	0.028	0.007	4	115.00	150	28.00
3	Aquifer SP-SM	14.00	1.40	10	125.00	0	34.00
4	Aquifer SW-SM	14.00	1.40	10	125.00	0	34.00
5	Blanket CL CH	0.028	0.007	4	115.00	150	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0009_001S, WL0009_007A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 84 ft.

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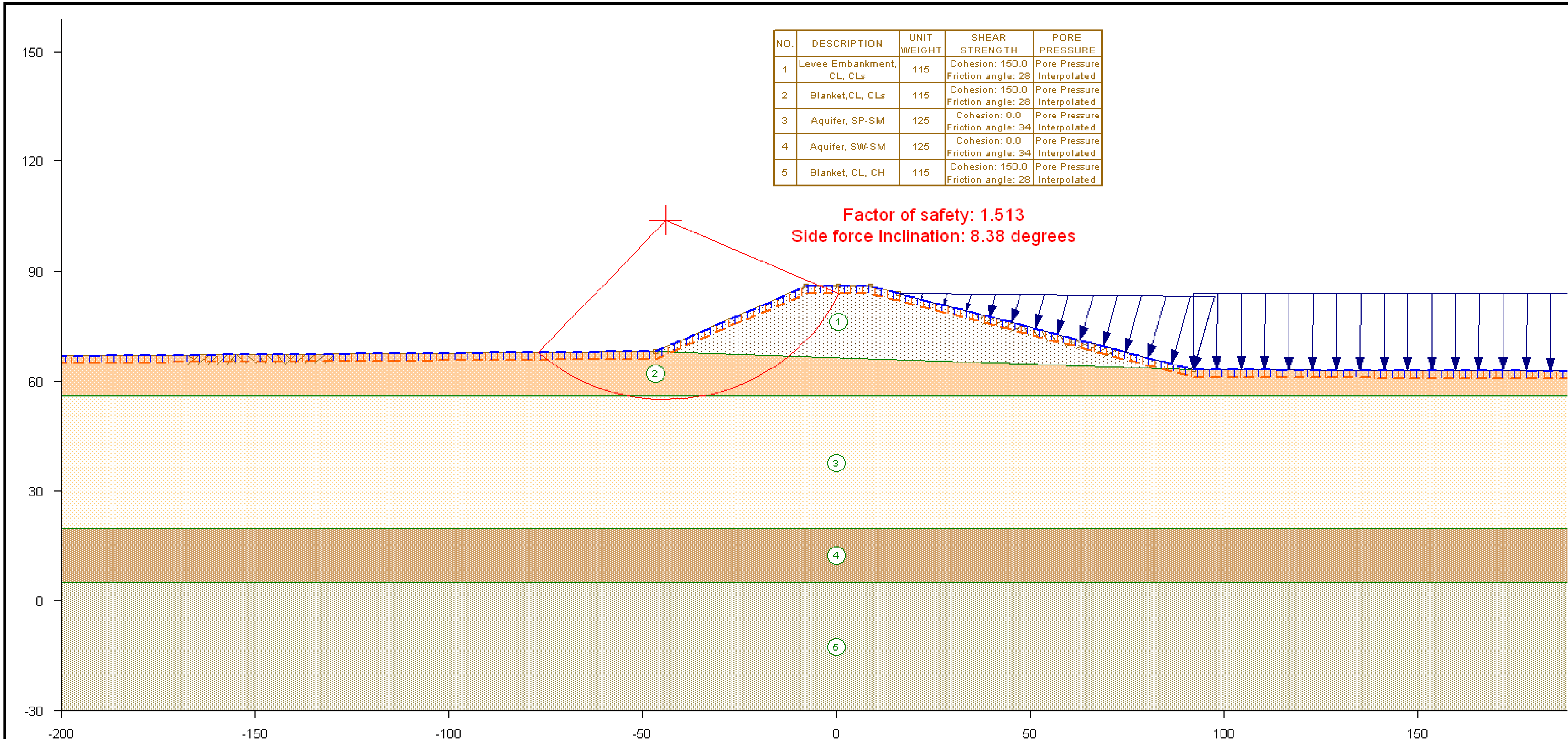
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_9, STATION 1138+68
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment CL CLs	0.028	0.007	4	115.00	150	28.00
2	Blanket CL CLs	0.028	0.007	4	115.00	150	28.00
3	Aquifer SP-SM	14.00	1.40	10	125.00	0	34.00
4	Aquifer SW-SM	14.00	1.40	10	125.00	0	34.00
5	Blanket CL CH	0.028	0.007	4	115.00	150	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WL0009_001S, WL0009_007A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 84 ft.

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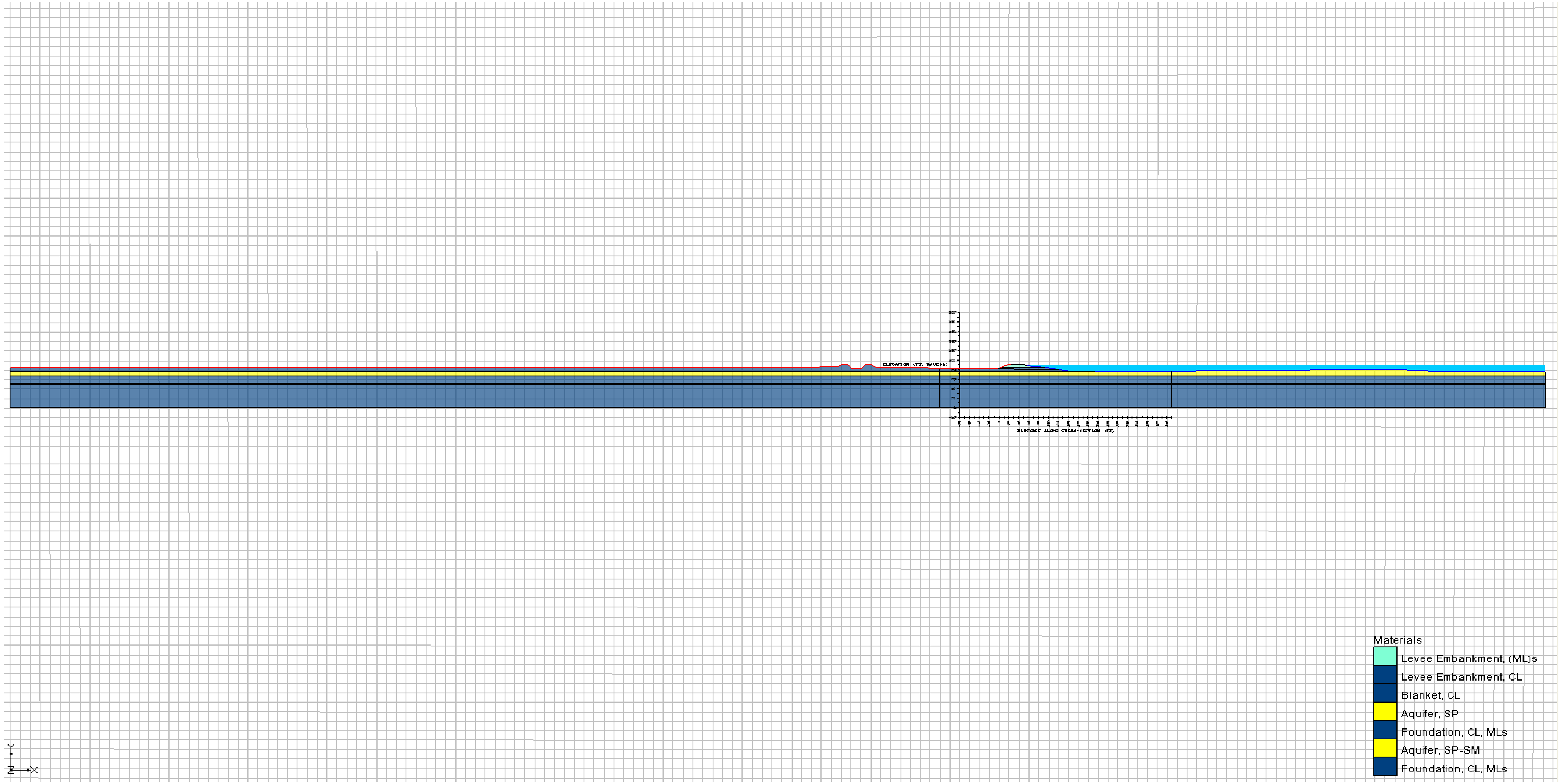
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER, LM_9, STATION 1138+68

EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL	0.028	0.007	4	115	150	28
3	Blanket, CL	0.028	0.007	4	115	150	28
4	Aquifer, SP	28.00	2.80	10	125	0	34
5	Foundation, CL, MLs	0.57	0.14	4	120	150	29
6	Aquifer, SP-SM	14.00	1.40	10	125	0	32
7	Foundation, CL, MLs	0.57	0.14	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_011B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 88.79 ft.

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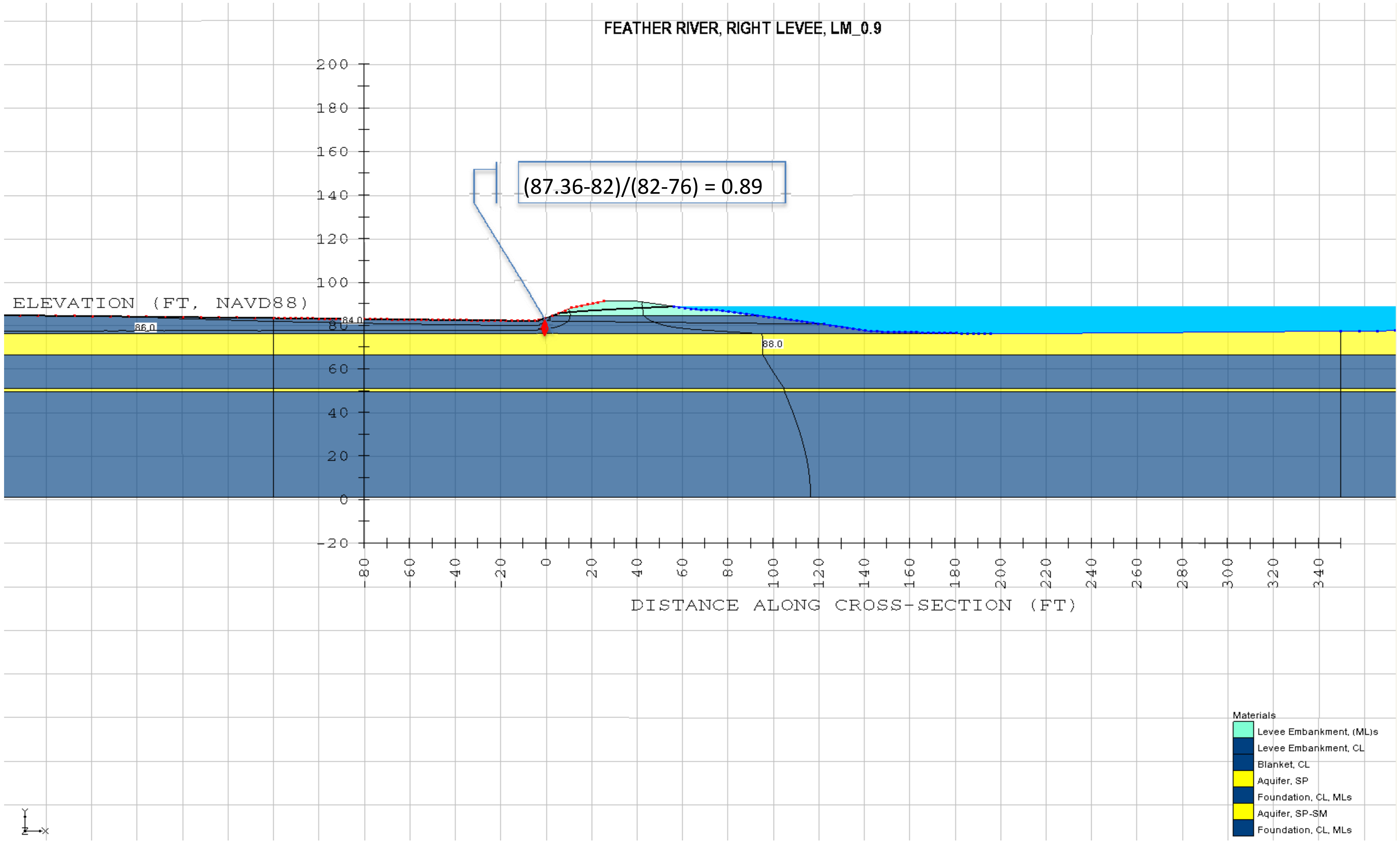
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_0.9, STA 1508+33
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL	0.028	0.007	4	115	150	28
3	Blanket, CL	0.028	0.007	4	115	150	28
4	Aquifer, SP	28.00	2.80	10	125	0	34
5	Foundation, CL, MLs	0.57	0.14	4	120	150	29
6	Aquifer, SP-SM	14.00	1.40	10	125	0	32
7	Foundation, CL, MLs	0.57	0.14	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_011B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 88.79 ft.

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GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_0.9, STA 1508+33
EXISTING CONDITION SEEPAGE DWSE

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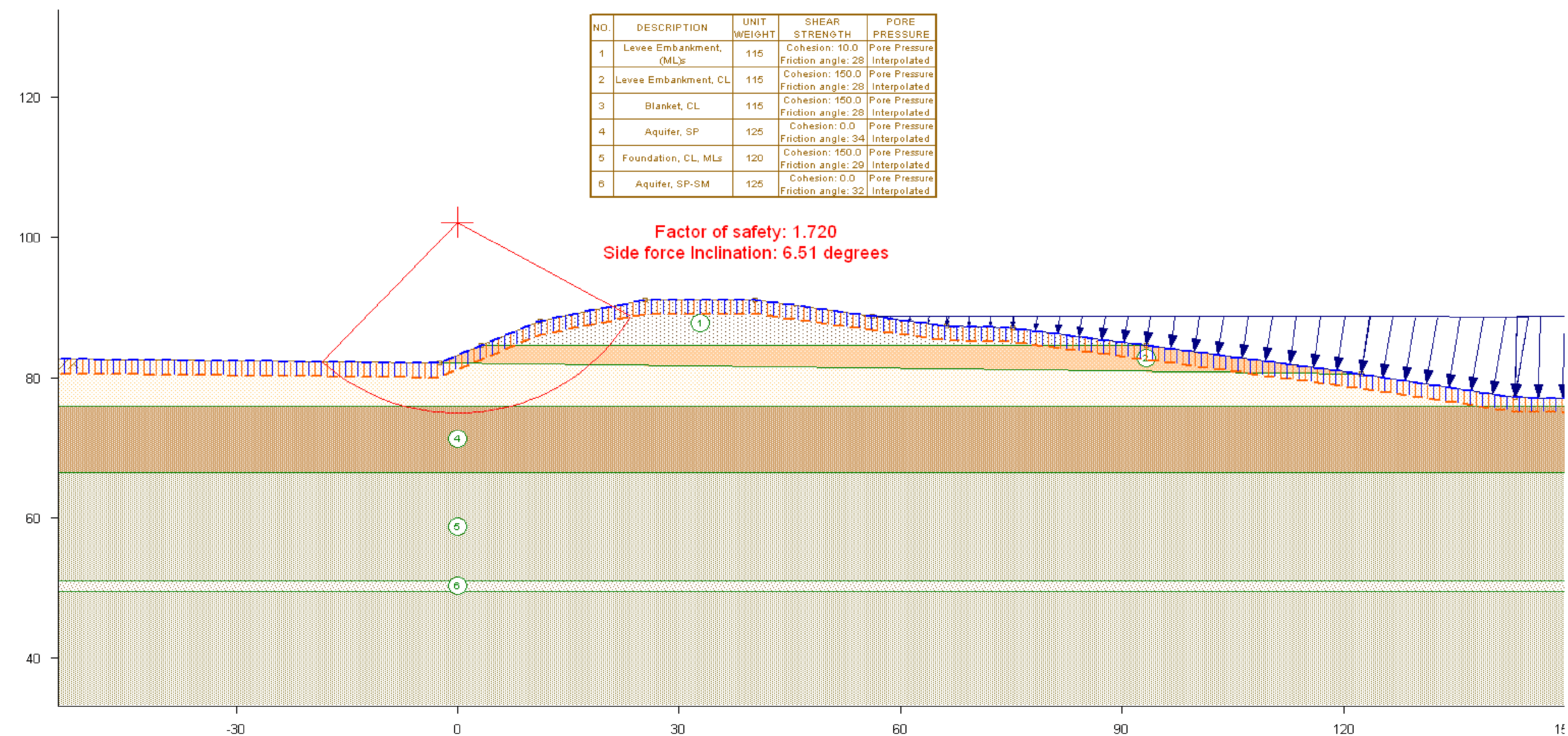

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, (ML)s	0.57	0.14	4	115	10	28
2	Levee Embankment, CL	0.028	0.007	4	115	150	28
3	Blanket, CL	0.028	0.007	4	115	150	28
4	Aquifer, SP	28.00	2.80	10	125	0	34
5	Foundation, CL, MLs	0.57	0.14	4	120	150	29
6	Aquifer, SP-SM	14.00	1.40	10	125	0	32
7	Foundation, CL, MLs	0.57	0.14	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_011B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 88.79 ft.

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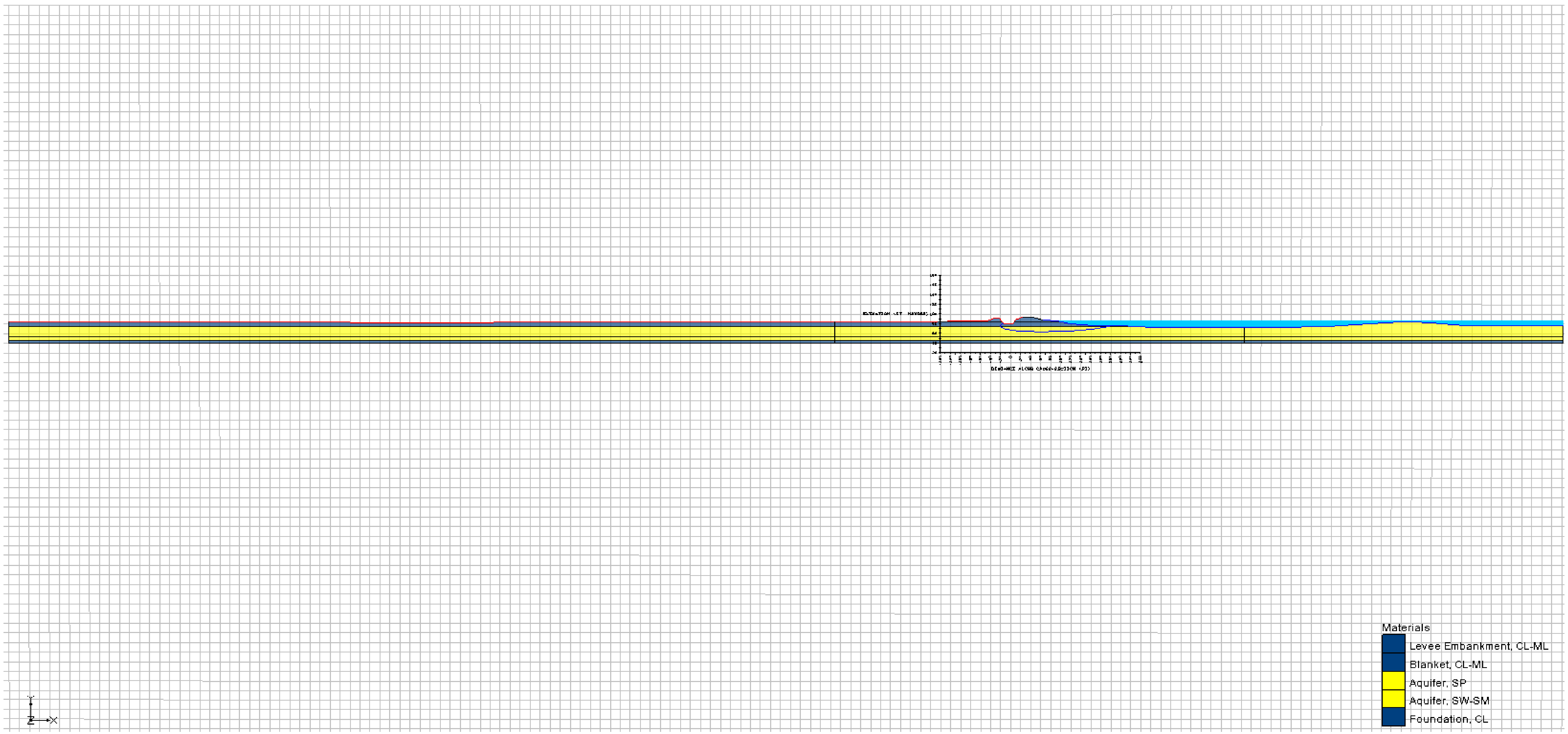
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_0.9, STA 1508+33
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, CL-ML	0.57	0.14	4	115.00	50.00	28.00
2	Blanket, CL-ML	0.570	0.140	4	115.00	50.00	28.00
3	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
4	Aquifer, SP	28.000	2.8	10	125.00	0.00	34.00
5	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
6	Foundation, CL	0.03	0.007	4	120.00	100.00	28.00

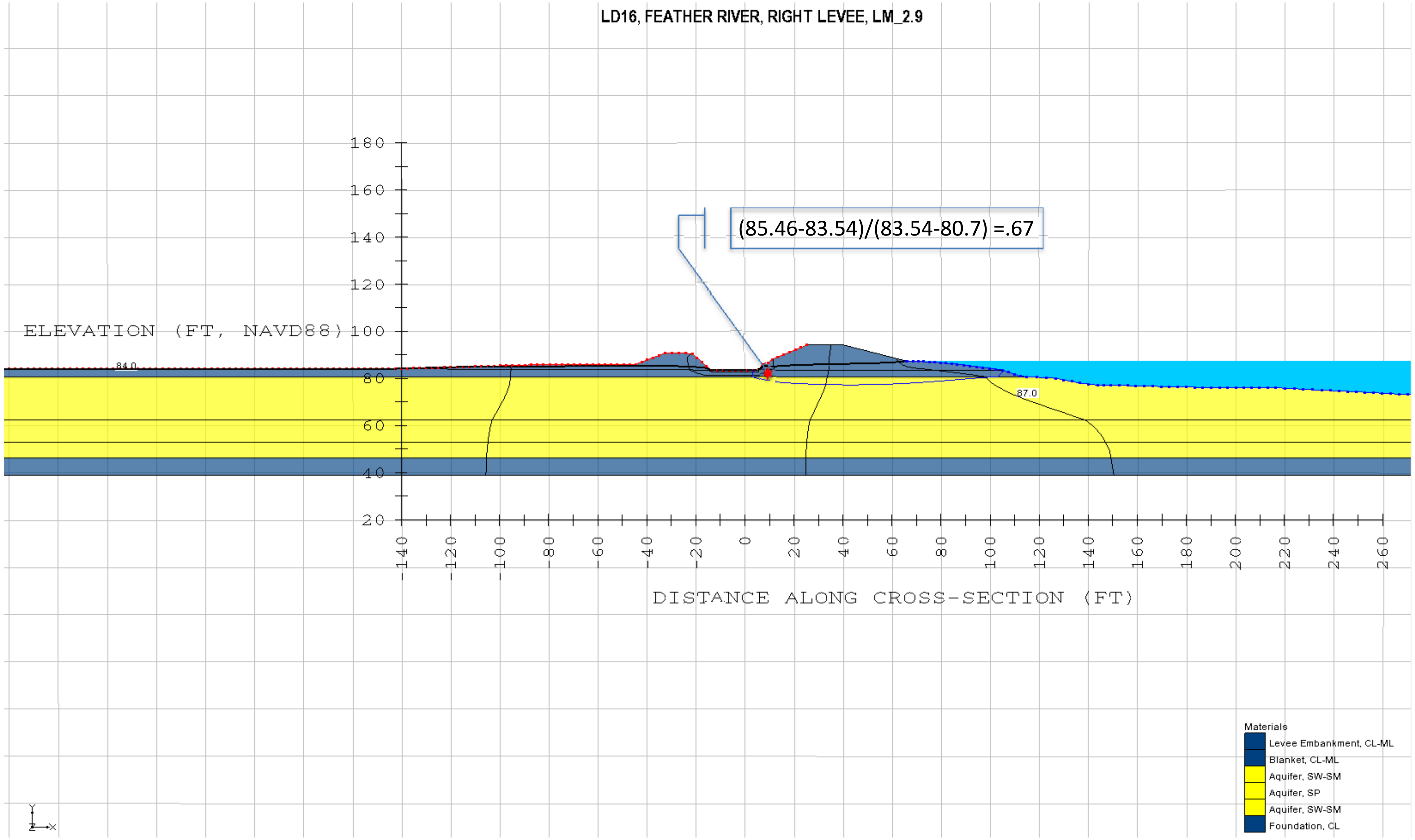
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_019S, WM0016-020B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 87.4 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER STATION 1615+62
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, CL-ML	0.57	0.14	4	115.00	50.00	28.00
2	Blanket, CL-ML	0.570	0.140	4	115.00	50.00	28.00
3	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
4	Aquifer, SP	28.000	2.8	10	125.00	0.00	34.00
5	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
6	Foundation, CL	0.03	0.007	4	120.00	100.00	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_019S, WM0016-020B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 87.4 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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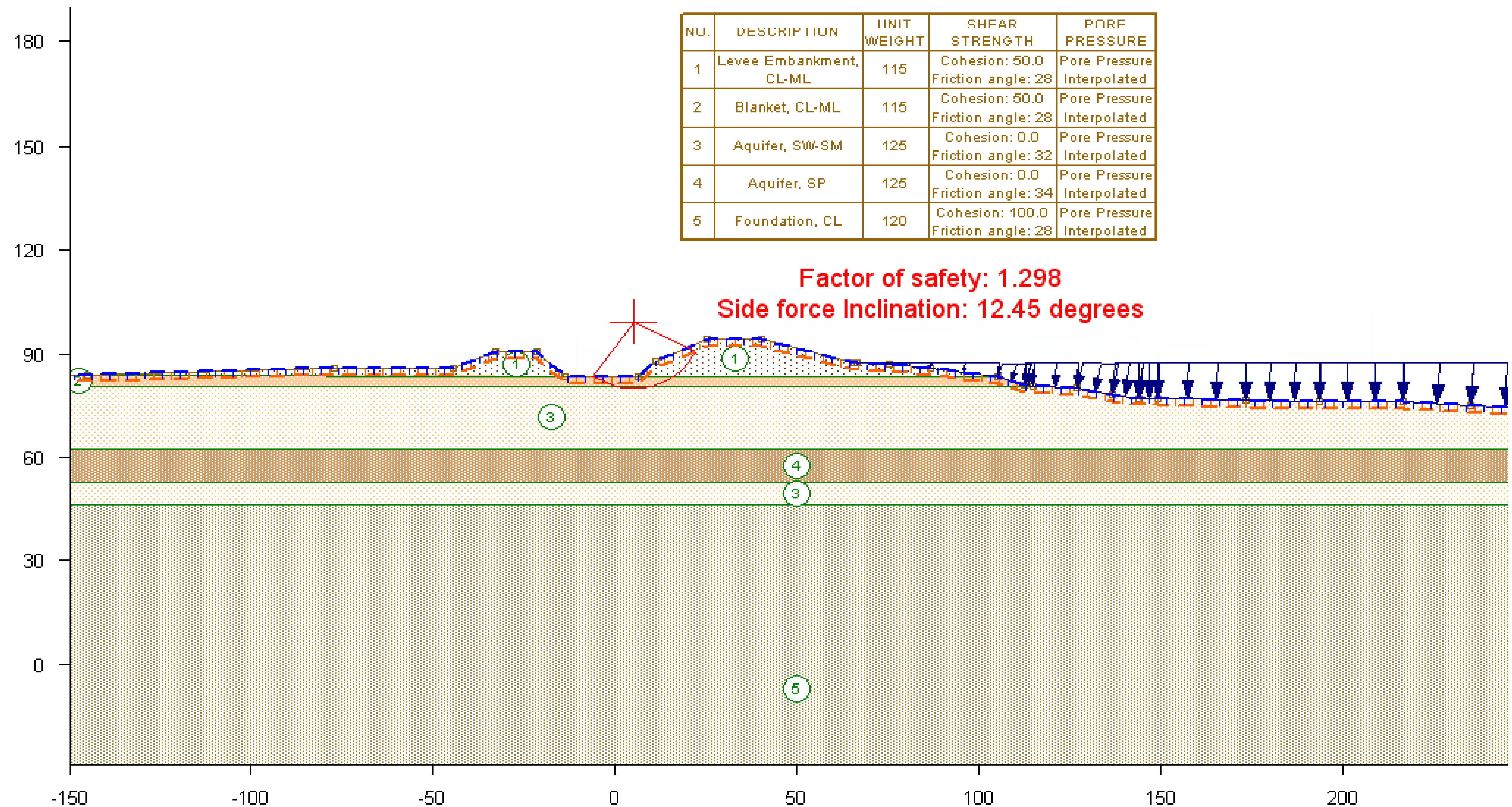
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER STATION 1615+62

EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012	SCALE: AS SHOWN	SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, CL-ML	115	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
2	Blanket, CL-ML	115	Cohesion: 50.0 Friction angle: 28	Pore Pressure Interpolated
3	Aquifer, SW-SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
4	Aquifer, SP	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
5	Foundation, CL	120	Cohesion: 100.0 Friction angle: 28	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, CL-ML	0.57	0.14	4	115.00	50.00	28.00
2	Blanket, CL-ML	0.570	0.140	4	115.00	50.00	28.00
3	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
4	Aquifer, SP	28.000	2.8	10	125.00	0.00	34.00
5	Aquifer, SW-SM	14.00	1.40	10	125.00	0.00	32.00
6	Foundation, CL	0.03	0.007	4	120.00	100.00	28.00

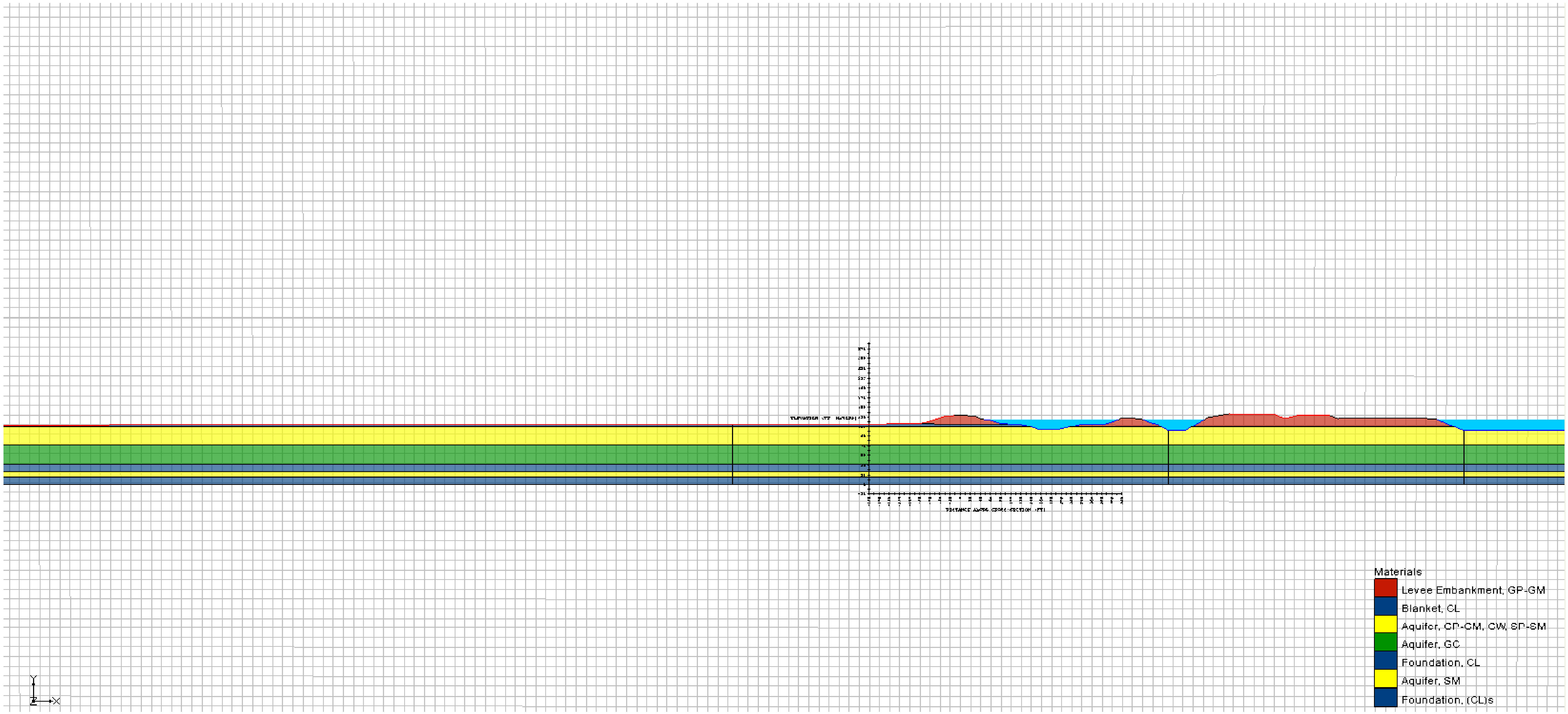
NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WM0016_019S, WM0016-020B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 87.4 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER STATION 1615+62
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, GP-GM	14.0	1.4	10	125.00	0.00	34.00
2	Blanket, CL	0.028	0.007	4	115.00	150.00	28.00
3	Aquifer, GP-GM, GW, SP-SM	14.0	1.4	10	125.00	0.00	34.00
4	Aquifer, GC	2.80	0.71	4	135.00	0.00	35.00
5	Foundation, CL	0.03	0.01	4	115.00	150.00	28.00
6	Aquifer, SM	1.40	0.35	4	125.00	0.00	32.00
7	Foundation, (CL)s	0.028	0.007	4	115.00	150.00	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WL0007_072S, WL0007_014S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 130 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

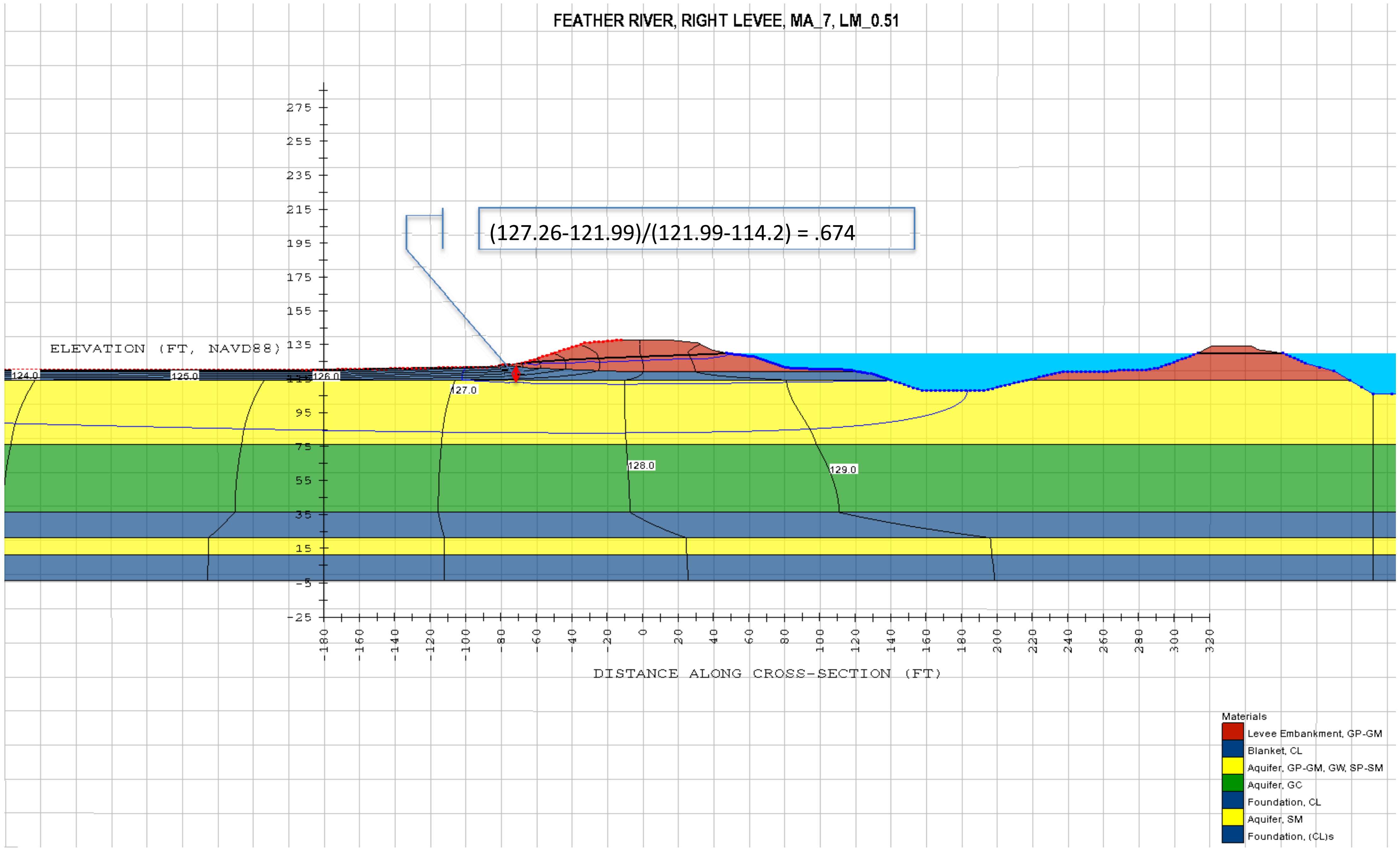
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER, LM_0.51, STATION 2332+91

COMPLETE SUBSURFACE MODEL

28-Jun-2012	SCALE: AS SHOWN	SHEET 1 of 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment, GP-GM	14.0	1.4	10	125.00	0.00	34.00
2	Blanket, CL	0.028	0.007	4	115.00	150.00	28.00
3	Aquifer, GP-GM, GW, SP-SM	14.0	1.4	10	125.00	0.00	34.00
4	Aquifer, GC	2.80	0.71	4	135.00	0.00	35.00
5	Foundation, CL	0.03	0.01	4	115.00	150.00	28.00
6	Aquifer, SM	1.40	0.35	4	125.00	0.00	32.00
7	Foundation, (CL)s	0.028	0.007	4	115.00	150.00	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WL0007_072S, WL0007_014S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 130 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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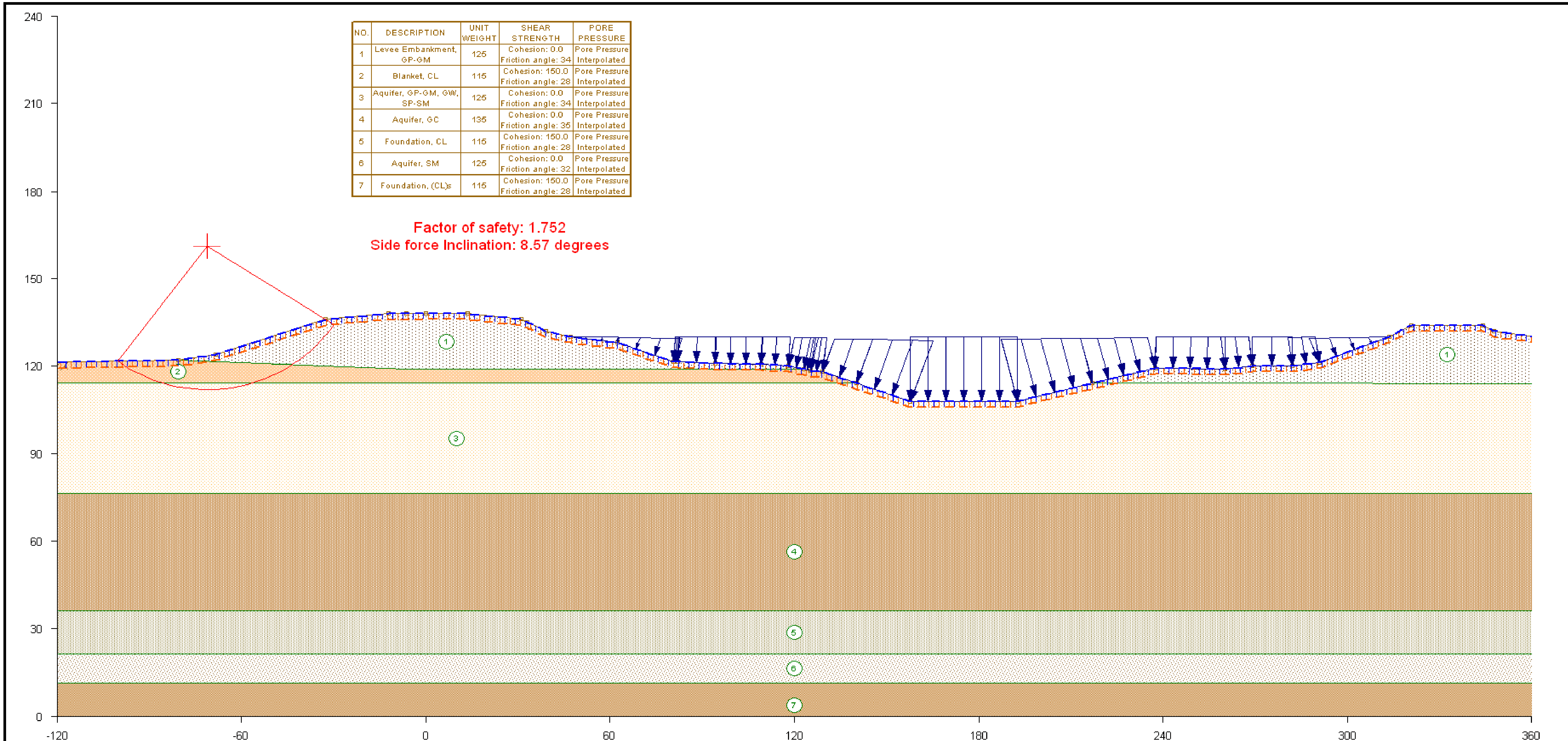
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

FEATHER RIVER, LM_0.51, STATION 2332+91

EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Embankment, GP-GM	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
2	Blanket, CL	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated
3	Aquifer, GP-GM, GW, SP-SM	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
4	Aquifer, GC	135	Cohesion: 0.0 Friction angle: 35	Pore Pressure Interpolated
5	Foundation, CL	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated
6	Aquifer, SM	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
7	Foundation, (CL)s	115	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment, GP-GM	14.0	1.4	10	125.00	0.00	34.00
2	Blanket, CL	0.028	0.007	4	115.00	150.00	28.00
3	Aquifer, GP-GM, GW, SP-SM	14.0	1.4	10	125.00	0.00	34.00
4	Aquifer, GC	2.80	0.71	4	135.00	0.00	35.00
5	Foundation, CL	0.03	0.01	4	115.00	150.00	28.00
6	Aquifer, SM	1.40	0.35	4	125.00	0.00	32.00
7	Foundation, (CL)s	0.028	0.007	4	115.00	150.00	28.00

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on borings WL0007_072S, WL0007_014S
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 130 ft.

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SACRAMENTO DISTRICT
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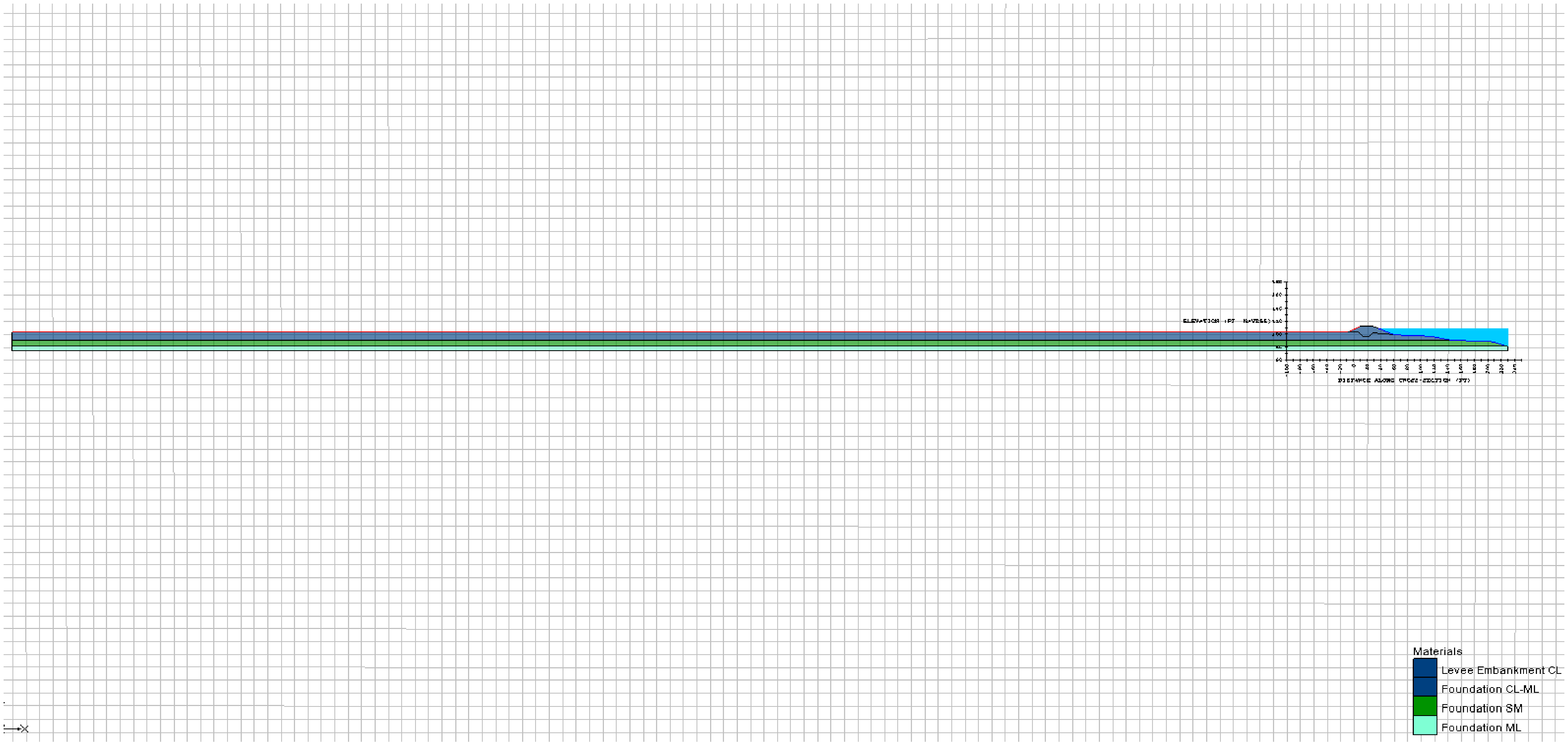
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
FEATHER RIVER, LM_0.51, STATION 2332+91
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment CL	0.028	0.0071	4	115	150	28
2	Foundation CL-ML	0.028	0.0071	4	115	150	28
3	Foundation SM	2.8	0.71	4	120	0	32
4	Foundation ML	0.14	0.035	4	120	0	31

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-08-34
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 109 ft.

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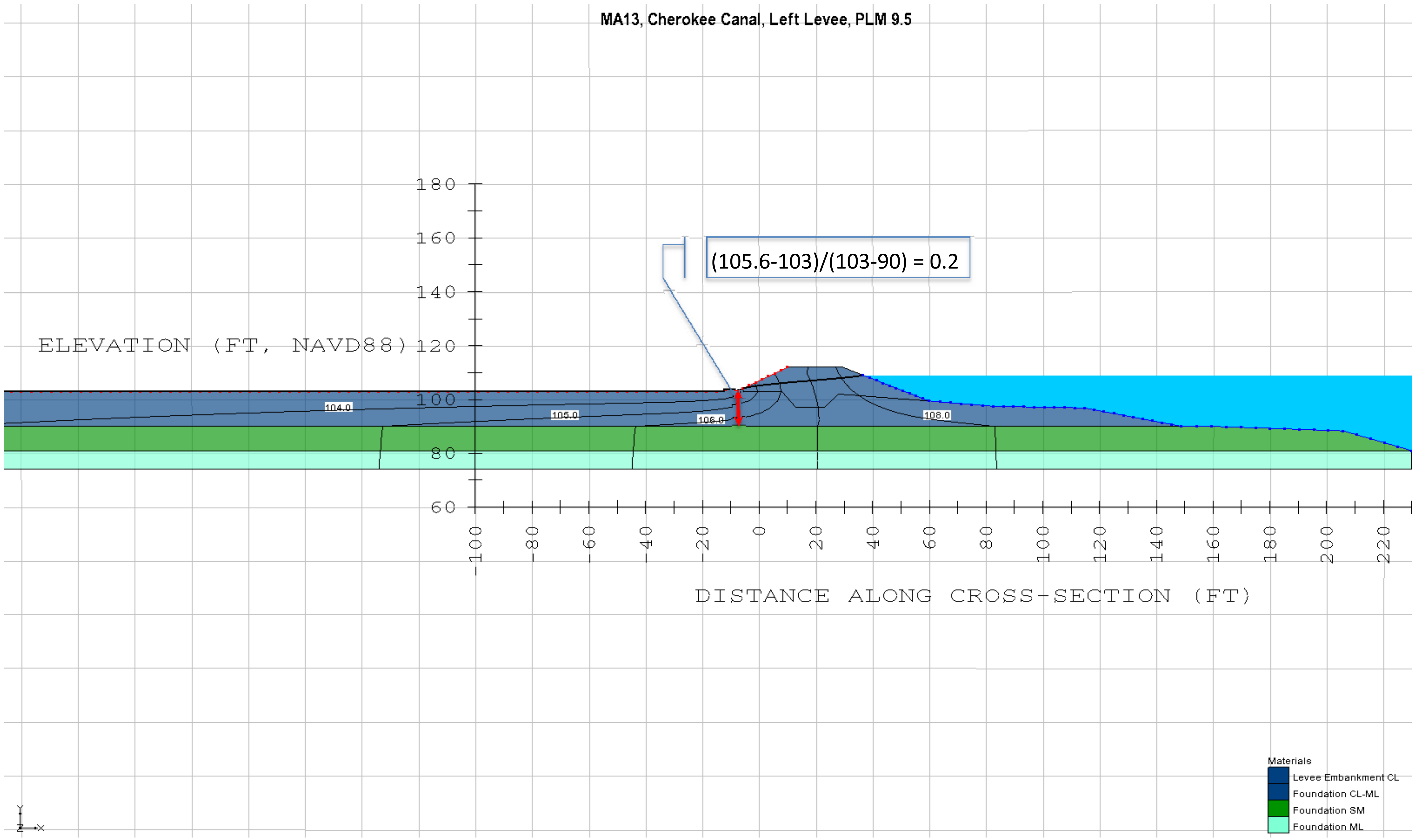
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
CHEROKEE CANAL-LEFT LEVEE, PLM 9.5
COMPLETE SUBSURFACE MODEL

28-Jun-2012 | SCALE: AS SHOWN | SHEET 1 of 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Embankment CL	0.028	0.0071	4	115	150	28
2	Foundation CL-ML	0.028	0.0071	4	115	150	28
3	Foundation SM	2.8	0.71	4	120	0	32
4	Foundation ML	0.14	0.035	4	120	0	31

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-08-34
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 109 ft.

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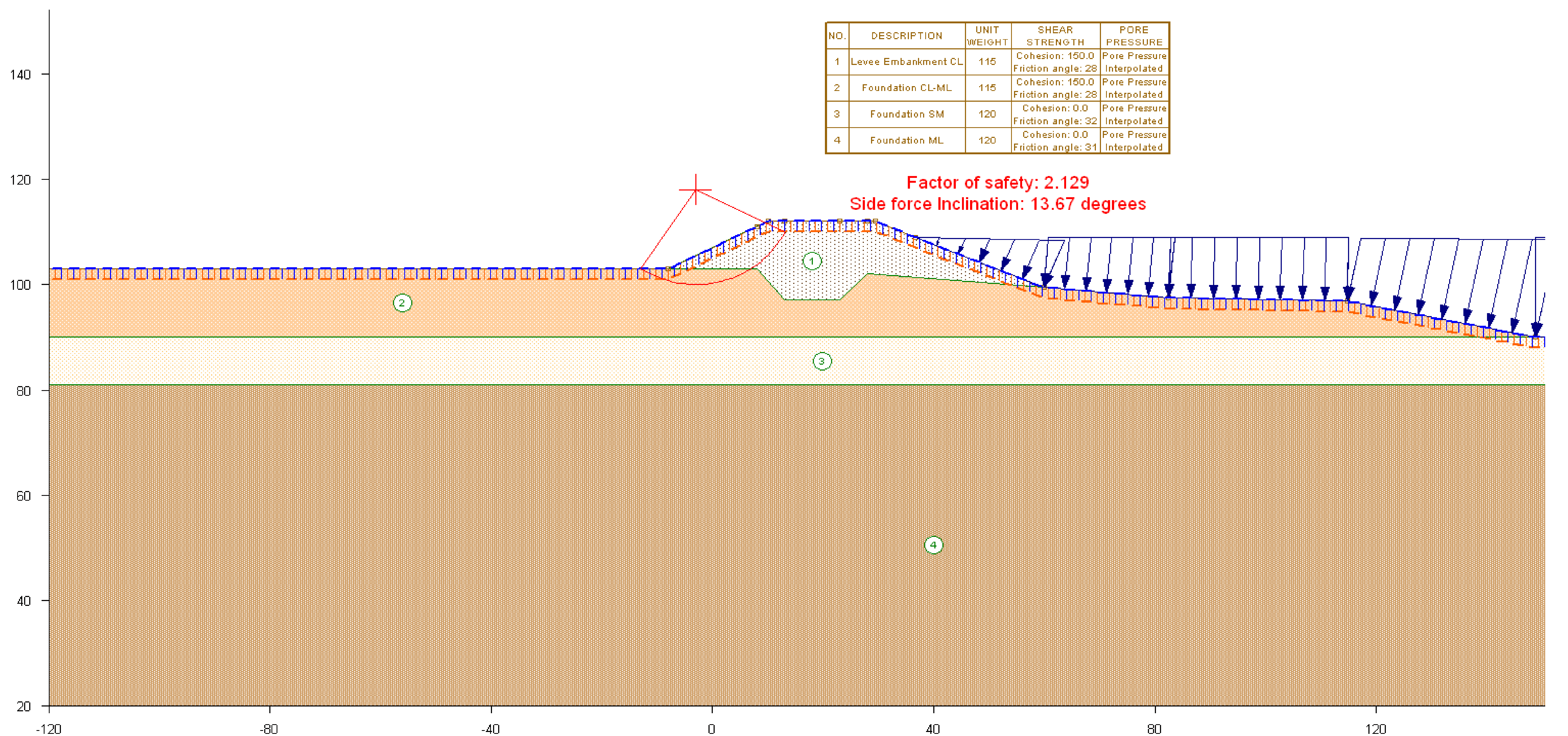
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
CHEROKEE CANAL-LEFT LEVEE, PLM 9.5
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	Levee Embankment CL	0.028	0.0071	4	115	150	28
2	Foundation CL-ML	0.028	0.0071	4	115	150	28
3	Foundation SM	2.8	0.71	4	120	0	32
4	Foundation ML	0.14	0.035	4	120	0	31

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring 2F-08-34
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 109 ft.

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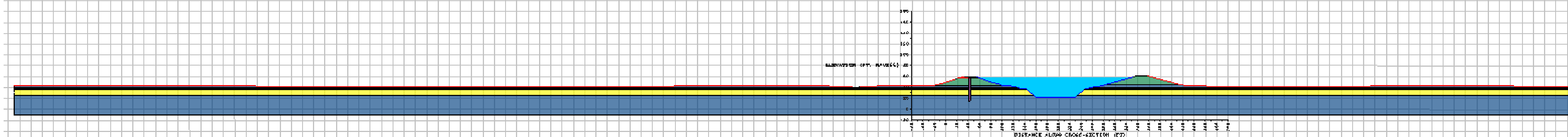
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
CHEROKEE CANAL-LEFT LEVEE, PLM 9.5
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	See Attached Sheet A						
2							
3							
4							
5							
6							
7							
8							
9							
10							

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_029B, WSEWWC_001B, WSEWWC_002A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation =58.5 ft.

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SACRAMENTO, CALIFORNIA

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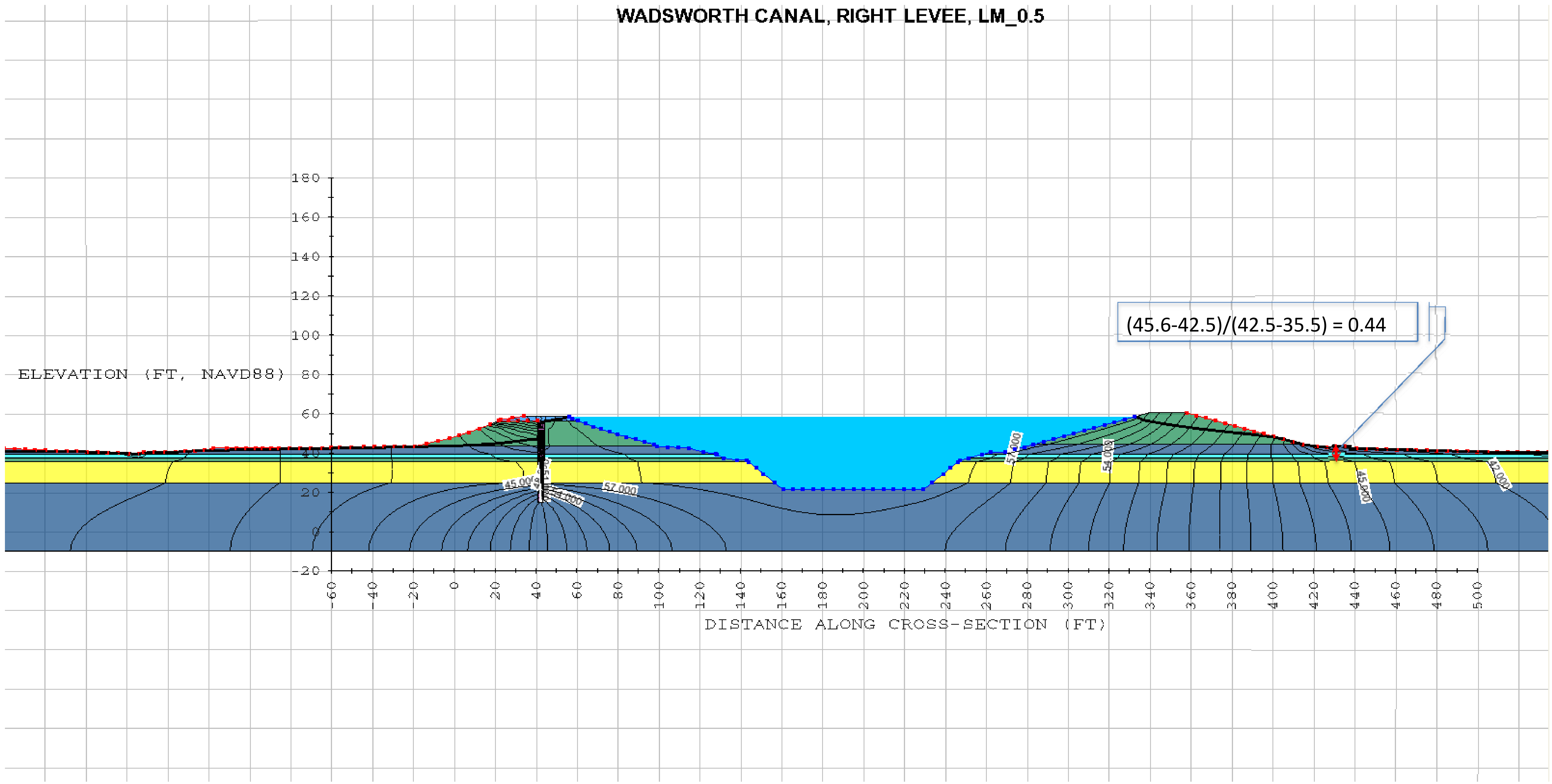
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, RIGHT LEVEE, LM_0.5
COMPLETE SUBSURFACE MODEL

28-Jun-2012	SCALE: AS SHOWN	SHEET 1 of 3



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	See Attached Sheet A						
2							
3							
4							
5							
6							
7							
8							
9							
10							

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_029B, WSEWWC_001B, WSEWWC_002A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation =58.5 ft.

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SACRAMENTO DISTRICT
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SACRAMENTO, CALIFORNIA

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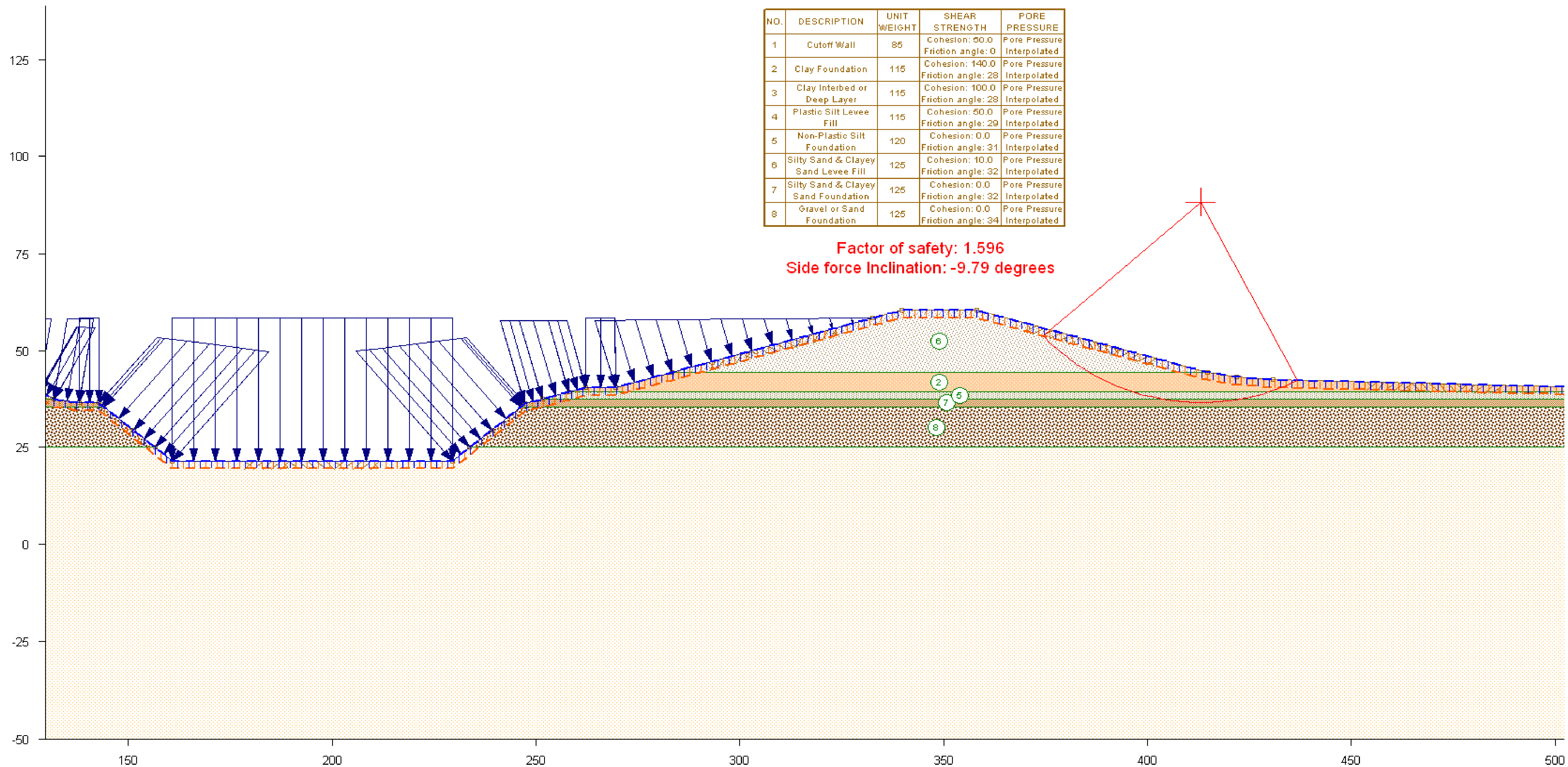
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, RIGHT LEVEE, LM_0.5
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012	SCALE: AS SHOWN	SHEET 2 OF 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Cutoff Wall	85	Cohesion: 50.0 Friction angle: 0	Pore Pressure Interpolated
2	Clay Foundation	115	Cohesion: 140.0 Friction angle: 28	Pore Pressure Interpolated
3	Clay Interbed or Deep Layer	115	Cohesion: 100.0 Friction angle: 28	Pore Pressure Interpolated
4	Plastic Silt Levee Fill	115	Cohesion: 50.0 Friction angle: 29	Pore Pressure Interpolated
5	Non-Plastic Silt Foundation	120	Cohesion: 0.0 Friction angle: 31	Pore Pressure Interpolated
6	Silty Sand & Clayey Sand Levee Fill	125	Cohesion: 10.0 Friction angle: 32	Pore Pressure Interpolated
7	Silty Sand & Clayey Sand Foundation	125	Cohesion: 0.0 Friction angle: 32	Pore Pressure Interpolated
8	Gravel or Sand Foundation	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	ϕ'
1	See Attached Sheet A						
2							
3							
4							
5							
6							
7							
8							
9							
10							

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_029B, WSEWWC_001B, WSEWWC_002A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation =58.5 ft.

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SACRAMENTO, CALIFORNIA

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GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
WADSWORTH CANAL, RIGHT LEVEE, LM_0.5
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3

lope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Cutoff Wall	0.00284	0.00071	4.0	85	50	0
2	Clay Levee Fill	0.0284	0.0071	4.0	115	150	27
3	Clay Foundation	0.0284	0.0071	4.0	115	150	27
4	Clay Interbed or Deep Layer	0.0284	0.0071	4.0	115	150	27
5	Plastic Silt Levee Fill	0.14	0.035	4.0	115	10	28
6	Plastic Silt Foundation	0.14	0.035	4.0	115	10	28
7	Plastic Silt Interbed or Deep Layer	0.14	0.035	4.0	115	10	28
8	Non-Plastic Silt Levee Fill	0.57	0.1425	4.0	115	0	28
9	Non-Plastic Silt Foundation	0.57	0.1425	4.0	115	0	28
10	Non-Plastic Silt Interbed or Deep Layer	0.57	0.1425	4.0	115	0	28
11	Silty Sand & Clayey Sand Levee Fill	0.14	0.035	4.0	125	0	32
12	Silty Sand & Clayey Sand Foundation	0.28	0.07	4.0	125	0	32
13	Silty Sand & Clayey Sand Interbed or Deep Layer	0.28	0.07	4.0	125	0	32
14	Silty or Clayey Gravel	1.13	0.113	10.0	120	0	32
15	Gravel or Sand Foundation	2.8	0.70	4.0	135	0	34
16	Gravel or Sand Interbeds or Deep Layer	17	1.7	10.0	135	0	34
17	Sand Levee Fill (drainage layer)	1	1	1.0	125	0	32

Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSEWWC_029B, WSEWWC_001B, WSEWWC_002A
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation =58.5 ft.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
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SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

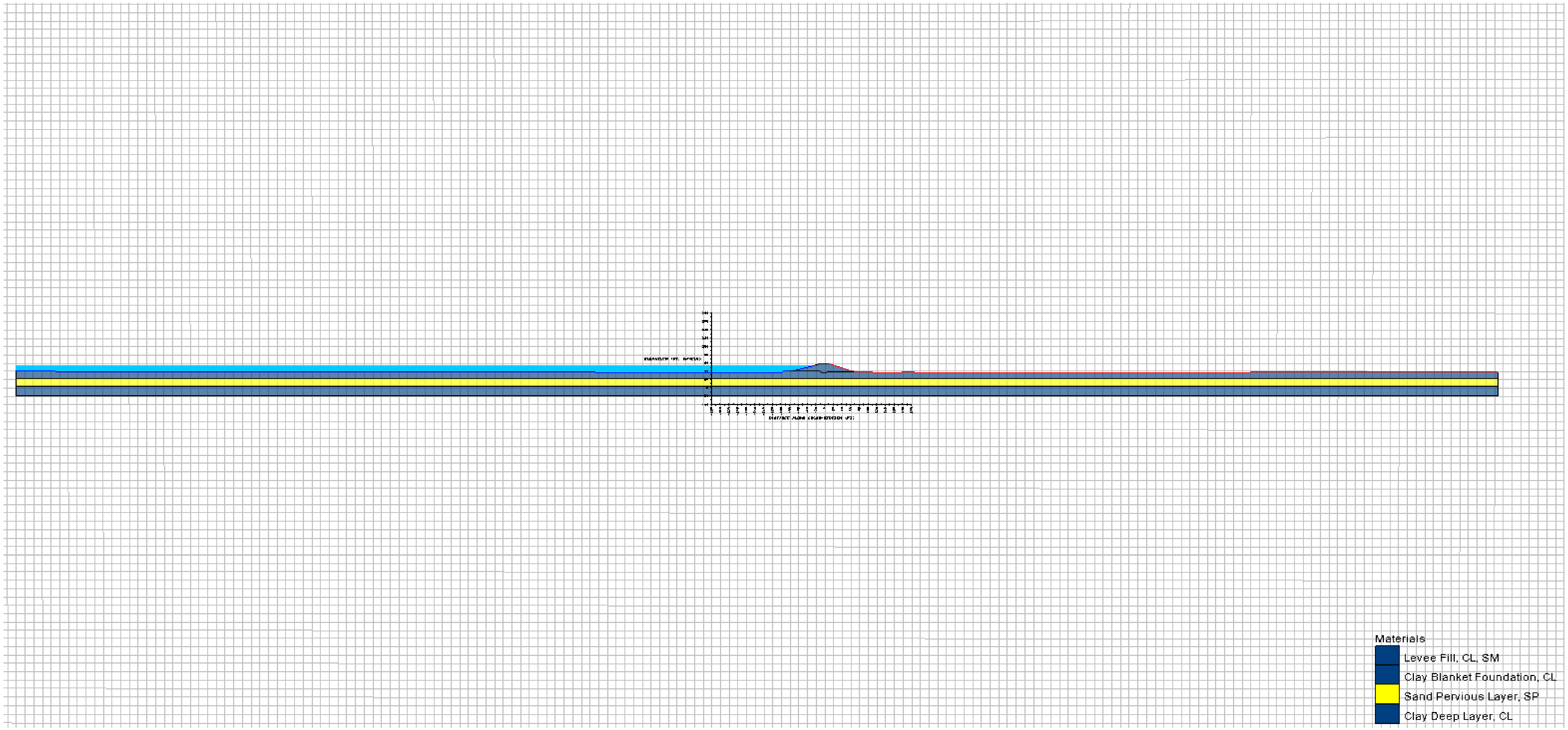
GEOTECHNICAL ENGINEERING APPENDIX

SEEPAGE AND STABILITY ANALYSIS

WADSWORTH CANAL, RIGHT LEVEE, LM_0.5

EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET A OF A



Seepage and Slope Stability Parameters							
Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Fill Sand-Clay Clam Fill	2.80	0.71	4	115	50	28
2	Clay Blanket Foundation	0.028	0.007	4	120	100	28
3	Sand Pervious Layer	28.000	2.800	10	125	0	34
4	Clay Deep Layer	0.03	0.01	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_019B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 55.3 ft.

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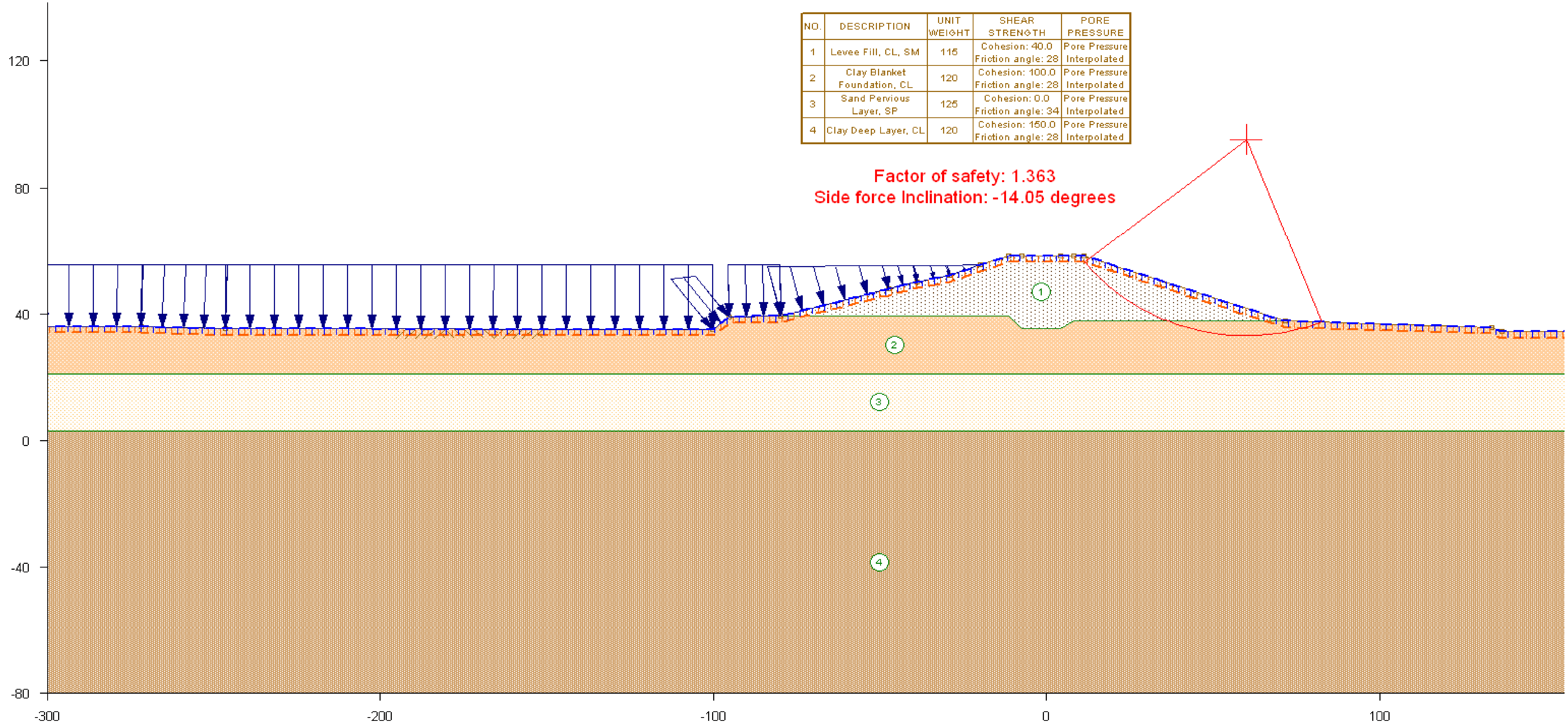
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER BASIN CALIFORNIA

GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS LM 4.0
COMPLETE SUBSURFACE MODEL

28-Jun-2012	SCALE: AS SHOWN	SHEET 1 of 3



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill, CL, SM	115	Cohesion: 40.0 Friction angle: 28	Pore Pressure Interpolated
2	Clay Blanket Foundation, CL	120	Cohesion: 100.0 Friction angle: 28	Pore Pressure Interpolated
3	Sand Pervious Layer, SP	125	Cohesion: 0.0 Friction angle: 34	Pore Pressure Interpolated
4	Clay Deep Layer, CL	120	Cohesion: 150.0 Friction angle: 28	Pore Pressure Interpolated

Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Fill Sand-Clay Clam Fill	2.80	0.71	4	115	50	28
2	Clay Blanket Foundation	0.028	0.007	4	120	100	28
3	Sand Pervious Layer	28.000	2.800	10	125	0	34
4	Clay Deep Layer	0.03	0.01	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_019B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation = 55.3 ft.

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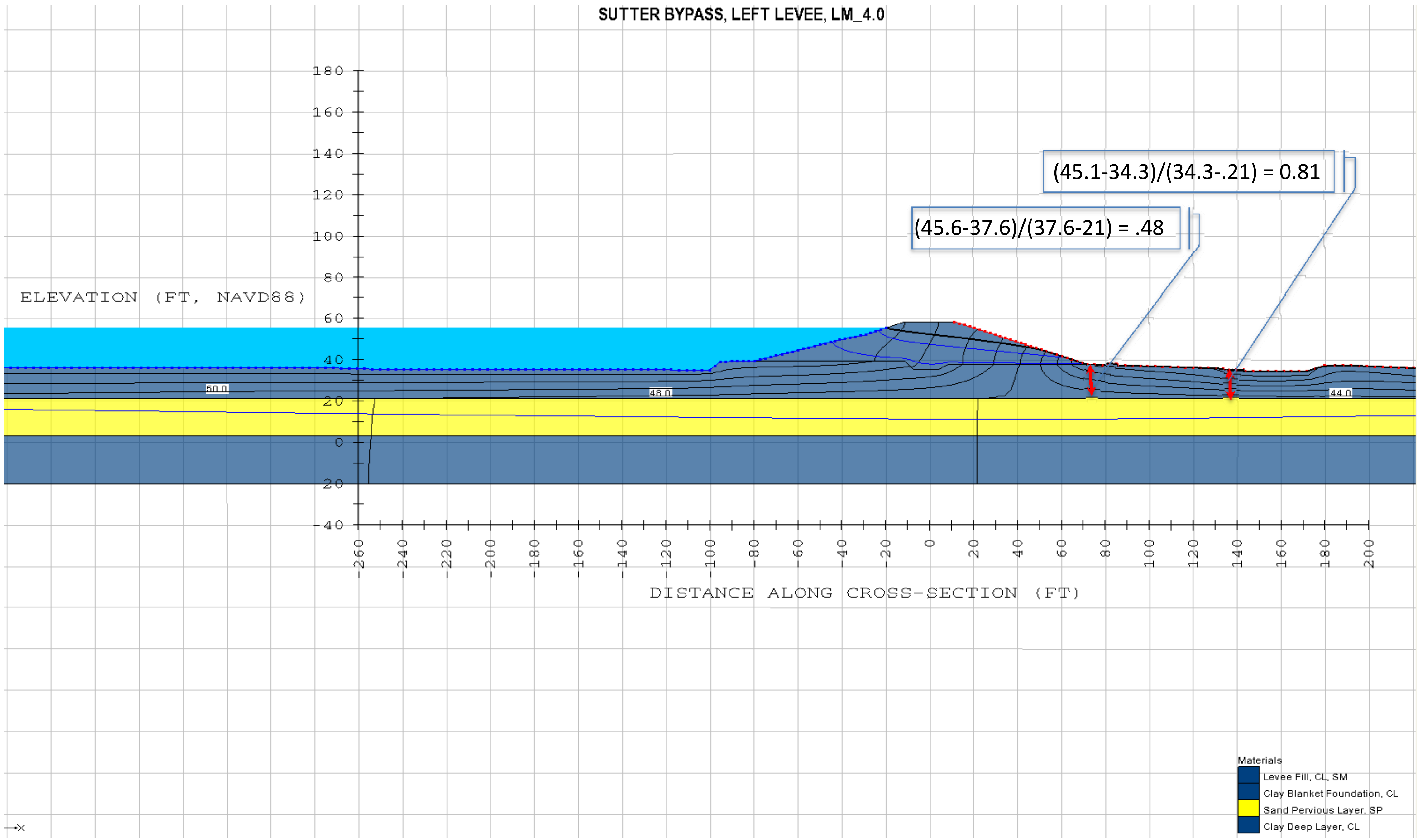
GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS LM 4.0
EXISTING CONDITION STABILITY DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 3 OF 3



Seepage and Slope Stability Parameters

Model Layer No.	Model Layer Name	K_h (ft/day)	K_v (ft/day)	K_h/K_v	γ_{sat} (pcf)	c' (psf)	φ'
1	Levee Fill Sand-Clay Clam Fill	2.80	0.71	4	115	50	28
2	Clay Blanket Foundation	0.028	0.007	4	120	100	28
3	Sand Pervious Layer	28.000	2.800	10	125	0	34
4	Clay Deep Layer	0.03	0.01	4	120	150	28

NOTES:
1. Complete model shown.
2. Waterside boundary set at mid-point of high flow channel.
3. Lower boundary is no flow
4. Side boundary on landside and waterside is no flow
5. Model based on boring WSESBP_019B
6. Elevation shown referenced to NAVD88
7. Water Surface Elevation =55.3 ft.

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GEOTECHNICAL ENGINEERING APPENDIX
SEEPAGE AND STABILITY ANALYSIS
SUTTER BYPASS LM 4.0
EXISTING CONDITION SEEPAGE DWSE

28-Jun-2012 | SCALE: AS SHOWN | SHEET 2 OF 3

ENCLOSURE H

TECHNICAL MEMORANDUM FOR ALTERNATIVES SELECTION

GEOTECHNICAL ENGINEERING

TECHNICAL MEMORANDUM

FOR

ALTERNATIVES SELECTION

SUTTER BASIN, CALIFORNIA

Prepared By:

U.S. Army Corps of Engineers

Sacramento District

Geotechnical Branch

Levee Safety Section

January 2013

1.0 INTRODUCTION

This technical memorandum presents the geotechnical recommendations for the Sutter Feasibility Study. These feasibility level recommendations are intended to facilitate the selection of a final alternative. As a result, the recommendations of this report are general and not site-specific, though each reach or segment recommendation is adjusted for large scale understanding of local conditions. Additionally, construction considerations were provided to the Civil Design section including typical design sections, haul and staging, construction sequencing, and borrow. This technical memorandum was prepared not using the analysis typically to develop seepage and stability mitigation measures (i.e. finite element seepage analysis or limit-equilibrium slope stability calculations). Instead engineering judgment and experience on local projects was utilized.

2.0 EVALUATION

This section presents the evaluation of the site conditions based on previous reports and studies, which provides the basis upon which the conclusions and recommendation are developed for the fix-in-place seepage measures.

2.1 Existing Condition Report

The Corps existing condition report indicates that pervasive underseepage concerns exist through the Sutter Feasibility Study area. Each reach has existing conditions where design criteria are not met for embankment geotechnical requirements (i.e. underseepage and stability). Breach failure before overtopping has happened three times during the life of this levee system (1909, 1911, and 1955) near Shanghai Bend, just south of Yuba City; the 1955 breach resulted in 38 fatalities. Without seepage control measures, the system will be subject to failure at less-than-overtopping loading conditions in the future. Levee modifications to the levee systems of the Yuba Basin on the opposite side of the Feather River, including Reclamation District 784 and the City of Marysville levees, have improved or are in the process of improving adjacent systems. Historically, at least one levee system on the Feather River has failed during each major flood during the last century.

2.2 External Document Review

Previous work for this project includes studies done by the local sponsors, the State of California Department of Water Resources and the Sutter Butte Flood Control Agency. This work was reviewed and utilized as part of the basis of our evaluation of the existing conditions , and used to develop the conclusions and recommendations presented for the Sutter – Butte Feasibility Study. These reports indicate that the levees of the Sutter –Butte Basin, including the east (left) levee of the Sutter Bypass from Wadsworth Canal to the Feather River, and the Feather River west (right) levee from Thermolito Afterbay Dam to the Sutter Bypass (see Map Plate 1), do not meet steady-state stability criteria for underseepage. The extent to which each reach fails to meet design criteria varies, but without exception, each reach has locations that do not meet underseepage criteria, often in multiple locations.

2.3 Key Assumptions

The following assumptions were used in the evaluation of the existing conditions (including for a potential ring levee around Yuba City, bypass and setback levees.) and for the analyzed alternatives.

- Levees will be modified or constructed considering seepage control or stability measures to provide embankment performance in accordance with USACE criteria for the entire levee system.
- Seepage and stability mitigation will be provided by construction of a cut-off wall along the entire system for all alternatives. Alternative selections are not based on a choice of seepage or stability mitigation measure. Selection of different mitigation measures such as seepage berms, relief wells, or cut-off walls along each reach is considered a design refinement and will be addressed in PED. Cutoff walls are less impactful to the environment and do not require the acquisition of real estate. The cost of soil-bentonite (SB) cutoff walls is competitive with berm construction.
- The exception to cutoff walls as seepage control mitigation measures are:

  Existing Relief Well Systems: used to the maximum extent possible in the reaches south of Yuba City at Shanghai Bend and Abbot Lakes/Star Bend.

  Seepage and Stability Berms: used in the northern reach near Thermalito Afterbay. The sponsor had previous indicated an interest and using gravel from the Oroville Goldfields dredger tailings to facility environmental mitigation. The presence of this gravel very close to the levee makes berm construction highly cost-effective in this area.

- Fix-in-place levee modifications have adequate subsurface geotechnical data for the development of feasibility level alternatives. For ring, setback, and bypass levees, there is no geotechnical subsurface data meeting the needs of a feasibility study. In order to develop feasibility recommendations that provide a reasonable basis for comparing alternatives, the structural measures for existing levees and new levees should ideally have a similar level of detail. This would best be achieved by having subsurface information and loading conditions developed to the same level of detail for new and existing levees. Since this is not possible without subsurface data on the ring, bypass and setback levee measures, a parametric approach was used. This approach was based only on a generalized understanding of the subsurface condition in Sutter and Butte counties, combined with past experience on fix-in-place and setback construction projects in the area (Yuba Basin).

- The parametric approach uses the maximum and minimum expected value concept. For example, design of a cutoff wall used estimates of the minimum and maximum expected depth and percentage of reach length requiring seepage control. After the maximum and minimum values were chosen, they were used to guide the selection of expected value. The median value was not used directly. These values were provided to civil designer so that quantities could be developed.
- New levees founded on older alluvium and located away from depressions, ditches or canals will likely require less seepage control (e.g. cutoff walls, seepage berms, etc) by percent of length.
- New levees will require more seepage control by percentage of length if the levee is taller (e.g. 8 feet or more) and less if it is shorter (e.g. 8 feet or less).
- For existing and new levees, deep cutoffs requiring specialized technology (e.g. deep soil mixing, jet grouting, etc) were considered design refinements and are not recommended at the feasibility level. It is expected that the reaches requiring the use of these methods will be short (e.g. bridge abutments or major utility crossing which may be left in place, etc). The increased cost of deep soil mixing (DSM) at $25 per square foot versus SB cutoff walls using slurry methods and excavators at $12 per square foot is significant; however, the percentage of the reaches is likely less than one percent of the entire project length. If long reaches requiring deep seepage control were encountered, the design level determination would most likely be the utilization of seepage using seepage berms or relief wells in lieu of cutoff walls.
- For areas where levee modification is required, including the addition of seepage control or stability mitigation, the resulting levee after construction will provide 1V:2H landside and 1V:3H waterside slopes and a 20-ft crest width.
- O&M easements for existing levees will be provided in the design of reaches requiring modification to not less than that prescribed by the current O&M manual. Additional O&M should be required to 15 or 20 ft of the landside to when feasible.
- Vegetation removal for ETL 1110-1-571 compliance will require at a minimum, removal of all non-compliant vegetation on the upper two-thirds of the waterside of the levee, the crest, landside slope and the existing landside easement.(if less than 15 feet, than a flooding easement will be required at least 15 feet from the landside toe free of woody vegetation)
- Seismic evaluation will be performed later in the Feasibility Study process, but seismic considerations are not considered to have a significant impact in alternative selection. Seismic analyses will be performed only to evaluate the extent of the damages but the levee alternatives will be not be affected by the seismic evaluation.
- A levee design template will be used by the civil and cost engineers to develop quantities based on inputs from this geotechnical engineering report. This template was developed by URS, and takes input for several key factors of a levee modification design (e.g. existing levee height and crest width, new depth of cutoff wall or width of seepage berm) and provides a parametric cost estimate for the levee segment for which this template is applied. The typical sections used are provided in the plates for Typical Drawing in this report.

3.0 CONCLUSIONS

The following geotechnical engineering conclusions are based on the evaluation of the subsurface conditions identified along the levees part of the Sutter basin and on the past performance of these levee and of the levees east site of Feather River. These conclusions are based heavily on engineering judgment which is guided strongly by experience in the local region and knowledge of the local soils.

3.1 General Conclusions for Fix-in-Place Alternatives

Based on the results of seepage and stability modeling, the existing levee system fails seepage design criteria within every reach, typically within multiple subreaches, and at a range of water surface profiles. The existing levee system fails to meet seepage and stability to varying degrees through the system, and underseepage mitigation in every reach is required to varying degrees.

3.2 Conclusions by Structural Segment

The conclusions are presented in clockwise order beginning with the Wadsworth Canal at the East/West Interceptor and progressing along the Sutter Bypass and Feather River to Thermalito Afterbay. Structural measure designations were developed by the Sutter PDT, and are broken down by geotechnical reach for the fix-in-place alternatives.

3.2.1 Wadsworth Canal (S7J)

The levee in this reach is approximately 20 ft in height (as measured from the landside toe to crest) at the downstream end, and the upper one-half of the segment, the levee is less than 7 feet tall, dropping to 3 feet at the upstream connection to the interceptor canals. A one-half mile long, 30 to 60 ft deep centerline soil-cement-bentonite (SCB) cutoff wall was constructed in this segment by the Corps near the downstream connection with the Sutter Bypass. Exploration performed by the State of California as part of their levee evaluation show that upstream of the cutoff wall, borings indicate sand layers covered by a thin impervious blanket or next to the surface which may require seepage control. Considering the adequate existing cutoff wall, the underseepage concerns upstream of the cutoff wall, and the lack of height of the levee upstream, a seepage control consisting of a seepage cut-off wall is recommended along approximately 25% of the segment. Based on the geotechnical conditions and of the existing cut-off wall, the depth varies between 20 and 50 feet, with an average depth of 40 ft.

3.2.2 Sutter Bypass (S7I)

The geotechnical and geomorphologic data provided by the State of California for the levee evaluation studies were used to evaluate the existing conditions and to recommend the mitigation alternatives. Exploratory borings and cone penetrometer testing (CPT) spaced at 1000 ft intervals, a long history of general past performance issues, and a series of seepage control measures of unknown quality and performance (toe drains, small seepage berms) indicate that additional seepage mitigation is required. The geology of this area consists of shallow basin deposits overlying hard pan layers. The general direction of geologic trend of soil deposits is roughly perpendicular to the levee, meaning a high frequency of small sand deposits is likely. The borrow pit cut through the hardpan layer near the waterside levee toe for construction of the existing levee creates a direct seepage connection to underlying sand layers. These factors indicate this levee has seepage concerns. Seepage cut-off wall is recommended for 100% of the reach, the depths of positive cutoff walls being relatively shallow (30 ft depth) but vary as much as 20 to 50 ft.

3.2.3 Sutter Bypass (S7H)

Borings and CPTs spaced at 1000 ft intervals, a long history of general past performance issues, and a series of seepage control measures of unknown quality and performance (toe drains, small seepage berms) indicate that additional seepage mitigation is required. The geology of this area consists of shallow basin deposits overlying hardpan clay layers. The general direction of geologic trend of soil deposits is roughly perpendicular to the levee, meaning a high frequency of small sand deposits is also likely and not easily detected by 1000-ft spaced borings. This reach contains Gilsizer Slough which is the active overbank flow channel from the Feather River to the west. The borrow pit cut through the hardpan layer near the waterside levee toe to construct the existing levee creates a direct seepage connection to underlying sand layers. These factors indicate this levee has seepage concerns and combined with the borings at Gilsizer Slough the cutoff wall may be deeper than other reaches of the Sutter Bypass. Seepage cut-off wall is recommended for 100% of the reach, the depths of positive cutoff walls relatively shallow (30 ft depth) but vary as much as 20 to 75 ft.

3.2.4 Sutter Bypass (S7G)

Borings and CPT spaced at 1000 ft intervals, a long history of general past performance issues, and a series of seepage control measures of unknown quality and performance (toe drains, small seepage berms) indicate that seepage mitigation is required. The geology of this area consists of shallow basin deposits overlying hardpan layers. The general direction of geologic trend of soil deposits is roughly perpendicular to the levee, meaning a high frequency of small sand deposits is likely. The borrow pit cut through the hardpan near the waterside levee toe for construction of the existing levee creates a direct seepage connection to underlying sand layers. These factors indicate this levee has seepage concerns. The levee height in this reach is the tallest of the entire Sutter Bypass. Therefore, seepage cut-off wall is recommended for 100% of the reach, the depths of positive cutoff walls being relatively shallow (30 ft depth) but vary as much as 20 to 50 ft.

3.2.5 Feather River (S7F)

The levee in this reach is tall (20 plus ft), and irrigation ditches are excavated near the landside levee toe. Borings and CPTs indicate shallow sand layers and thin to moderate blanket thicknesses that may lead to seepage sand boils. Seepage cut-off walls are recommended for 75% of the reach, the depths of positive cutoff walls to being moderately deep, 50 ft, but vary as much as 20 to 75 ft.

3.2.6 Feather River (S7E)

The levee in this reach is tall (20 plus ft), and has irrigation ditches excavated near the landside toe. Borings indicate deep sand layers and thin to moderate blanket thicknesses that may lead to high gradients and sand boils, however portions of the reach have deep clay foundations. A portion of this reach has existing relief wells. Seepage cutoff walls are recommended for 25% of the reach, and relief well collection system on additional 25% of the reach length. We expect depths of positive cutoff walls to be moderately deep, 65 ft, but vary as much as 20 to 75 ft.

3.2.7 Feather River (S7D/S5D)

The levee in this reach is tall (20 plus ft), and has irrigation ditches excavated near the landside toe. Borings indicate shallow and deep sand layers and thin to moderate blanket thicknesses that may lead to high gradients at the levee toe; however, portions of the reach have deep clay foundations. A portion of this reach has existing relief wells. Seepage cutoff walls is recommended for 50% of the reach and an additional 15% will require relief well collection system modification. We expect depths of positive cutoff walls to be moderately deep, 65 ft, but vary as much as 20 to 75 ft.

3.2.8 Feather River (S7C)

The levee in this reach is tall (20 plus ft), and has irrigation ditches excavated near the landside toe. Borings indicate shallow and deep sand layers and thin to moderate blanket thicknesses that may lead to high gradients at the levee toe. Based on these considerations, seepage cut-off wall is recommended for 75% of the reach. The depths of positive cutoff walls should be moderately deep, 65 ft, but vary as much as 20 to 75 ft.

3.2.9 Feather River (S7B)

The levee in this reach variable in levee height from as little as 3 feet to as much as 15 ft, and has ditches excavated along the landside toe, including the major irrigation Sutter Butte Canal. Borings indicate shallow sand layers that may lead to high gradients at the levee toe for parts of this reach. Seepage control may be required as part the method in which the Sutter Butte Canal will be addressed. Based on these considerations, seepage cutoff walls are necessary for 100% of the reach, and we expect depths of positive cutoff walls to be moderately deep, 65 ft, but vary as much as 20 to 75 ft.

3.2.10 Feather River (S7A)

The levee in this reach varies in height from 3 to 4 feet to 18 ft. Borings indicate shallow sand layers and thin to moderate blanket thicknesses that may be seepage paths. The levee may require modification to address insufficient geometry in the Goldfields portion. Based on these considerations, seepage mitigation is recommended for 50% of the segment, seepage and stability mitigation for 25% of the segment and stability mitigation is required for 25% of the reach. Seepage berms and stability berms may be the best mitigation here, due to the sponsor's interest in using local borrow sources.

3.3 General Conclusions for New Levees (Setback Levees, Ring Levees and Cross Levees)

Based on experience with new levees, including setback levees previously constructed within the basin (e.g. Star and Shanghai Bends in Levee District 1, and the Feather River Setback Levee in Reclamation District 784), new levees will require a cutoff wall for some percentage of length, though typically to a lesser extent than existing levees. This is likely due to the distance from active river channels, and the lower likelihood of founding the levee on poor foundation conditions. Therefore we conclude that new levees far from the active river system will need seepage mitigation measures at a lower rate per unit length.

3.4 Conclusions by Reach

The conclusions for new levee segments are presented in clockwise order beginning with the Wadsworth Canal at the East/West Interceptor and progressing along the Sutter Bypass and Feather River to Thermalito Afterbay.

3.4.1 Sutter Bypass Setback Levee (S9I)

Seepage mitigation is assumed to be similar to the associated segment of the existing Sutter Bypass levee. Increased seepage path length by moving the levee back from the waterside borrow pit may result in a lower percentage of cutoff walls, but it will not eliminate the need for seepage control.

3.4.2 Sutter Bypass Setback Levee (S9H)

Seepage mitigation is assumed to be similar to the associated segment of the existing Sutter Bypass levee. Increased seepage path length by moving the levee back from the waterside borrow pit may result in a lower percentage of cutoff walls, but it will not eliminate the need for seepage control.

3.4.3 Sutter Bypass Setback Levee (S9G)

Seepage mitigation is assumed to be similar to the associated segment of the existing Sutter Bypass levee. Increased seepage path length by moving the levee back from the waterside borrow pit may result in a lower percentage of cutoff walls, but it will not eliminate the need for seepage control.

3.4.4 Lower Feather River Setback Levee (S11)

Depending on the final alignment, seepage control may be reduced below levels on the Sutter Bypass and south Feather River segment, due to the presence of an intact waterside blanket.

3.4.5 Star Bend Setback Levee (S12)

Based on borings for the Star Bend Setback Levee, seepage mitigation is required through the entire reach to depths of approximately 40 to 60 ft.

3.4.6 Yuba City, Ring Levee – South (S4) and J-Levee - South (S6)

Ring levees are assumed to be relatively short, and if they are constructed in areas with an intact blanket on the waterside of the levee (likely in this area based on experience), a lesser percentage of cutoff walls is expected. Seepage control is likely for one-third of this alignment.

3.4.7 Yuba City, Ring Levee – West (S4), J-Levee – West Lower (S6) and J-Levee – West Upper (S6)

Ring levees are assumed to be relatively short, and if they are constructed in areas with an intact blanket on the waterside of the levee (likely in this area based on experience), a lesser percentage of cutoff walls is expected. Seepage control is likely for one-third of this alignment.

3.4.8 Yuba City, Ring Levee – North (S4)

Ring levees are assumed to be relatively short, and if they are constructed in areas with an intact blanket on the waterside of the levee (likely in this area based on experience), a lesser percentage of cutoff walls is expected. Seepage control is likely for one-third of this alignment.

3.4.9 Northern Feather River Setback Levee (S10)

There is more uncertainty with the foundations conditions in this reach. Deep coarse-grained materials are known to exist in the Biggs area, so seepage mitigation is likely to be extensive. Also, this area was an overflow area for the Feather River during the 1800's and early 1900's.

This indicates that shallow sand layers and a high-frequency of sand lenses or stringers are likely to be present. Seepage mitigation is likely for 100% of the reach.

4.0 RECOMMENDATIONS

Recommendations for typical design sections (templates), new levees, construction staging and hauling, borrow, and structural segment-specific recommendations are provided within this section.

4.1 Typical Design Sections

Typical design sections are provided in the Plates section of this memorandum. These include all typical designs, which correspond to the templates used by URS in their parametric analysis performed for this project.

4.2 New Levee Design

All levees designs shall provide 1V:3H landside and waterside slope with a 20 ft crest width and 15 ft waterside and 20 ft landside O&M easements. Table 3 provides typical levee corridor widths, and Typical Detail Plates 6 and 7 provide the geometrical relationships of new levees with and without SB slurry walls.

4.3 Construction Staging and On-Site Hauling

Levee modification projects typically require long linear haul routes for on-site construction and staging areas situated at periodic intervals. For long slurry wall projects, we recommend that 2 acre sites every 2500 linear feet of levee. Typically slurry wall construction requires bentonite slurry ponds, and typically these are located about every one-half mile. The bentonite slurry is pumped to the active excavation site through pipes. Major staging areas are also required. Major staging areas are where equipment maintenance, employee parking and job trailers are located. Assume 5 acres for a typical 5 mile long slurry wall project. Multiple concurrent cutoff wall projects may require a large central staging area.

4.4 Borrow

We recommend that 15 miles as a typical haul distance for borrow. Values as low as 10 or as high as 30 may be reasonable, but 15 miles is conservative, and higher values are not warranted considering that suitable material can be typically found within the basin. It is likely that borrow will become cost prohibitive if not obtained within this distance, primarily due to air quality impacts. A conservative shrinkage percentage should be used. We recommend 15% at the feasibility level.

4.5 Seepage and Stability Mitigation by Structural Measure

These recommendations are presented as ranges (i.e. remediate reach X with a cutoff wall along 50 to 100% of the reach with a depth ranging from 20 ft to 60 ft). The measures are presented in a clockwise organization beginning on the Wadsworth Canal and progressing through the Sutter Bypass and Feather River to Thermalito Afterbay.

4.6 Fix-in-Place Measures

This section presents recommendation for seepage and stability mitigation, presented by fix-in—place structural segment.

4.6.1 Wadsworth Canal

Construct a centerline SB cutoff wall for 25% of the reach, to an expected depth of 40, with a range of typical depths of 20 to 50 ft.

4.6.2 Sutter Bypass (S7I)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 30 ft, with a range of typical depths of 20 to 50 ft.

4.6.3 Sutter Bypass (S7H)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 30 ft, with a range of typical depths of 20 to 75 ft.

4.6.4 Sutter Bypass (S7G)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 30 ft, with a range of typical depths of 20 to 50 ft.

4.6.5 Feather River (S7F)

Construct a centerline SB cutoff wall for 75% of the reach, to an expected depth of 50 ft, with a range of typical depths of 20 to 75 ft.

4.6.6 Feather River (S7E)

Construct a centerline SB cutoff wall for 25% of the reach, to an expected depth of 40 ft, with a range of typical depths of 20 to 75 ft.

4.6.7 Feather River (S7D/S5D)

Construct a centerline SB cutoff wall for 50% of the reach, to an expected depth of 50 ft, with a range of typical depths of 20 to 75 ft. Modify existing relief well systems by reconfiguration of the collection system, including lower well head rise height from 4 above grade to 1 ft below grade for 15% of the reach for a total of 65% seepage mititgation.

4.6.8 Feather River (S5C)

Construct a centerline SB cutoff wall for 75% of the reach, to an expected depth of 65 ft, with a range of typical depths of 20 to 75 ft.

4.6.9 Feather River (S5B)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 65 ft, with a range of typical depths of 20 to 75 ft.

4.6.10 Feather River (S5A)

Construct a seepage berm for 75% of the reach, to height of 5 ft and width of 12 ft. Construct a stability berm for 50% of the reach, with a height of 12 ft and top width of 12 ft. The resulting segment recommendation is for 50% of the segment to have a seepage berm only, 25% to have a stability berm only and 25% to have a combined seepage and stability berm.

4.7 Ring, Bypass and Setback Measures

This section presents recommendation for seepage control for new levees, presented by ring, bypass or setback levee structural segment.

4.7.1 Sutter Bypass Setback Levee (S9I)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 35 ft, with a range of typical depths of 20 to 50 ft.

4.7.2 Sutter Bypass Setback Levee (S9H)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 35 ft, with a range of typical depths of 20 to 50 ft.

4.7.3 Sutter Bypass Setback Levee (S9G)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 35 ft, with a range of typical depths of 20 to 50 ft.

4.7.4 Lower Feather River Setback Levee (S11)

Construct a centerline SB cutoff wall for 50% of the reach, to an expected depth of 35 ft, with a range of typical depths of 30 to 70 ft.

4.7.5 Star Bend Setback Levee (S12)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 65 ft, with a range of typical depths of 30 to 70 ft.

4.7.6 Yuba City, Ring Levee – South (S4) and J-Levee - South (S6)

Construct a centerline SB cutoff wall for 34% of the reach, to an expected depth of 35 ft, with a range of typical depths of 30 to 60 ft.

4.7.7 Yuba City, Ring Levee – West (S4), J-Levee – West Lower (S6) and J-Levee – West Upper (S6)

Construct a centerline SB cutoff wall for 34% of the reach, to an expected depth of 35 ft, with a range of typical depths of 30 to 60 ft.

4.7.8 Yuba City, Ring Levee – North (S4)

Construct a centerline SB cutoff wall for 34% of the reach, to an expected depth of 35 ft, with a range of typical depths of 30 to 60 ft.

4.7.9 Northern Feather River Setback Levee (S10)

Construct a centerline SB cutoff wall for 100% of the reach, to an expected depth of 75 ft, with a range of typical depths of 20 to 75 ft.

5.0 REFERENCES

- State of California, Department of Water Resources Urban Levee Geotechnical Evaluations Program, Phase 1 Preliminary Geotechnical Evaluation Report, Sutter Study Area, March 2008, URS
- State of California, Department of Water Resources Urban Levee Geotechnical Evaluations Program, Phase 1 Geotechnical Data Report, Sutter Study Area, November 2008, URS
- State of California, Department of Water Resources Urban Levee Geotechnical Evaluations Program, Supplemental Geotechnical Data Report, Sutter Study Area, April 2010, URS
- Sutter Butte Flood Control Agency, Pre-Design Formulation Report, West Feather River Project, August 2011, prepared by HDR, Wood Rodgers, URS and MHM
- Sutter Butte Flood Control Agency, 60% design submittal package, West Feather River Project, January 2012, prepared by HDR, Wood Rodgers, URS and MHM

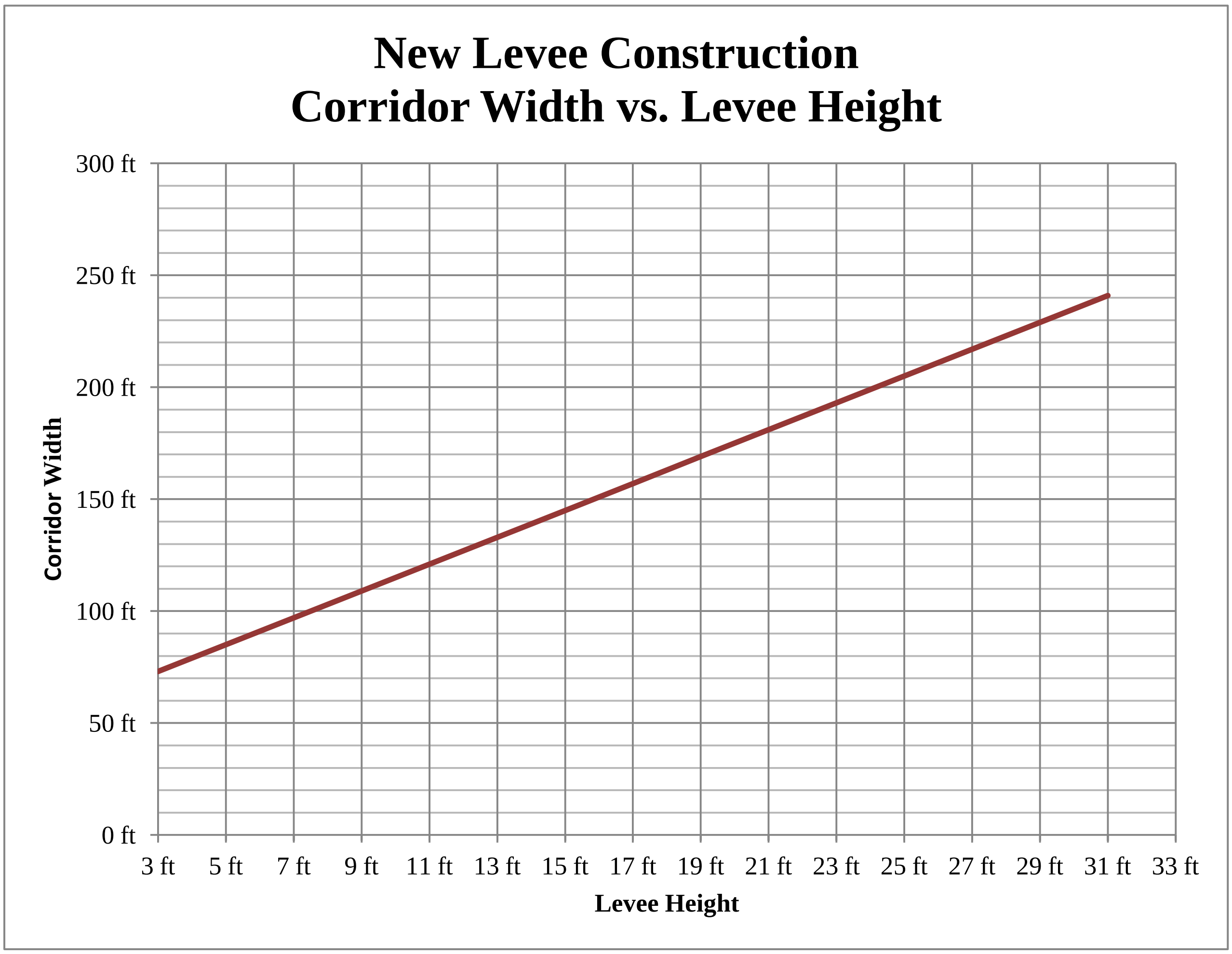
TABLES

Table 1, Summary of Recommendations by Structural Element of the Existing Levee System							
Structural Element	Name	PLM	PLM	Feature	% Length	Depth/Height	Width
S7J	Wadsworth Canal	0	4.66	SB CL Cutoff wall	25	40 ft (20 to 50)	N/A
S7I	Sutter Bypass	4.4	12.65	SB CL Cutoff wall	100	30 ft (20 to 50)	N/A
S7J	Sutter Bypass	12.65	14.35	SB CL Cutoff wall	100	30 ft (20 to 75)	N/A
S7H	Sutter Bypass	14.35	22.37	SB CL Cutoff wall	100	30 ft (20 to 50)	N/A
S7F	Feather River	MA03 0.00 LD1 0.00	MA 5.19 LD1 2.70	SB CL Cutoff wall	75	50 ft (20 to 75)	N/A
S7E	Feather River	LD1 2.70	LD1 6.20	SB CL Cutoff wall	25	65 ft (20 to 75)	N/A
S7D	Feather River	LD1 6.20	LD1 10.5	SB CL Cutoff wall[1]	50	65 ft (20 to 75)	N/A
S5D/S4-EAST	Feather River	LD1 10.5	LD1 16.65	SB CL Cutoff wall[1]	50	65 ft (20 to 75)	N/A
S5C	Feather River	LD9 0.00	LD9 5.50	SB CL Cutoff wall	75	65 ft (20 to 75)	N/A
S5B	Feather River	LD9 5.50| MA16 0 MA7 0.00	LD9 6.24 MA16 4.69 MA7 1.80	SB CL Cutoff wall	100	65 ft (20 to 75)	N/A
S5A	Feather River	MA7 1.80 MA7 HB 0.00	MA7 12.07 MA7 HG 0.08	Stability Berm Seepage Berm	50 75	12 ft 150 ft	12 ft 5 ft

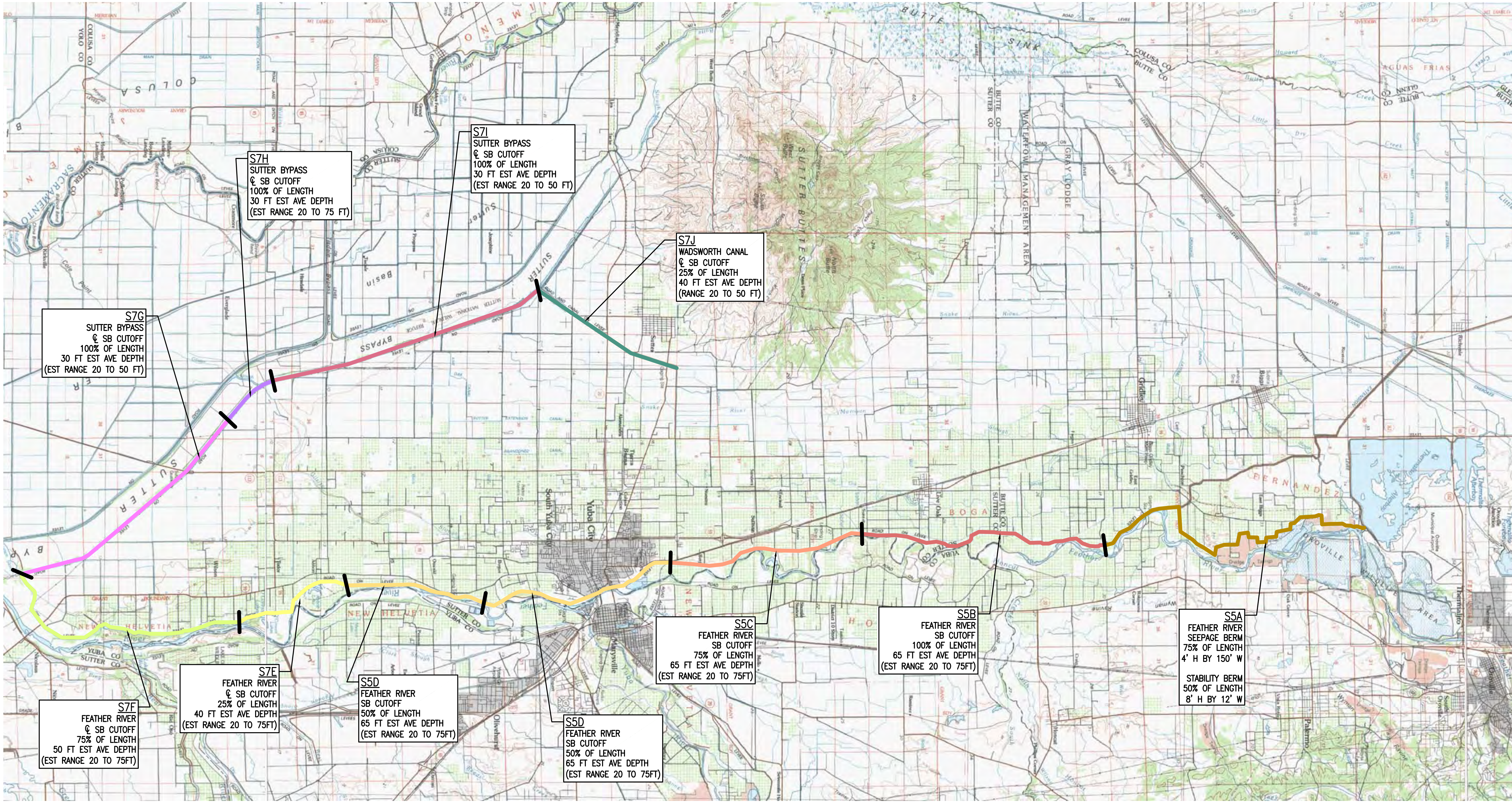
[1] Modifications to the existing relief well system are required per section 4.6.7.

Table 2 Summary of Recommendations for Segments of Ring Levees, Setback Levees, and Bypass Levees							
Structural Element	Name	PLM	PLM	Feature	% Length	Depth/Height	Width
S9I	Sutter Bypass Setback	N/A	N/A	SB CL Cutoff wall	25	35 ft (20 to 50)	N/A
S9H	Sutter Bypass Setback	N/A	N/A	SB CL Cutoff wall	25	35 ft (20 to 50)	N/A
S9G	Sutter Bypass Setback	N/A	N/A	SB CL Cutoff wall	25	35 ft (20 to 50)	N/A
S11	Sutter Bypass and Feather River Confluence Setback Levee	N/A	N/A	SB CL Cutoff wall	50	35 ft (20 to 50)	N/A
S12	Star Bend Setback	N/A	N/A	SB CL Cutoff wall	100	65 ft (30 to 70)	N/A
S4-SOUTH **S6-SOUTH**	Yuba City J-Levee	N/A	N/A	SB CL Cutoff wall	34	35 ft (20 to 60)	N/A
S4-WEST **S6-WEST**	Yuba City J-Levee	N/A	N/A	SB CL Cutoff wall	34	35 ft (20 to 60)	N/A
S6-WEST-UPPER	Yuba City J-Levee	N/A	N/A	SB CL Cutoff wall	34	35 ft (20 to 60)	N/A
S4-NORTH	Yuba City J-Levee	N/A	N/A	SB CL Cutoff wall	34	35 ft (20 to 60)	N/A
S10	Northern Feather River Setback Levee	N/A	N/A	SB CL Cutoff wall	100	75 ft (20 to 50)	N/A

TABLE 3, Levee Geometrical and O&M Requirements for New Levees										
Levee Height	Landside O&M	Landside Slope	Landside Length	Landside Height	Crest Width	Waterside Slope	Waterside Length	Waterside Height	Waterside O&M	Corridor
3 ft	20 ft	1V:3H	9 ft	3 ft	20 ft	1V:3H	9 ft	3 ft	15 ft	73 ft
5 ft	20 ft	1V:3H	15 ft	5 ft	20 ft	1V:3H	15 ft	5 ft	15 ft	85 ft
7 ft	20 ft	1V:3H	21 ft	7 ft	20 ft	1V:3H	21 ft	7 ft	15 ft	97 ft
9 ft	20 ft	1V:3H	27 ft	9 ft	20 ft	1V:3H	27 ft	9 ft	15 ft	109 ft
11 ft	20 ft	1V:3H	33 ft	11 ft	20 ft	1V:3H	33 ft	11 ft	15 ft	121 ft
13 ft	20 ft	1V:3H	39 ft	13 ft	20 ft	1V:3H	39 ft	13 ft	15 ft	133 ft
15 ft	20 ft	1V:3H	45 ft	15 ft	20 ft	1V:3H	45 ft	15 ft	15 ft	145 ft
17 ft	20 ft	1V:3H	51 ft	17 ft	20 ft	1V:3H	51 ft	17 ft	15 ft	157 ft
19 ft	20 ft	1V:3H	57 ft	19 ft	20 ft	1V:3H	57 ft	19 ft	15 ft	169 ft
21 ft	20 ft	1V:3H	63 ft	21 ft	20 ft	1V:3H	63 ft	21 ft	15 ft	181 ft
23 ft	20 ft	1V:3H	69 ft	23 ft	20 ft	1V:3H	69 ft	23 ft	15 ft	193 ft
25 ft	20 ft	1V:3H	75 ft	25 ft	20 ft	1V:3H	75 ft	25 ft	15 ft	205 ft
27 ft	20 ft	1V:3H	81 ft	27 ft	20 ft	1V:3H	81 ft	27 ft	15 ft	217 ft
29 ft	20 ft	1V:3H	87 ft	29 ft	20 ft	1V:3H	87 ft	29 ft	15 ft	229 ft
31 ft	20 ft	1V:3H	93 ft	31 ft	20 ft	1V:3H	93 ft	31 ft	15 ft	241 ft



MAP PLATES



SUTTER BASIN

STUDY MAP SHOWING GEOTECHNICAL RECOMMENDATIONS BY REACH FOR FIX–IN–PLACE MEASURES

MEASURE NAMING CONVENTION
S = STRUCTURAL MEASURE
5, 7, ETC = DESIGNATION INDICATING ASSOCIATED ALTERNATIVE
H, G, I = GEOTECHNICAL FIX–IN–PLACE REACH DESIGNATION

SCALE IN FEET
0 7500 15000 30000

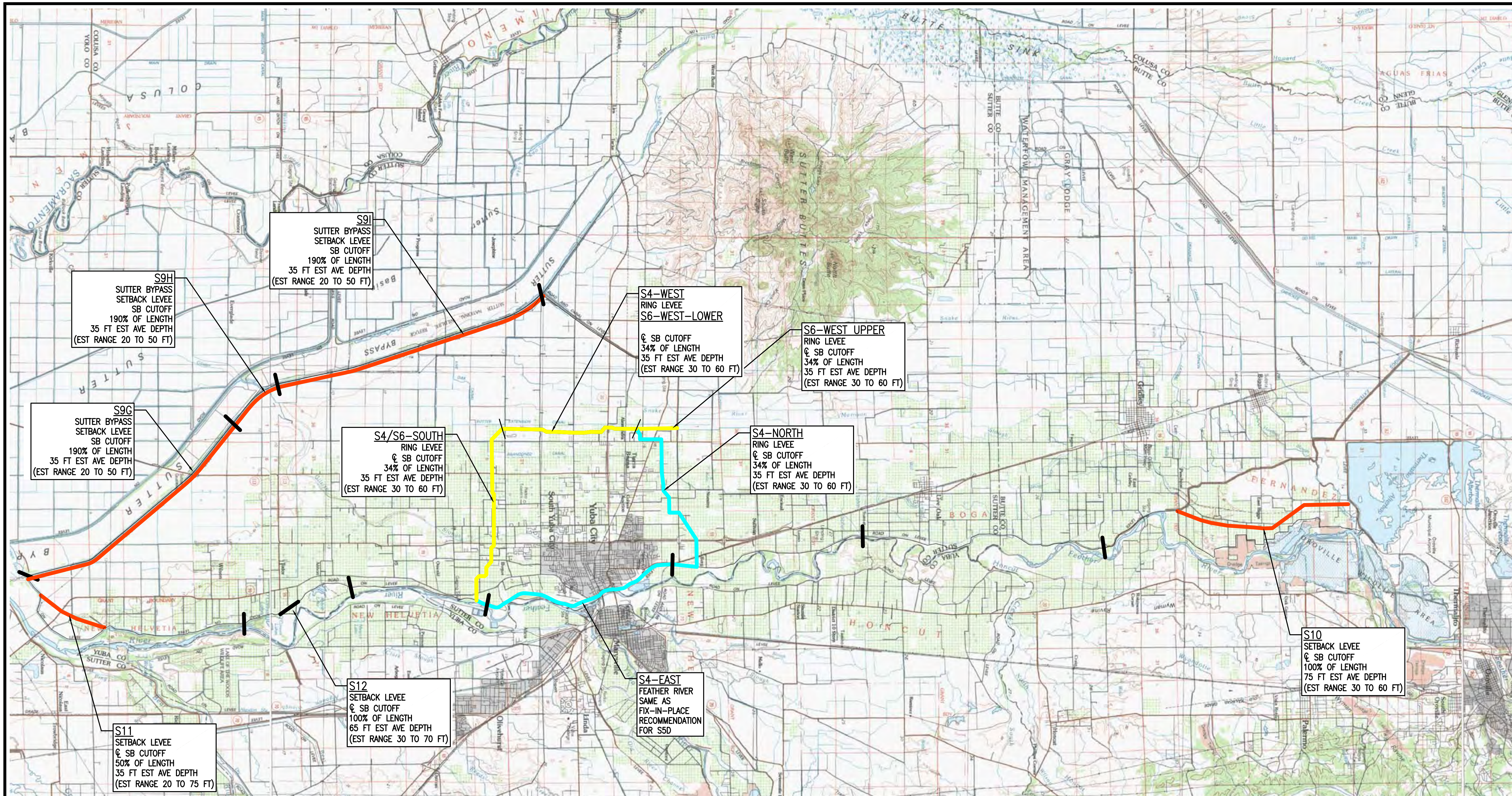
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER AND BUTTE COUNTIES — CALIFORNIA

SUTTER FEASIBILITY STUDY
TECHNICAL MEMORANDUM FOR ALTERNATIVE EVALUATION

GEOTECHNICAL RECOMMENDATIONS FOR FIX–IN–PLACE MEASURES

DATE:	SCALE:	
5/30/12	1" = 15000'	MAP 1



SUTTER BASIN

STUDY MAP SHOWING GEOTECHNICAL RECOMMENDATIONS BY REACH FOR RING, SETBACK, AND BYPASS MEASURES

MEASURE NAMING CONVENTION
S = STRUCTURAL MEASURE
5, 7, ETC = DESIGNATION INDICATING ASSOCIATED ALTERNATIVE
H, G, I = GEOTECHNICAL FIX-IN-PLACE REACH DESIGNATION

SCALE IN FEET

0 7500 15000 30000

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SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER AND BUTTE COUNTIES — CALIFORNIA

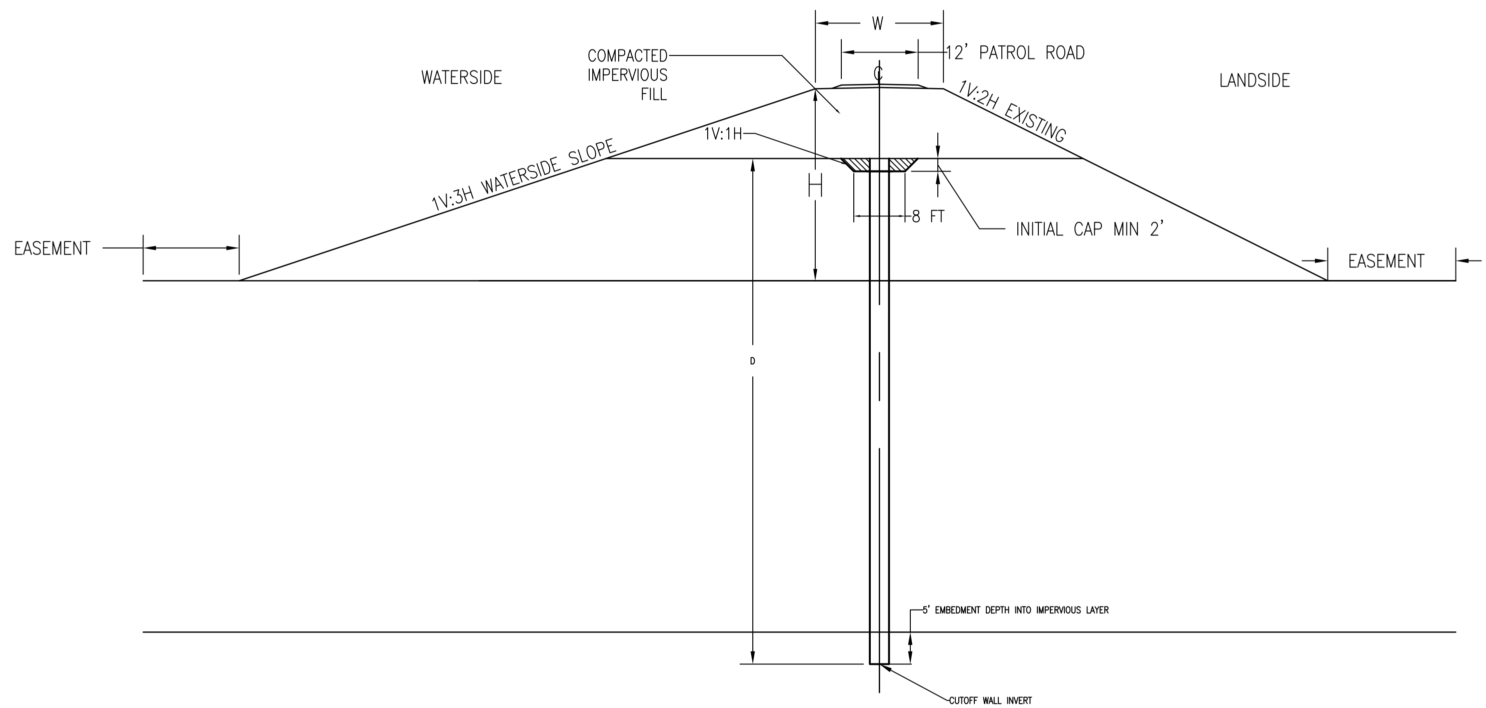
SUTTER FEASIBILITY STUDY
TECHNICAL MEMORANDUM FOR ALTERNATIVE EVALUATION

GEOTECHNICAL RECOMMENDATIONS
RING, SETBACK AND BYPASS ALTERNATIVES

DATE:	SCALE:	
5/30/12	1" = 15000'	MAP 2

TYPICAL DETAIL PLATES

TYPICAL LEVEE DETAILING–EXISTING LEVEE.dwg PLOT 1=30 EWJ 5/30/12

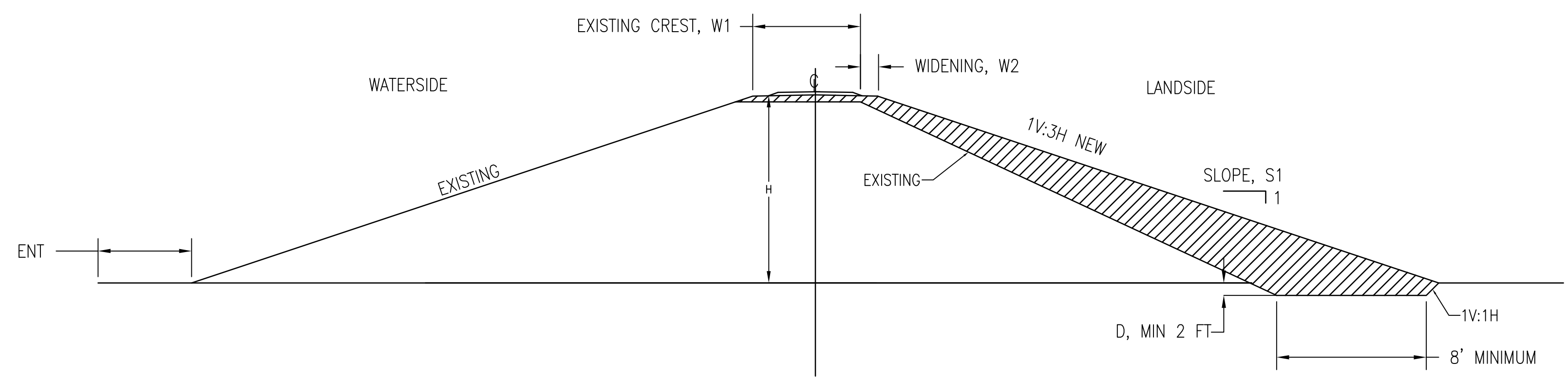


SUTTER BASIN

LEVEE REHABILITATION WITH CENTERLINE
SOIL–BENTONITE CUTOFF WALL
SLURRY METHOD

PARAMETERS:
D, CUTOFF DEPTH
H, LEVEE HEIGHT
H1, DEGRADE HEIGHT

DEPARTMENT OF THE ARMY SACRAMENTO DISTRICT CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA	
SUTTER AND BUTTE COUNTY	CALIFORNIA
SUTTER BASIN GEOTECHNICAL ENGINEERING TECHNICAL MEMORANDUM LEVEE REMEDIATION MEASURES SB SLURRY METHOD CUTOFF WALL TYPICAL SECTION	
DATE: 5/30/12	SCALE: NO SCALE
TYP 1	



SUTTER BASIN

LEVEE REHABILITATION

WIDENING AND SLOPE MODIFICATIONS

PARAMETERS:

D, KEY DEPTH
H, LEVEE HEIGHT
W1 EXISTING CREST WIDTH
W2, ADDITIONAL CREST
S1, SLOPE RUN PER 1' RISE

DEPARTMENT OF THE ARMY SACRAMENTO DISTRICT CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA	
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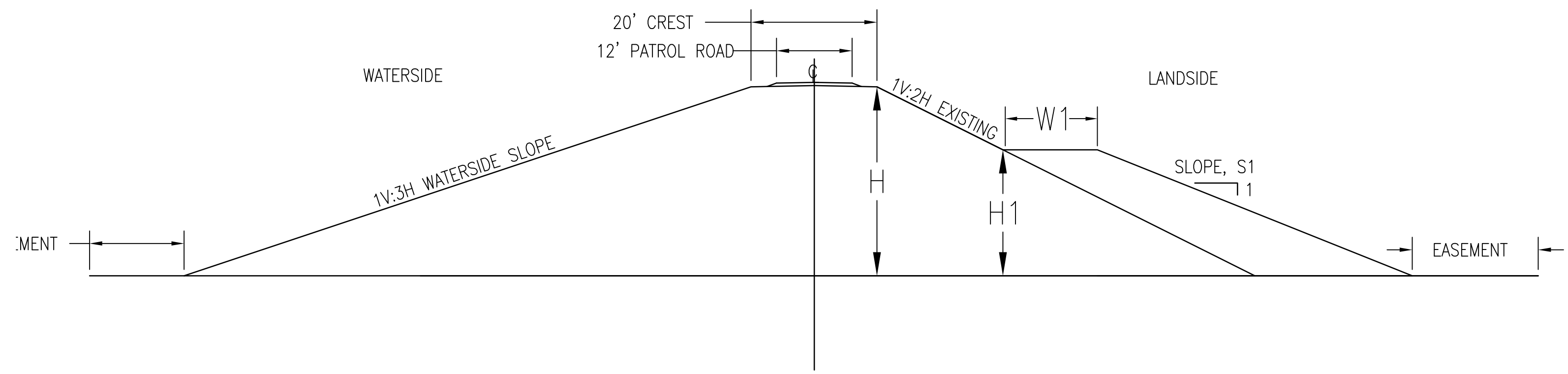
SUTTER BASIN GEOTECHNICAL ENGINEERING TECHNICAL MEMORANDUM LEVEE REMEDIATION MEASURES WIDENING AND SLOPE MODIFICATIONS TYPICAL SECTION	
DATE: 5/30/12	SCALE: NO SCALE
TYP 2	

TYPICAL LEVEE DETAILING–EXISTING LEVEE.dwg PLOT 1=30 EWJ 5/30/12



SUTTER BASIN

LEVEE REHABILITATION WITH LANDSIDE STABILITY BERM

PARAMETERS:
H, LEVEE HEIGHT
H1, BERM HEIGHT
W1, BERM WIDTH
S1, SLOPE RUN PER 1' RISE

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GEOTECHNICAL ENGINEERING
TECHNICAL MEMORANDUM
LEVEE REMEDIATION MEASURES
STABILITY BERM
TYPICAL SECTION

DATE:	SCALE:	
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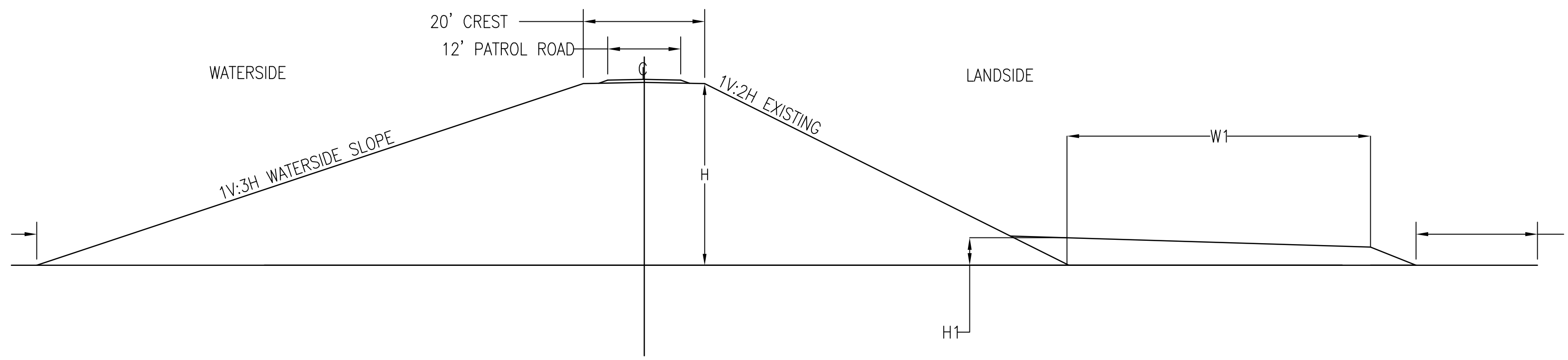

SUTTER BASIN

LEVEE REHABILITATION WITH SEEPAGE BERM

PARAMETERS:
H1, BERM HEIGHT
H2, BERM WIDTH
H, LEVEE HEIGHT

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GEOTECHNICAL ENGINEERING
TECHNICAL MEMORANDUM
LEVEE REMEDIATION MEASURES
SEEPAGE BERM
TYPICAL SECTION

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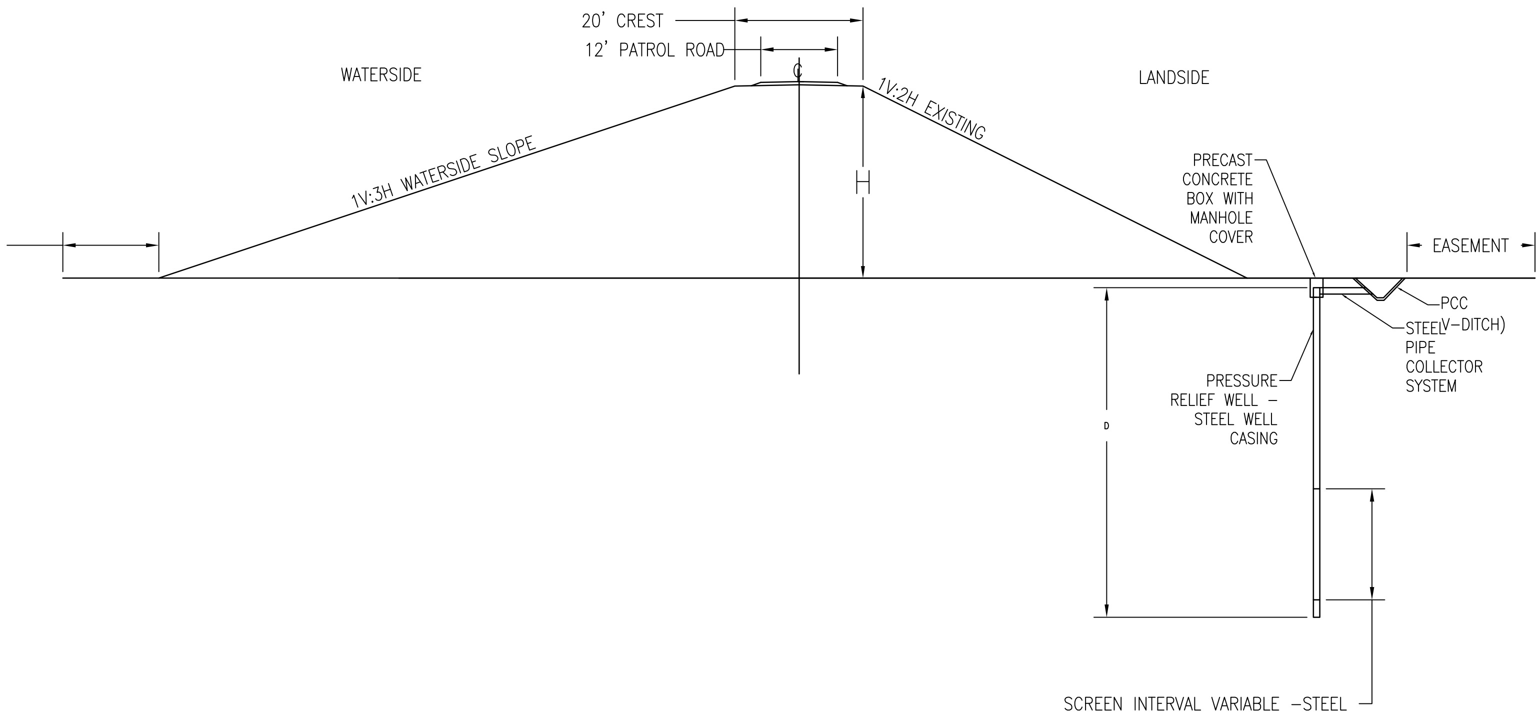

SUTTER BASIN

LEVEE REHABILITATION WITH PRESSURE RELIEF WELLS AND COLLECTOR SYSTEM

PARAMETERS:
D, WELL DEPTH
H, LEVEE HEIGHT

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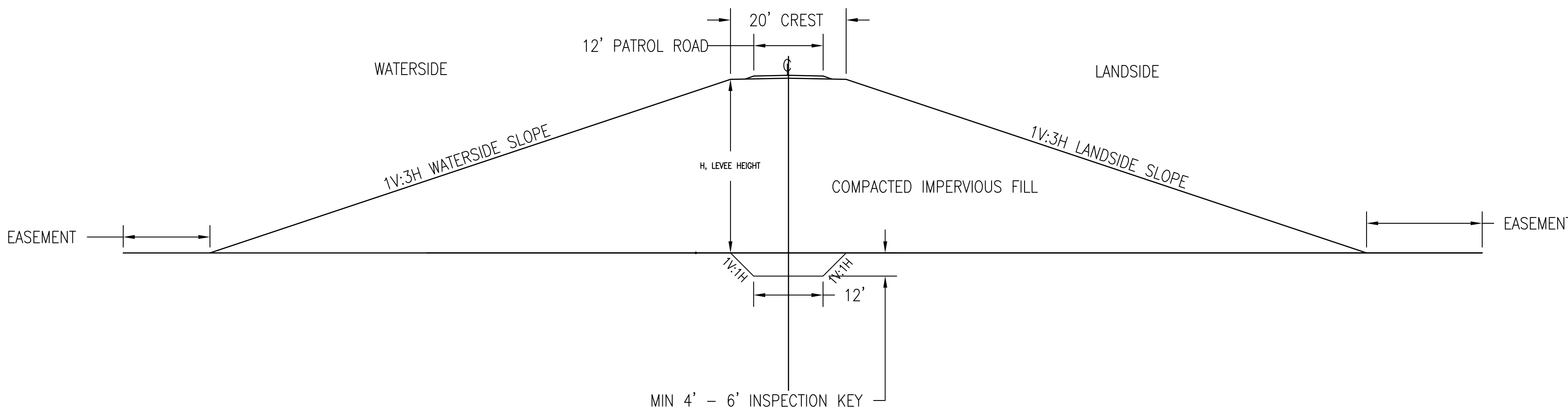
SUTTER BASIN
GEOTECHNICAL ENGINEERING
TECHNICAL MEMORANDUM
LEVEE REMEDIATION MEASURES
PRESSURE RELIEF WELLS
TYPICAL SECTION

DATE:	SCALE:	
5/30/12	NO SCALE	TYP 5

TYPICAL DETAILING–NEW LEVEE.dwg PLOT 1=30 EWJ 5/30/12



SUTTER BASIN

NEW LEVEE – NO SEEPAGE CONTROL

PARAMETERS:
H, LEVEE HEIGHT

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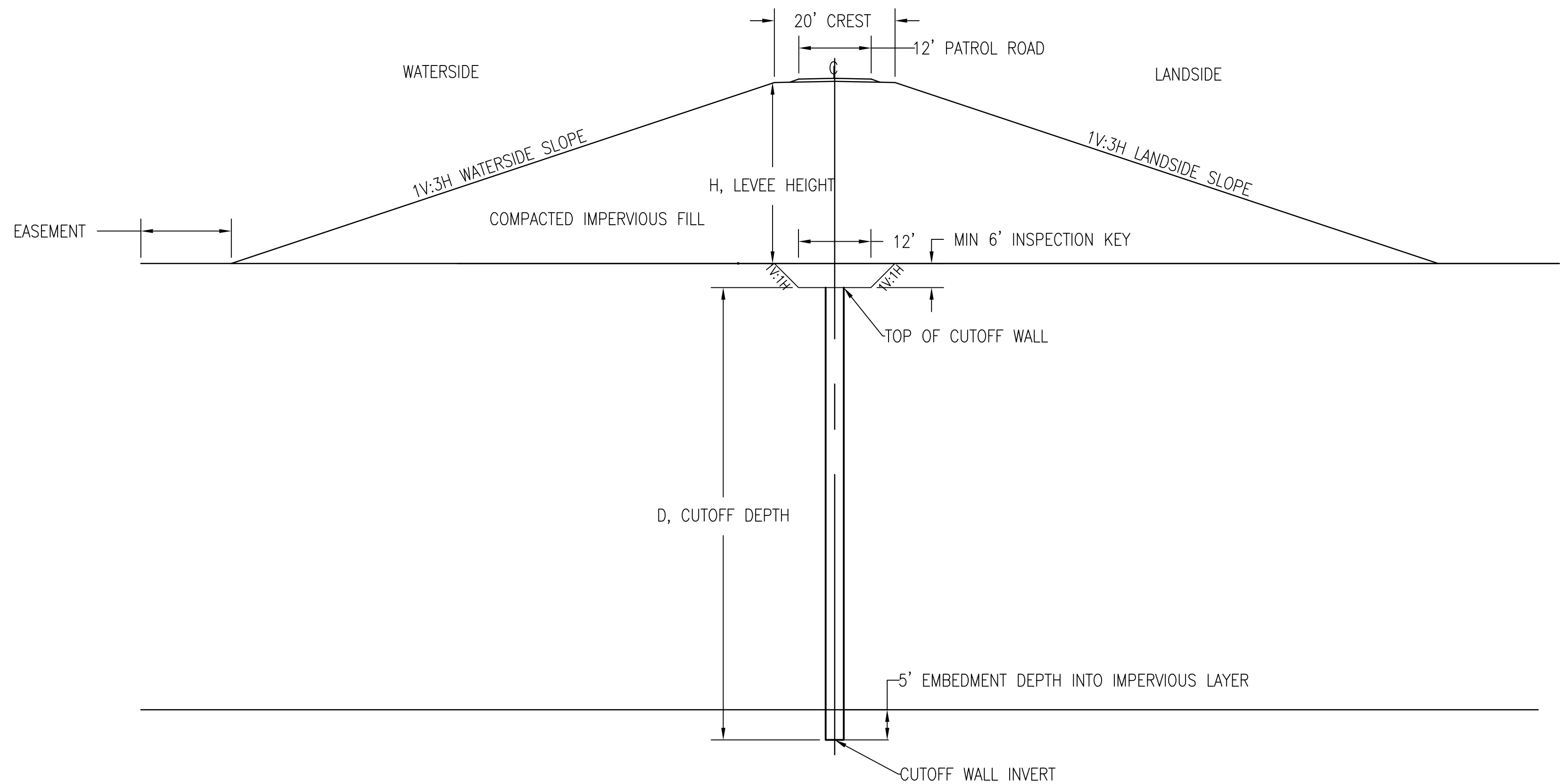
SUTTER ABD BUTTE COUNTIES CALIFORNIA
SUTTER BASIN
GEOTECHNICAL ENGINEERING
TECHNICAL MEMORANDUM
NEW LEVEE CONSTRUCTION
NO SEEPAGE CONTROL
TYPICAL SECTION

DATE:	SCALE:	
5/30/12	NO SCALE	TYP 6

TYPICAL DETAILING–NEW LEVEE.dwg PLOT 1=30 EWJ 5/30/12



SUTTER BASIN

NEW LEVEE – SOIL–BENTONTITE SLURRY WALL

PARAMETERS:
D, WELL DEPTH
H, LEVEE HEIGHT

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SUTTER ABD BUTTE COUNTIES CALIFORNIA

SUTTER BASIN
GEOTECHNICAL ENGINEERING
TECHNICAL MEMORANDUM
NEW LEVEE CONSTRUCTION
SOIL–BENTONITE SLURRY WALL
TYPICAL SECTION

DATE:	SCALE:	
5/30/12	NO SCALE	TYP 7

ENCLOSURE I

VEGETATION ETL COMPLIANCE MEMORANDUM

CESPK-PD-R **15 June 2012**
Revised 24 February 2013

MEMORANDUM FOR FILE: Sutter Basin Pilot Feasibility Study

SUBJECT: Compliance with ETL 1110-2-571

1. REFERENCES

a. Engineering and Design: Guidelines for landscape planting and vegetation management at levees, floodwalls, embankment dams, and appurtenant structures. Engineer Technical Letter (ETL), April 10, 2009.
b. White paper, Sutter Basin Pilot Feasibility Study, Engineer Technical Letter 1110-2-251 Compliance, dated 8 February 2013.
c. Feather River Sutter Basin Protection Area Periodic Inspection January 2010, Report No. 1.
d. Memorandum for Record. Sutter Feasibility Study – Summary of vegetation concerns impacting Sutter Project identified in the Periodic Inspection, dated August 9, 2012.
e. Process for Requesting Variance from Vegetation Standards for Levees and Floodwalls, Federal Register Notice (Friday, February 17, 2012).
f. Sutter Basin Feasibility Study, Future-Without Project Conditions Report, December 2011.
g. Corps Memorandum, Reconstruction of U.S. Army Corps of Engineers Structural Flood Damage Reduction Projects for which Non-Federal Interests are Responsible for Operation, Maintenance, Repair, Rehabilitation and Replacement, August 16, 2005.
h. Pre-Design Formulation Report, Feather River West Levee Project, Sutter-Butte Flood Control Agency, August 2011.

2. PURPOSE

The purpose of this memorandum is to assess the extent of vegetation that is in non compliance with ETL 1110-2-571 (Reference a), to assess the magnitude of the mitigation that might be required, and to identify potential strategies in dealing with this issue. For the study alternatives, impact scale, study risk, and ETL variance and ETL mitigation costs are also identified.

3. BACKGROUND

USACE Vegetation Management.

USACE Engineer Technical Letter 1110-2-571, *Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures*, dated 10 April 2009, provides guidance for maintenance of said structures in order to maintain the authorized level of flood risk management. The ETL mandates

USACE will maintain a "vegetation free zone" (VFZ), a three dimensional zone surrounding all levees, floodwalls, embankment dams, and critical appurtenant structures in all flood damage reduction systems (shown below). The purpose of the vegetation-free-zone is to provide a reliable corridor of access to and along federally authorized and constructed flood risk management features for surveillance, inspection, maintenance, monitoring, and flood-fighting.

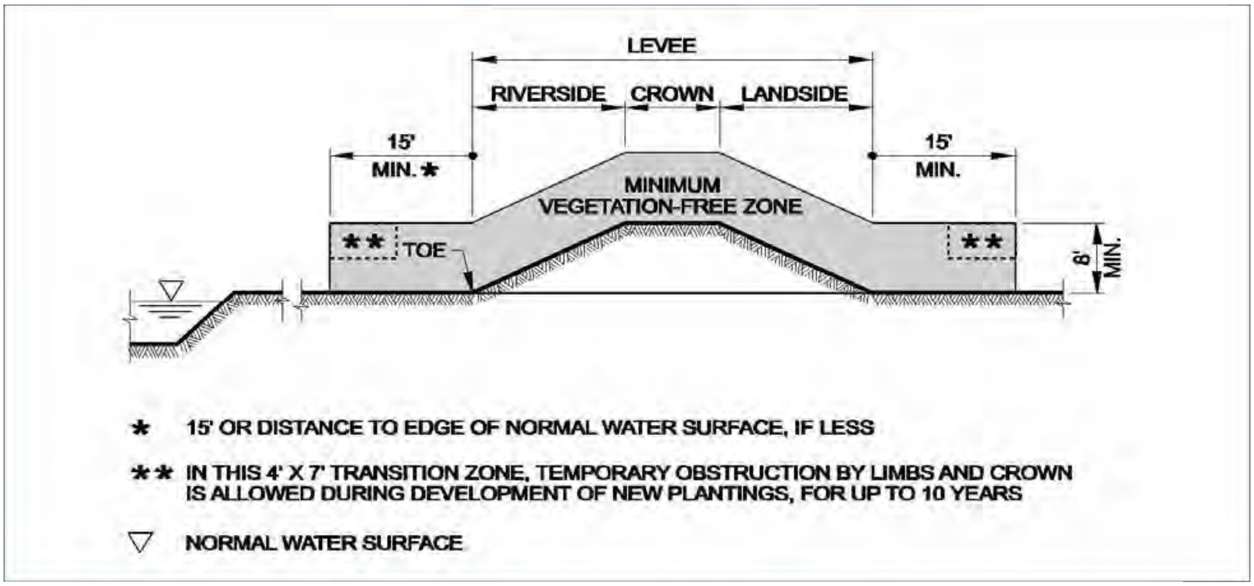


Illustration from Chapter 6, Engineering Technical Letter 1110-2-571, dated 10 April 2009

Existing Project OMRR&R Requirements.

The following description of existing OMRR&R was excerpted from Reference b.

The Sutter study area is fully contained with the Sacramento River Flood Control Project (SRFCP). The SRFCP OMRR&R requirements are contained within a standard manual for the entire SRFCP with supplements outlining more detailed requirements for each Unit within the project. The standard manual has been in place since 1955. In 1949, Headquarters U.S. Army Corps of Engineers approved a deviation from the vegetation standards in effect at the time. The sponsor is required to comply with the following vegetation standards contained within the standard manual for the SRFCP. "The Superintendent shall provide at all times such maintenance as may be required to insure serviceability of the structure at the time of floods. Measures shall be taken to exterminate burrowing animals and provide for clearing of brush, trees, and other wild growth from the levee crown and slopes. Brush and small trees may be retained on the waterward slope where desirable for the prevention of erosion and wave wash. Where practicable, measures shall be taken to retard bank erosion by the planting of willows or other suitable growths on areas riverward of the levees." In addition, each supplement applicable to the Sutter study area may have more detailed vegetation maintenance standards. Project sponsors for completed levee projects are responsible for OMRR&R of the completed project as provided in the O&M manuals. Issuance of the ETL setting forth new or clarified vegetation and access standards for levee projects does not alter the project O&M requirements and responsibilities. Inspections are conducted to document whether the sponsor is fulfilling their OMRR&R obligations as outlined in the O&M manual and inspection checklist and to determine if continued PL 84-99 rehabilitation assistance is warranted based on the outcome of those inspections. Sponsors receiving an unacceptable inspection rating may choose to pursue a conditional extension to PL 84-99 rehabilitation program eligibility by using a System Wide Improvement Framework (SWIF) as outlined in 29 November 2011 policy subject: Policy for Development and Implementation of System-Wide Improvement Frameworks.

Periodic Inspection Report.

In January 2010, an Inspection of Completed Works (ICW) periodic inspection (PI) (Reference c) was completed of the project levees surrounding the study area (Cherokee canal was not included as is addressed in a separate report). The PI evaluated nine segments and rated seven levee segments as unacceptable and two segments as minimally acceptable due to a number of deficiencies, including vegetative growth. Figure 1 identifies the segments and responsible maintaining entities. The PI report stated that "unwanted vegetative growth" along with other issues "would not prevent the system from performing as intended during the next flood event."

4. EXISTING NONCOMPLIANT VEGETATION

Identification of noncompliant vegetation was determined based on two sources:

- 2010 Periodic Inspection Report
- Vegetation Surveys conducted in 2012

2010 Periodic Inspection (PI) Report.

The PI conducted in 2010, collected GIS-based data on noncompliant vegetation (Reference c). The PI team gathered and recorded data electronically in the field using a GPS enabled tablet PC and GIS software. Vegetation points and lines represent incidents of non-compliance with ETL 1110-2-571. Separate tables were created for the Sutter Bypass, Wadsworth Canal, and Feather River West levees (Reference d). The tables present the data organized by levee mile for the Sutter Bypass and Wadsworth Canal levees, and by SBFCA sub reaches for the Feather River levees. Figure 2 shows the reaches of the Feather River West Levee, and the reaches of the Sutter Bypass and Wadsworth Canal. The non-compliant vegetation (as defined by ETL 1110-2-571) was categorized as being located on the upper third, middle third or lower third of the waterside slope, the landside slope, the crest, the waterside 15-ft easement or landside 15-ft easement. The purpose of the categories was to breakdown the incidents by degree of criticality.

The violations that were observed were either stands of vegetation or single trees or shrubs. Other reports prepared by ICF for SBFCA's Feather River West Levee Project (FRWLP) were also reviewed to further confirm the extent of vegetation.

The PI evaluation of vegetation assumes that at least a 15-foot ROW exists at the levee waterside and landside toe for levee maintenance and access. In some areas, the preexisting real estate interest may be less than 15 feet. The existing operations and maintenance manual permits brush and small trees to be present.

Existing vegetation is summarized below for the Feather River West Levee, Sutter Bypass East Levee, and Wadsworth Canal South Levee. Noncompliant vegetation is most abundant along the Feather River West Levee. Table 1 lists the significant vegetation identified based on the maintenance entities shown in Figure 1.

The following is a summary of vegetation concerns identified from the PI (Reference c).

West Feather River Levee

The West Feather River levee has 95 and 93 vegetation incidents located on the landside and waterside easements, respectively; and 79 vegetation incidents on the levee crest and slopes. Of the 79 incidents on the levee itself, 41 of them are at critical locations on the crest, landside slope and upper third of the waterside slopes. These critical incidents are concentrated in Levee District (LD) 1 of Sutter County, LD 9 of Sutter County, and State Maintenance Area (MA) 7/Hamilton Bend. These areas of greater critical vegetation impact are located within areas of significant (high percentage of reach) seepage mitigation, where cutoff wall construction is likely.

As shown in Figure 3, all 41 reaches, except one (reach 6), of the Feather River west levee, had at least one incident of noncompliant vegetation found within the VFZ. The number of incidents varied considerably between reaches; reach 16 had the most with 25 incidents, and 19 of the 41 reaches (46%) had less than 2 incidents.

Sutter Bypass Levee

For the Sutter bypass, the results are shown in Figure 4. Compared to the Feather River West Levee there was significantly less vegetation and of smaller size (primarily willows) on or near the levees. Sutter Bypass, with 82 vegetation incidents, 61 of them located on the waterside easement and 21 located on the landside easement and spread out over 18 miles, does not have a significant vegetation compliance problem with respect to ETL 1110-2-571.

Wadsworth Canal

Wadsworth Canal, with three vegetation incidents, all located on the waterside or landside easement and spread over 6 miles, does not have a significant vegetation compliance problem with respect to ETL 1110-2-571.

Vegetation Surveys.

To assess impacts to vegetation from alternatives in the final array, vegetation surveys were conducted in 2012 to identify the location of vegetation on and near the existing levees. The data was collected using GIS. The surveys identified approximately 7,600 trees, including riparian trees, orchards, and nonnative or ornamental trees, in the biological study area (200-foot corridor along the levee alignment).

5. ENVIRONMENTAL REQUIREMENTS

In accordance with Council on Environmental Quality (CEQ) Regulations for Implementing NEPA (40 CFR 1500-1508), and with Appendix C, paragraph C-3 of ER 1105-2-100, "Policy and Planning Guidance for Conducting Civil Works Planning Studies (Planning Guidance Notebook)", the planning of Corps projects must ensure that project-related adverse environmental impacts (i.e., impacts to fish and wildlife resources) have been avoided or minimized to the extent practicable, and that remaining unavoidable significant adverse impacts are compensated to the extent justified. Corps regulations stipulate that the Recommended Plan must contain sufficient mitigation measures to ensure that the plan selected will have no more than negligible net adverse impacts on fish and wildlife resources. Furthermore, a Cost Effectiveness Incremental Cost Analysis must be performed to identify the most cost-effective mitigation plan.

As part of the feasibility study, mitigation costs for vegetation impacts will have to be determined and a mitigation plan developed using incremental cost analysis and the recommendations of the USFWS and other resource agencies. Under WRDA 2007, Section 2036(a) , the Corps must fully develop a mitigation plan that includes the following: 1) monitoring until successful, 2) criteria for determining ecological success, 3) a description of available lands for mitigation and the basis for the determination of availability, 4) the development of contingency plans (i.e., adaptive management), 5) identification of the entity responsible for monitoring; and 6) establishing a consultation process with appropriate Federal and State agencies in determining the success of mitigation.

Significant vegetation losses that justify mitigation would either be mitigated through a commercial mitigation bank and/or a mitigation site acquired in the project vicinity. Mitigation site(s) selected would be located waterward of the project levees along the Feather River on lands currently in agricultural production and that could be restored to riparian habitat. Mitigation plans would be designed to avoid a net change in stage discharge relationships.

In regards to tree protection ordinances or policies that could affect construction, Sutter County does not currently have any specific ordinances or programs (heritage tree program) that protect native oaks in the County. The City of Yuba City protects trees through implementation of general plan policies, and with specific ordinances for street trees. General Plan Policy 8.4-G-3 states that heritage oaks would be preserved and enhanced in the City. Policy 8.4-I-2 requires preservation of oak trees and other native trees that are of significant size, by requiring site designs to incorporate these trees to the maximum extent feasible (Reference e). Butte County also has no specific ordinances or programs and no general plan policies concerning tree protection.

6. DISCUSSION

ETL variance guidance (Reference c) states “New federally authorized cost shared levee projects shall be designed to meet the current vegetation management standards.” It is expected that any potential levee project will be required to meet ETL 1110-2-571 requirements for congressional authorization and appropriation.

ETL Variance Procedure.

Corps guidance identifies the situations in which a Corps district may submit a vegetation variance request. Corps guidance defines a variance “as alternative vegetation management standards to be applied to a levee system or portion thereof that provide for the same levee functionality as intended in ETL 1110–2–571” (Reference c). Variances may only be granted to allow the preservation of waterside vegetation below the upper third of the waterside slope. Per the draft variance request procedure, no variance requests will be approved for noncompliant landside vegetation: “To ensure the ability to implement floodfighting activities, such as placement of sandbags or other temporary floodfight measures near the waterside crown, and to see areas of distress on the landside during a flood event, typically the upper third of the waterside slope, the crown, the landside slope, and within 15 feet of the landside toe (subject to preexisting real estate interest) of the levee needs to remain vegetation free, as defined in ETL 1110–2–571” (paragraph 9(d)).

During construction, existing vegetation would be removed adjacent to the riverward and landside toes by root plowing or clearing and grubbing to create the vegetation free zone. Since the landward side of the levee is currently maintained as an access road, very little woody vegetation exists. Following construction, disturbed soils including levee side slopes will be seeded with native grass seed to prevent wind and water erosion. A 15-foot-wide vegetation management zone along the riverward and landside toe of the levee will be permanently maintained to be devoid of trees and shrubs.

A variance request may be applicable because the subject levees are existing federally authorized levees in which the existing O&M manual allows vegetation within the VFZ. The existing O&M manual for the SRFCP states that "Brush and small trees may be retained on the waterward slope where desirable for the prevention of erosion and wave wash. Where practicable, measures shall be taken to retard bank erosion by the planting of willows or other suitable growths on areas riverward of the levees."

Also per the variance guidance (Reference c), a vegetation variance can be considered if one of the following applies:

a. Comply with applicable law concerning the environment, cultural or historic preservation;
b. Protect the right of Tribal Nations, pursuant to treaty, statute, or Executive Order;
c. Address a unique environmental consideration; and/or
d. Prior vegetation agreement in place.

Criteria (a), (c), and (d) may be an appropriate basis to consider a variance to retain vegetation. However, the guidance states that "even if one of the above criteria is met, life safety is still paramount and the vegetation variance must assure that the structural integrity and functionality of the levee are retained." The levee must still be accessible for maintenance, periodic inspection, monitoring during flood events, and access to perform flood-fighting if required. Because the application process, analysis and review time will be both lengthy and costly, a variance request should be considered in the preconstruction engineering and design phase.

6. ETL COMPLIANCE SCENARIOS

For the evaluation of initial alternatives, two scenarios or strategies were identified to address the ETL compliance issue for fix in place levee alternatives. The two scenarios provide for the full disclosure of environmental effects to vegetation from the extreme of: (1) removal of all (landside and waterside) levee vegetation, to the other extreme of (2) vegetation that is within the construction footprint that would be impacted by levee degrading and construction disturbance to construct the cutoff walls and/or seepage berms. The scale of impacts, the mitigation and variance costs, and the risk to the study was evaluated under each scenario for each alternative. The project alternatives are shown in Figures 5-11.

Scenario: Establish VFZ Per ETL. All vegetation, other than perennial grasses, would be removed from the levee slopes and out 15 feet from the waterside and landside levee toes (subject to preexisting real estate interest). Vegetation within the direct construction footprint will be removed.

a) Mitigation costs were estimated based on costs developed by SBFCA for the Feather River from Thermalito Afterbay to Star Bend (Reference h). The number of incidents for each alternative was compared to the SBFCA estimates, and used to develop an estimate of mitigation costs based on a ratio of incidents identified in the PI. The estimate assumes riparian habitat is mitigated at a ratio of 2 acres for each acre affected, using a mitigation bank cost of $100,000 to 140,000 per acre.

b) The study risk ranges from low to moderate risk depending on the amount of vegetation removal under each alternative. The associated risks are that a jeopardy opinion may be issued and that resource agency and public concerns could increase project costs, or delay compliance with applicable environmental laws. However, these risks could be substantially reduced by development of mitigation plan with the resource agencies to compensate for the loss of vegetation.

Scenario: Variance Per ETL. Corps approval of a variance would be obtained either as part of the pilot study or during the Preconstruction-Engineering and Design (PED) phase. The variance would retain all vegetation outside the immediate construction footprint located on the lower two thirds of the waterside levee slope and out 15 feet from the waterside levee toe; all other levee vegetation would still be removed in accordance with Corps policy.

a) Substantial engineering analysis would be required to support a variance request and to determine what reaches would likely receive variance approval during PED. This analysis would at a minimum entail identification of the tree species, size, root ball size, scour potential if trees fall, increase in seepage gradient, reduction in stability factor of safety, and conclusions regarding rationale for variance approval.

b) Costs to conduct the engineering analyses to support a waterside variance are based on an assumed range of $10,000 to $30,000 per incident. Recent engineering analysis costs have varied significantly from $6,334 per site (PL84-99) to $162,500 per site (Natomas 408 ($1.3 million to evaluate 8 areas)).

c) The study risk ranges from low to moderate depending on the alternative. Alternatives with the most waterside vegetation retained by a variance would have the highest risk if a variance was not granted.

The two scenarios were applied to each of the study alternatives. The estimated costs and study risk are shown in Tables 2 and 3. As shown in Table 2 and 3, the assumed cost to obtain a variance for the alternatives ranges between about $220,000 (SB-3, Ring Levee) to $5.8 million (SB-6, Sutter Bypass) while the assumed mitigation cost of a VFZ is between $1.8 (SB-3 Ring Levee) to $20.9 million (SB-6, Sutter Bypass). The noncompliant vegetation located on the waterside of the levee could be retained if a variance was granted.

7. FINAL ARRAY OF ALTERNATIVES

The planning study evaluated the initial study alternatives based on FRM benefits and costs and determined a final array of alternatives. Alternatives SB-7 and SB-7 were carried forward to the final array. The GIS-based vegetation land cover survey data was then used to more accurately determine vegetation impacts and mitigation costs. Tables 4, 5, and 6 show the results for SB-7 and SB-8. Total mitigation costs are $4.32 million for SB-7, and $9.01 million for SB-8 (Table 6). SB-7 would impact 25.07 acres of riparian forest, 1.30 acres of oak woodland and cause the loss of 129 elderberry shrubs. SB-8 would impact 41.68 acres of riparian forest and scrub shrub, 6.7 acres of oak woodland, and cause the loss of 217 elderberry shrubs. Of the total estimated costs, VFZ requirements account for $1.73 million (40%) for SB-7 and $4.50 million (50%) for SB-8.

8. RECOMMENDED APPROACH

The following recommended approach follows the recommendations provided in Reference b.

a. Future without project condition will assume that project sponsors have maintained the levee in accordance with the O&M manual or that deficiencies in maintaining the levee to the requirements of the O&M manual are being or will be addressed in an approved SWIF(As of this writing, the sponsor has submitted a SWIF draft Letter of Intent).

b. With-project condition will include work, including required mitigation, necessary to comply with access and vegetation requirements of the ETL, including the full 15-foot access (O&M) corridor/vegetation free easement extending from the toe of the levee. Removal or an approved vegetation variance will be considered as being in compliance with the ETL. However, a vegetation variance will not be issued until the design phase of the project. For the feasibility study a vegetation variance will generally not be assumed.

c. For levee segments where a new levee alignment is being proposed (e.g. setback levee), the project as formulated will be in compliance with the ETL (vegetation and access) and all costs associated with ETL compliance will be project costs subject to cost sharing .

d. For levee segments where measures to strengthen existing levees are fix- in-place recommendations (e.g. slurry wall, berms) the following will apply:

    i. If local maintenance has been in compliance with the O&M manual (no deferred maintenance) then all costs associated with ETL compliance, including access of additional real estate interests as may be necessary, will be project costs subject to cost sharing.

ii. If local maintenance has not been in compliance with the O&M manual, project sponsors are responsible for achieving compliance with the O&M manual and required costs to do so are non-federal, non-project costs (Reference g). If a SWIF has been approved, the deferred maintenance may be accomplished in accordance with the terms of the SWIF except that deferred maintenance within the footprint of the new construction must be completed by the sponsors before construction or during construction by the government contractor.

e. During feasibility investigations sufficient information may not be available to distinguish between deferred maintenance costs and new costs to achieve ETL compliance; therefore for the feasibility effort all vegetation clearing and associated mitigation within the construction footprint will be assumed to be a project costs subject to cost sharing.

Matt Davis
Environmental Resources Branch
Sacramento District
U.S. Army Corps of Engineers

FEATHER RIVER – SUTTER BASIN PROTECTION AREA
PERIODIC INSPECTION – JANUARY 2010
REPORT NO. 1

Report prepared by:
GENTERRA/Hultgren-Tillis/Taber Consultants, LLC
GHT2

US Army Corps of Engineers®
Sacramento District

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Figure 1. Segments and Responsible Maintaining Entities

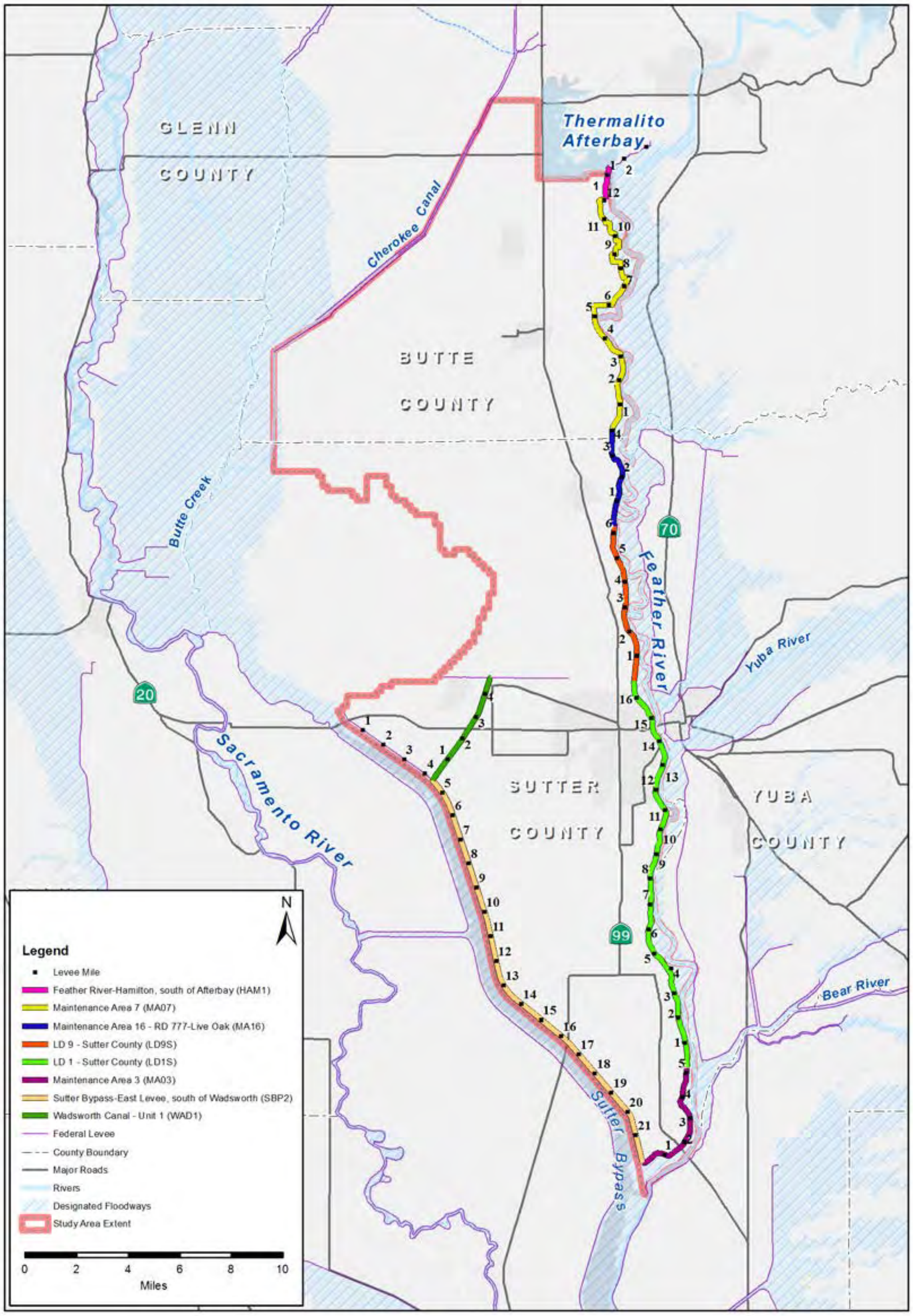

Figure 2. Feather River, Sutter Bypass, and Wadsworth Canal

Table 1. Sutter ETL Vegetation Inventory Summary

Feather River Afterbay
13 sites out of 18 with veg potentially not conforming to ETL
4 sites with large trees on crest (24”-30”)

LD1
36 sites out of 64 with potential ETL issues
1 site with 48" walnut in the landside toe
1 site with 72" cottonwood at WS toe
2 sites with 48" trees at the LS toe
6 sites with numerous unknown (30" & 48") at WS slope and 15' easement
8 sites with oaks (30" & 48)" at WS slope and 15' easement

LD9
5 sites out of 19 with potential ETL issues
1 site with 48" walnut in the landside toe
1 site with unknown 72"+ at WS slope and 15' easement
1 site with oaks 30" at WS slope and 15' easement

MA 3
9 sites out of 31 with potential ETL issues
3 sites with 30"-48"/7 total at WS slope and 15' easement
1 site with unknown 36" on LS easement over seepage berm
1 site with unknown 24" on LS toe

MA7
27 sites out of 76 with potential ETL issues
10 sites with 30"-60" (numerous unknown) at WS slope and 15' easement
14 sites with 30"-60" (numerous unknown) at LS easement

MA 16
14 sites out of 29 with potential ETL issues
4 sites with 30"-48" (numerous unknown) at WS slope and 15' easement
3 site with 30"-60" (numerous unknown) at LS easement

Sutter Bypass
21 sites out of 79 with potential ETL issues
26 sites with 24"-48" (numerous unknown) at WS slope and 15' easement
8 site with 24" (numerous unknown) at LS easement

Wadsworth Canal
2 sites out of 3 with potential ETL issues

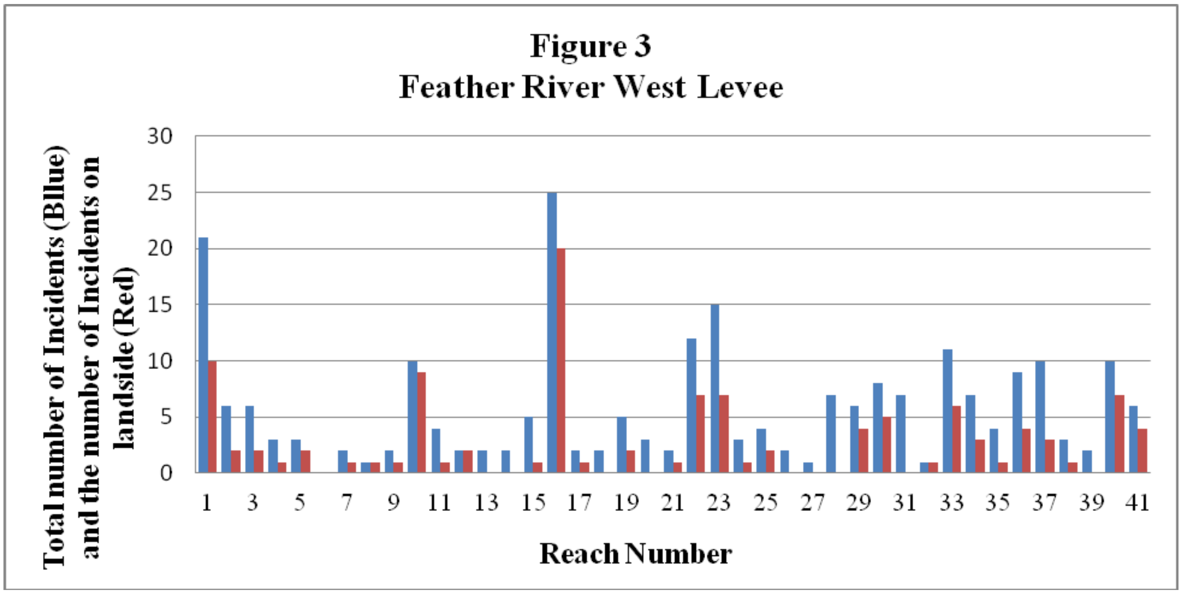
Figure 3
Feather River West Levee
Total number of Incidents (Bllue) and the number of Incidents on landside (Red)
Reach Number

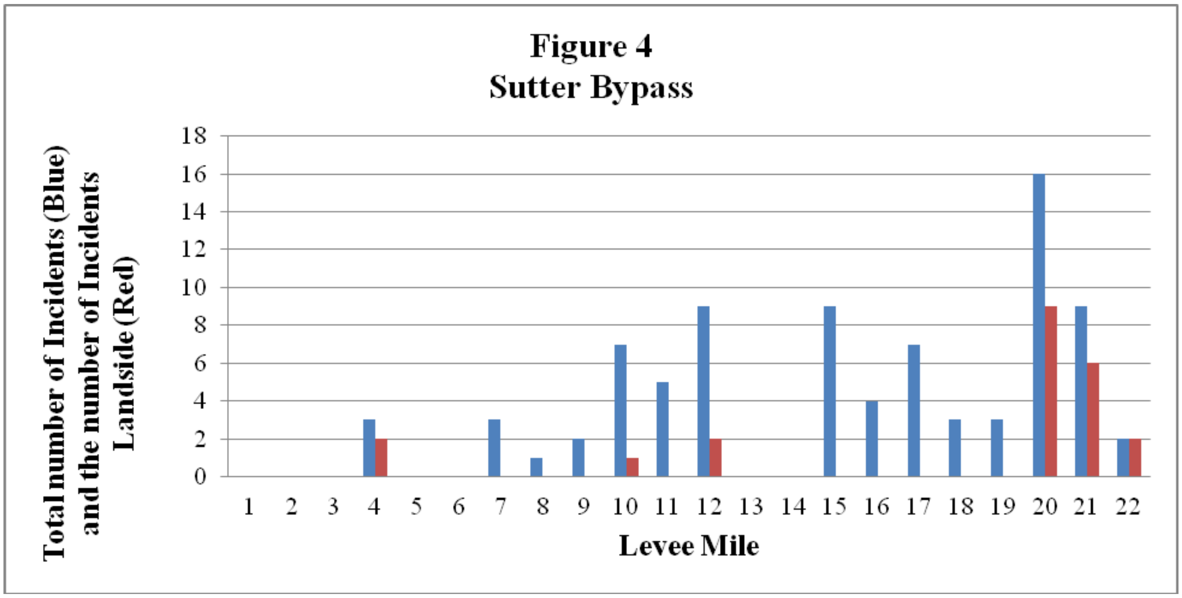
Figure 4
Sutter Bypass
Total number of Incidents (Blue) and the number of Incidents Landside (Red)
Levee Mile

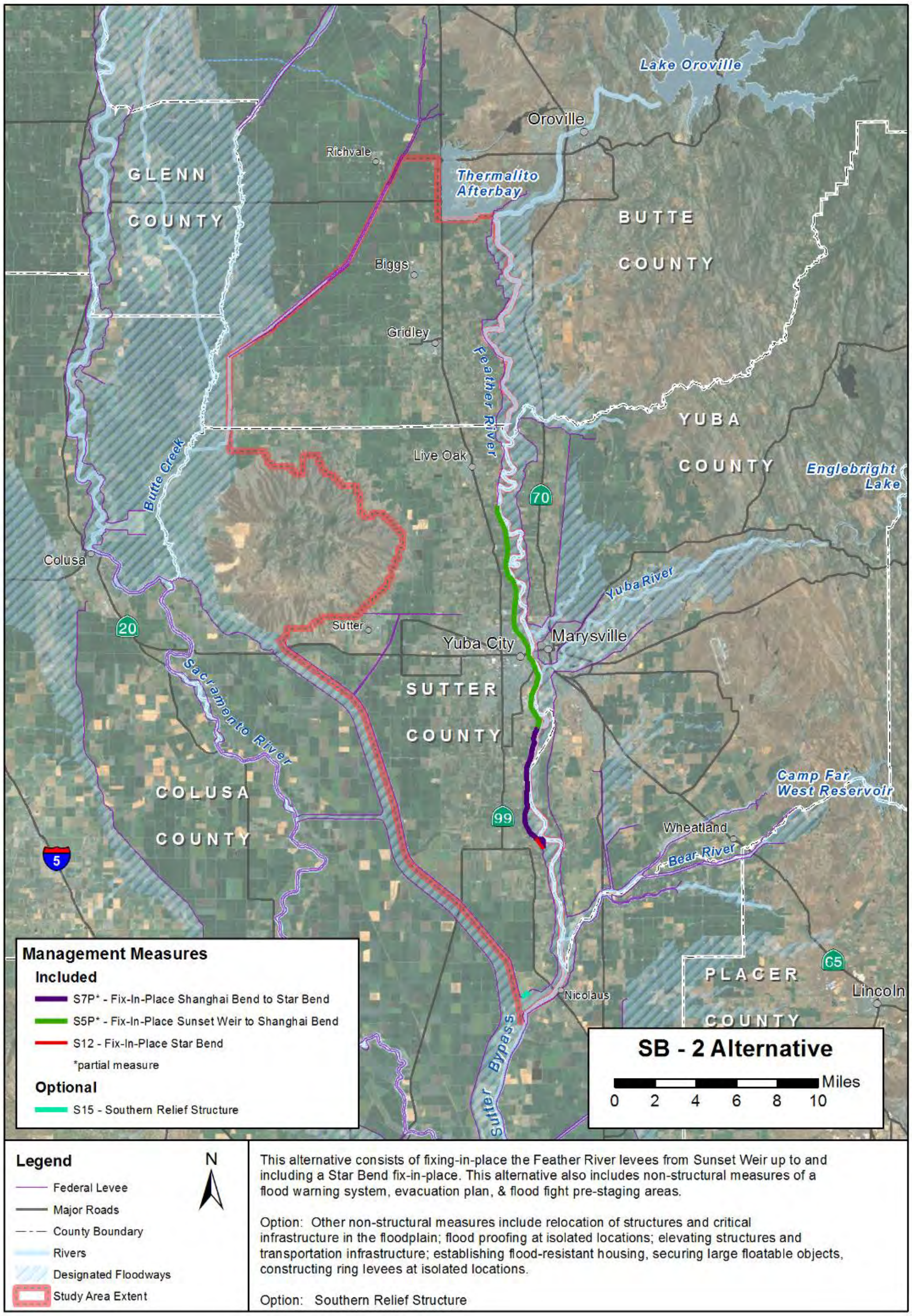

Figure 5. SB – 2 Alternative

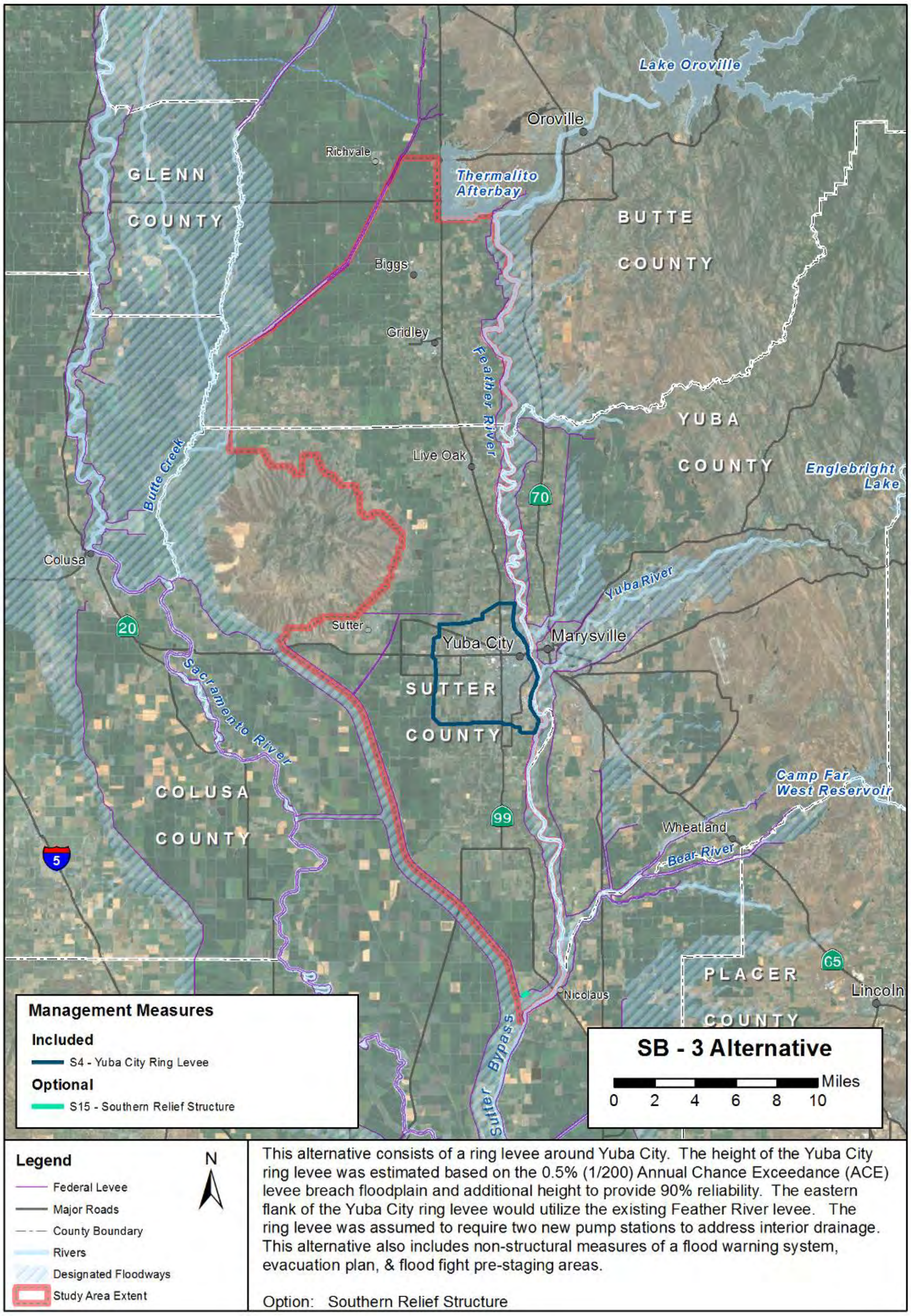

Figure 6. SB – 3 Alternative

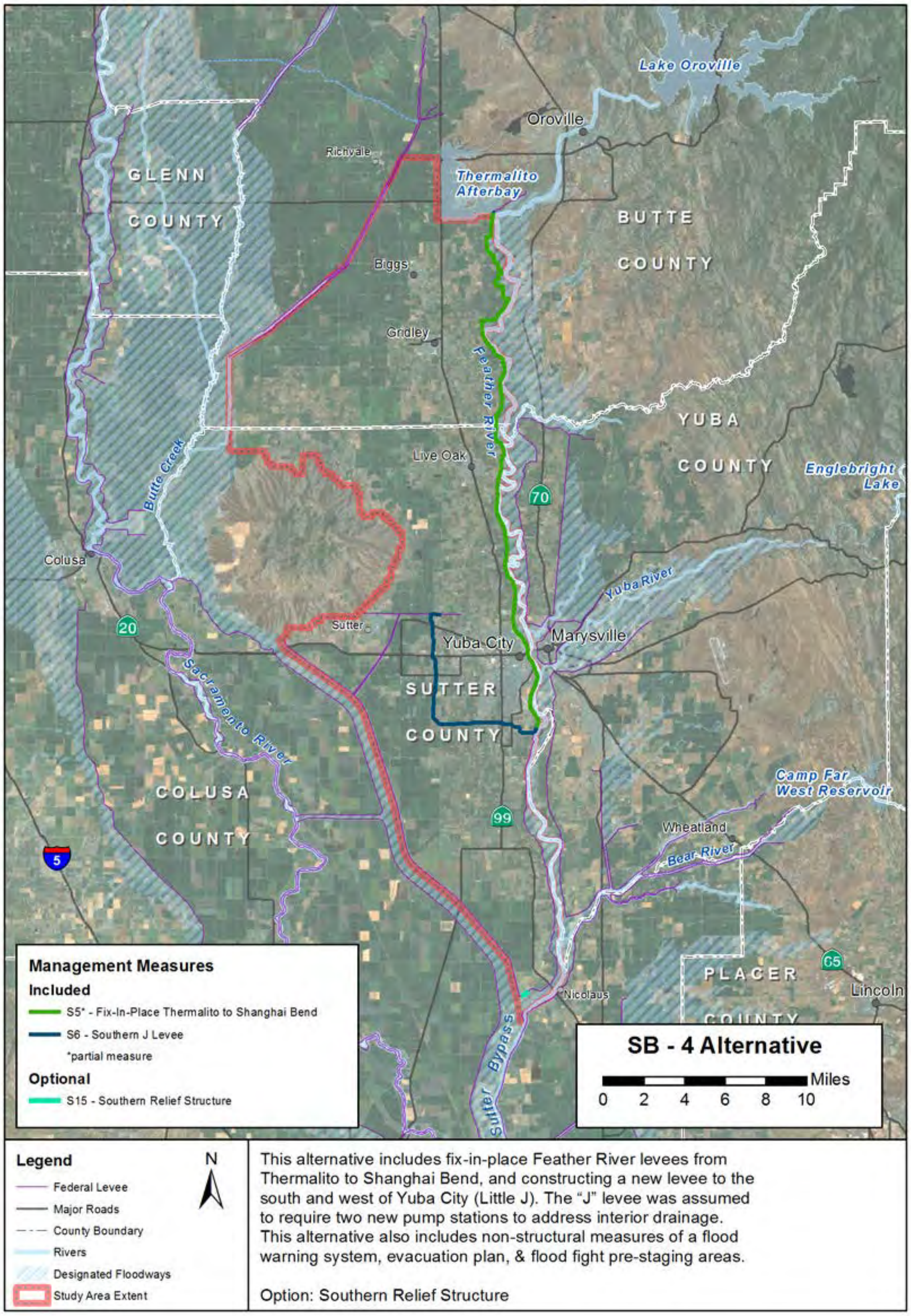

Figure 7. SB – 4 Alternative

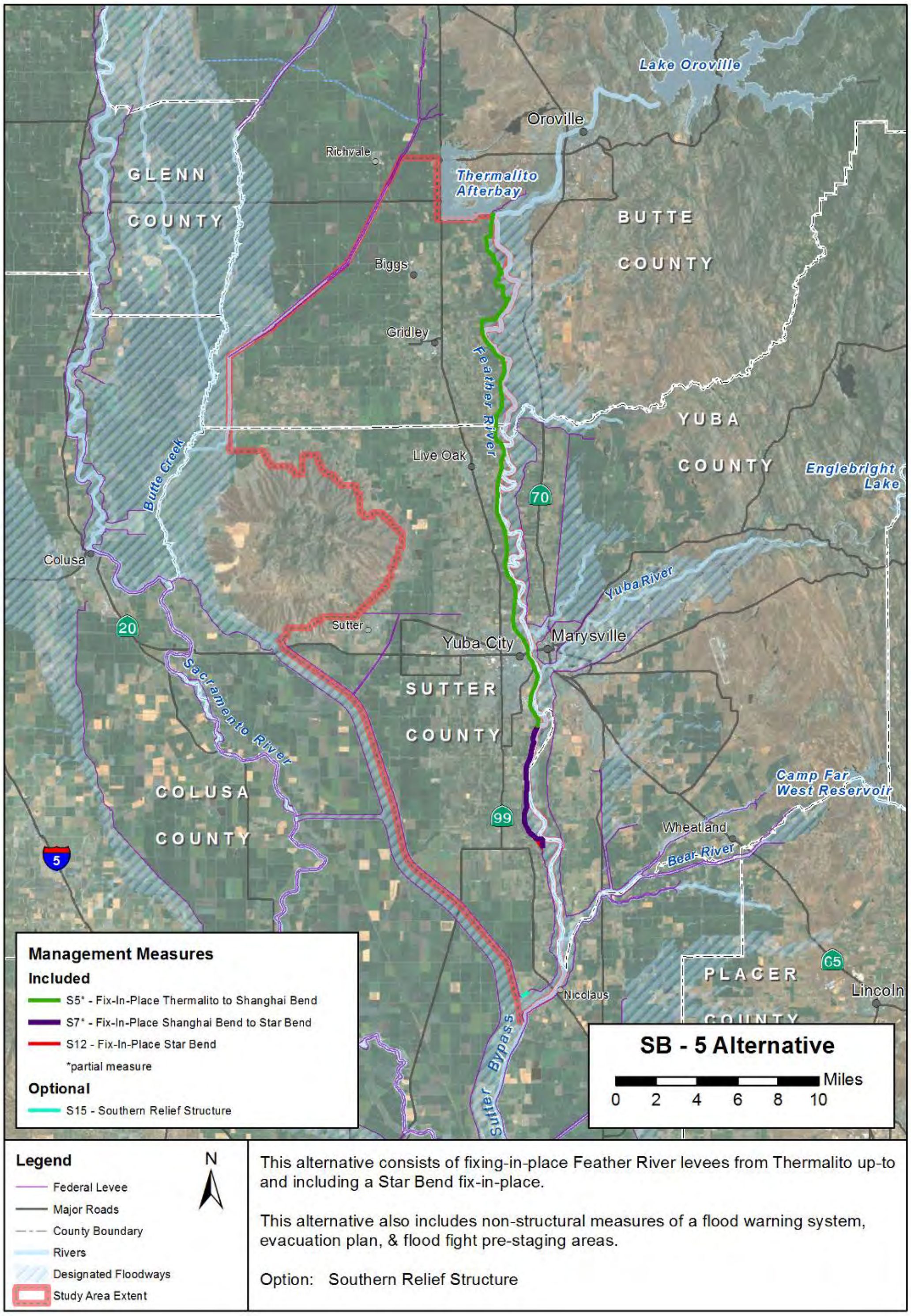

Figure 8. SB 5 Alternative

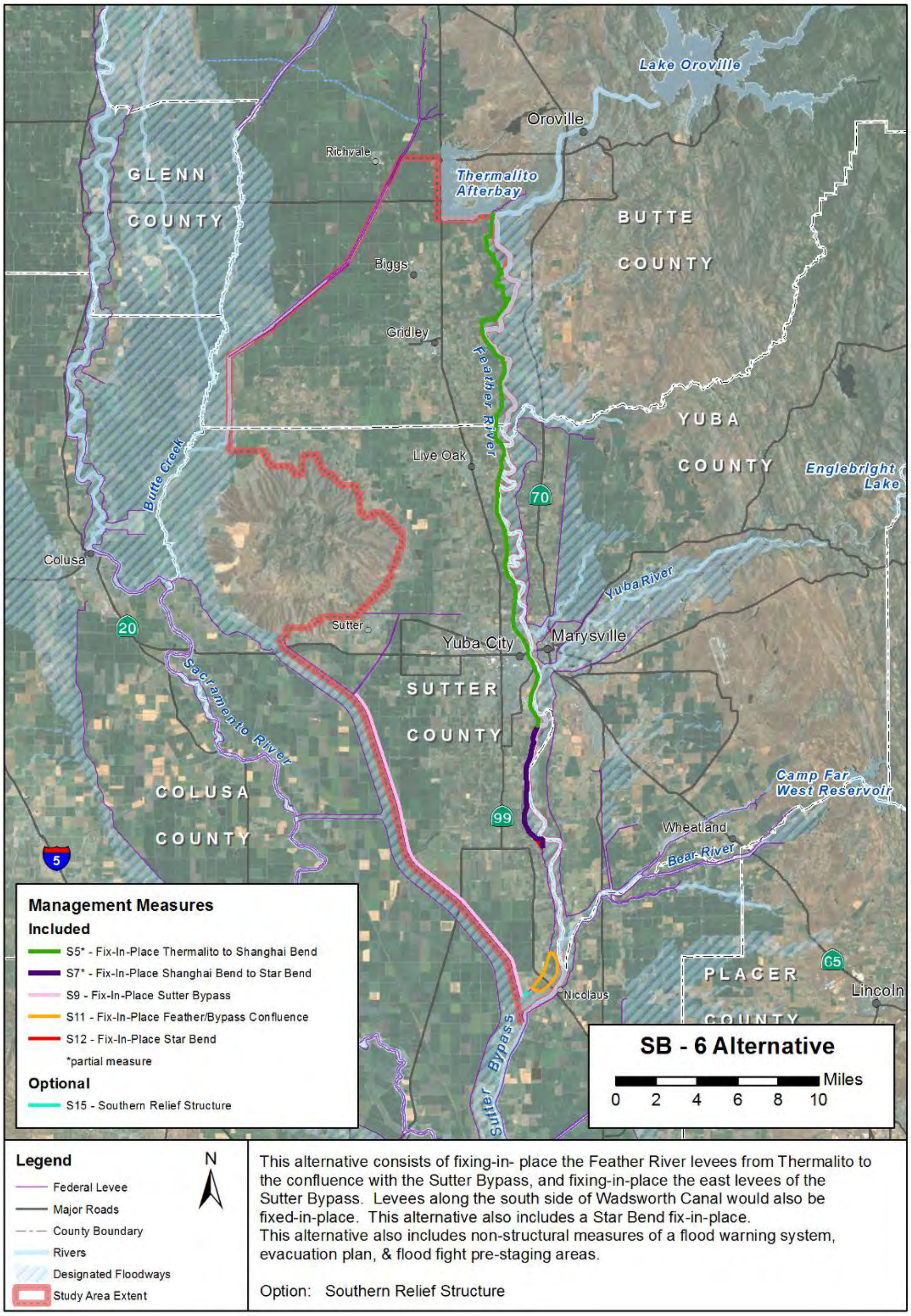

Figure 9. SB – 6 Alternative

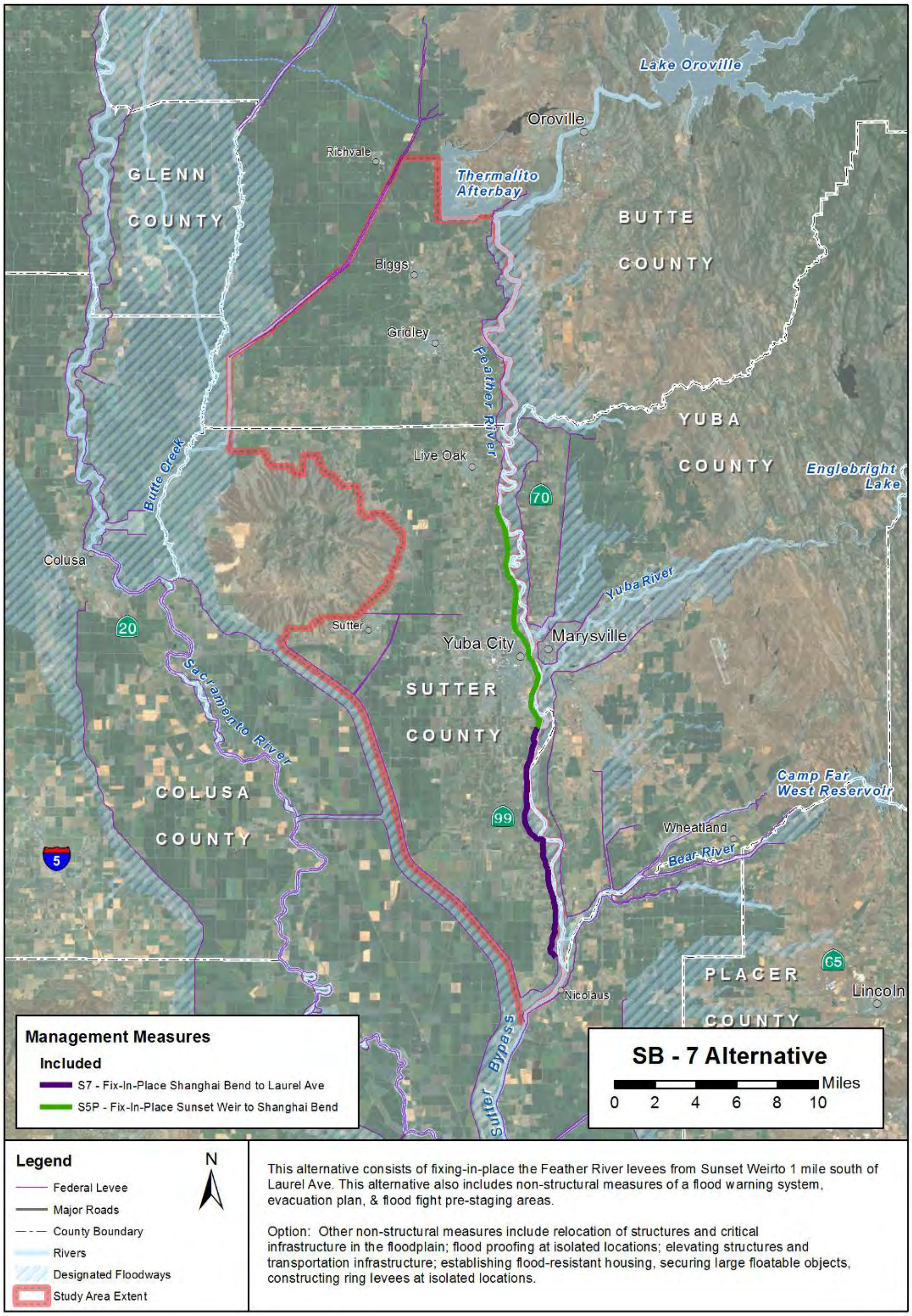

Figure 10. SB-7 Alternative

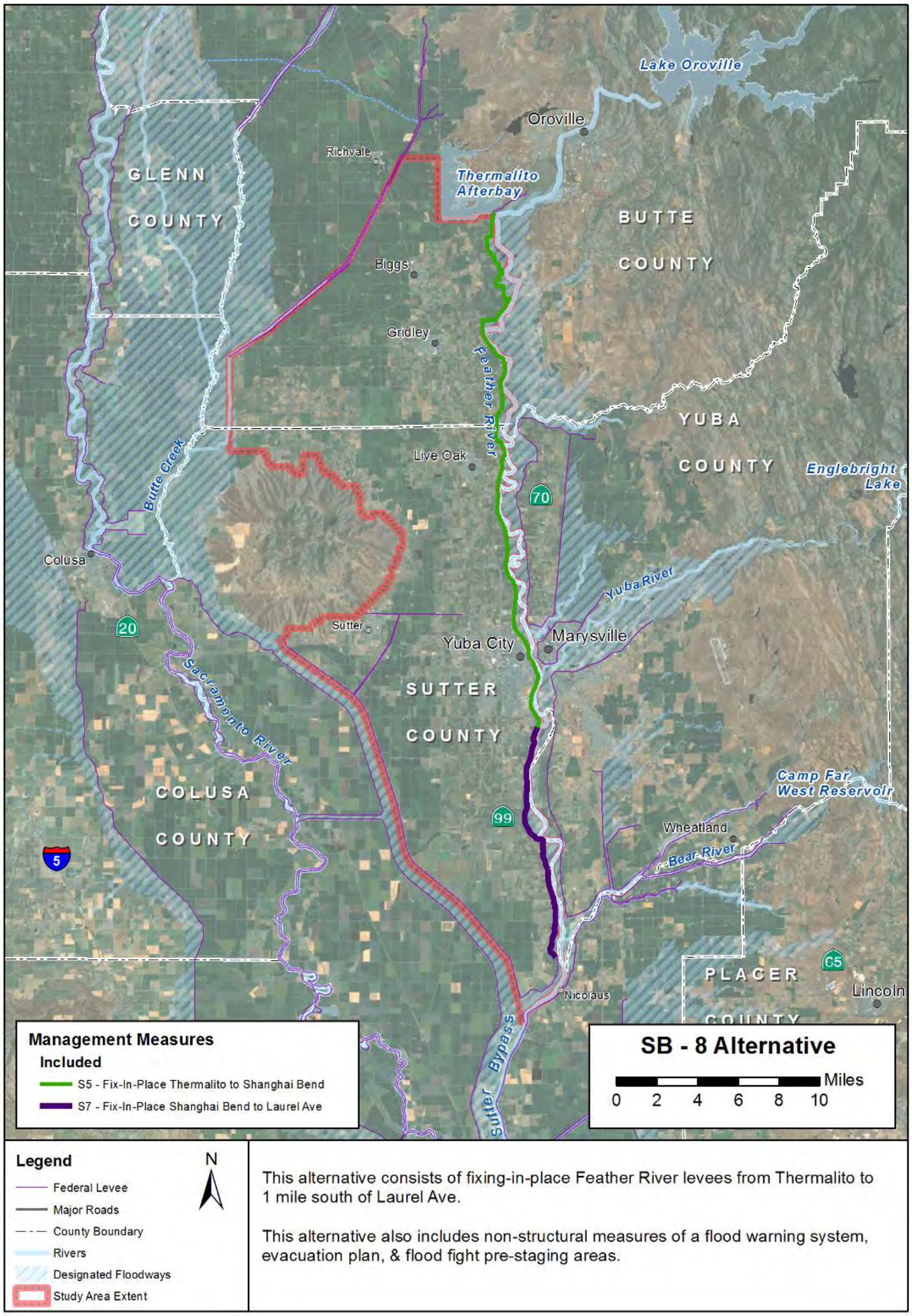

Figure 11. SB-8 Alternative

Table 2
Scenario: Establish VFZ Per ETL

Alternative	Number of Incidents	Mitigation Cost	Study Risk
SB-2, Minimal-Fix-in-Place (Feather reaches 6-21)	71	$3,301,911 to $4,627,675	Low-Moderate. Extent of vegetation removal is less than SB-5 and SB-6 but more than SB-3 and SB-4. Lower risk for jeopardy opinion and resource agency and public concerns.
SB-3, Ring Levee (Feather reaches 12-18)	38	$1,767,220 to $2,474,108	Low. The least amount of vegetation removal compared to other alternatives and therefore the least potential for a jeopardy opinion and project delays.
SB-4, Little J (Feather reaches 12-41)	179	$8,324,536 to $11,654,350	Moderate. Same as SB-5.
SB-5, Fix-in-Place, Thermalito to Star Bend (Feather reaches 6-41)	200	$9,301,158 to $13,021,621	Moderate. Substantial vegetation removal. Along with SB-6, most potential for a jeopardy opinion and permit delays. Compensatory mitigation would substantially reduce identified risk.
SB-6, Sutter Bypass (Feather reaches 1-41, Sutter Bypass LM 0-22, Wadsworth Canal LM 0-3)	321	$14,928,359 to $20,899,702	Moderate. Most vegetation loss and the most potential risk due to greater risk of a jeopardy opinion and permit delays. Compensatory mitigation would substantially reduce identified risk.
SB-7, Fix-In-Place, Feather River, Sunset Weir to south of Laurel Ave (Feather reaches 2-21)	87	$4,046,003 to $5,670,531	Low-Moderate. Same as SB-2.
SB-8, Fix-In-Place, Feather River, Thermalito to south of Laurel Ave (Feather reaches 2-41)	216	$10,045,250 to $14,078,560	Moderate. Similar as SB-5.

Table 3
Scenario: Variance Per ETL

Alternative	Number of Waterside Incidents	Cost to Obtain Variance	Study Risk
SB-2, Minimal Fix-in-Place (Feather reaches 6-21)	38	$380,000 to $1,140,000	Low. Additional ESA consultation and NEPA compliance could be required if a variance is not granted. Mitigation costs would increase but not significantly affect total project costs.
SB-3, Ring Levee (Feather reaches 12-18)	22	$220,000 to $660,000	Low. Similar to SB-2. Least amount of vegetation removal compared to other alternatives and therefore the least potential impact from not obtaining a variance.
SB-4, Little J (Feather reaches 12-41)	103	$1,030,000 to $3,090,000	Moderate. Same as SB-5.
SB-5, Fix-in-Place, Thermalito to Star Bend (Feather reaches 6-41)	111	$1,110,000 to $3,330,000	Moderate. In addition to the risks for other alternatives, this and alternative SB-6 have the most vegetation removal and thus the most potential for a jeopardy opinion and project delays if a variance is not obtained.
SB-6, Sutter Bypass (Feather reaches 1-41, Sutter Bypass LM 0-22, Wadsworth Canal LM 0-3)	193	$1,930,000 to $5,790,000	Moderate. Most vegetation loss and therefore the most potential risk to study if a variance is not granted.
SB-7, Fix-In-Place, Feather River, Sunset Weir to south of Laurel Ave (Feather reaches 2-21)	46	$460,000 to $1,380,000	Low. Same as SB-2.
SB-8, Fix-In-Place, Feather River, Thermalito to south of Laurel Ave (Feather reaches 2-41)	119	$1,190,000 to $3,570,000	Moderate. Same as SB-5.

Table 4. Effects on Land Cover Types for Alternatives SB-7 and SB-8

Land Cover Types	Alt SB-7 (acres)	Alt SB-8 (acres)
Riparian forest		
Construction footprint	14.16	20.63
VFZ Requirement	9.61	14.00
Subtotal	23.77	34.63
Riparian scrub-shrub		
Construction footprint	NA	0.32
VFZ Requirement	NA	NA
Subtotal	NA	0.32
Oak woodland		
Construction footprint	NA	NA
VFZ Requirement	1.30	6.73
Subtotal	1.30	6.73
Total	25.07	41.68

Table 5. Elderberry Shrub Impacts for Alternatives SB-7 and SB-8

Land Cover Types	Alt SB-7 (number of shrubs)	Alt SB-8 (number of shrubs)
Elderberry Shrub Loss		
Construction footprint	31	52
VFZ Requirement	98	165
Total	129	217

Table 6. Mitigation Cost Estimate for Alternatives SB-7 and SB-8

Land Cover Types	Alt SB-7 (X $1 Million)	Alt SB-8 (X $1 Million)
Riparian forest		
Construction footprint	2.83	4.13
VFZ Requirement	1.92	2.80
Subtotal	4.75	6.93
Riparian scrub-shrub		
Construction footprint	NA	0.06
VFZ Requirement	NA	NA
Subtotal	NA	0.06
Oak woodland		
Construction footprint	NA	NA
VFZ Requirement	0.26	1.35
Subtotal	0.26	1.35
Elderberry Shrub Loss		
Construction footprint	0.42	0.70
VFZ Requirement	1.58	2.66
Subtotal	2.00	3.36
Construction footprint Total	3.25	4.89
VFZ Requirement Total	3.76	6.81
*Grand Total**	4.32	9.01

* Reduced to reflect VELB compensation providing riparian and oak woodland compensation.

Cost Assumptions: (1) Compensation ratio of 2:1 for all cover types
(2) Star Bend Mitigation Site is limited to 28.5 acres of available area at a cost of $125,000 per acre;
(3) Elderberry mitigation would occur at Star Bend and would compensate for riparian impacts in addition to VELB impacts;
(4) Mitigation needs not met at Star Bend would be addressed at a mitigation bank at cost of $100,000 per acre; and
(5) VELB mitigation requirements based on the USFWS conservation guidelines.

ENCLOSURE J

SBFCA EXISTING CONDITIONS SEEPAGE RESULTS

Table 4-4. Seepage Gradient Criteria
Feather River West Levee Rehabilitation Early Implementation Project - Task Order 1

Levee Condition	Location	WSE Condition	Maximum Average Exit Gradient
Existing Conditions	Landside levee toe	DWSE	0.5
		HTOL	0.6
	Bottom of empty ditch at landside toe	DWSE	0.5
		HTOL	0.6
	Bottom of ditch and landside field between toe and 150 feet from toe	DWSE	linear interpolation between 0.5 and 0.8
		HTOL	linear interpolation between 0.6 and 0.9
With Rehabilitation Measures	Landside levee toe, without berm (e.g., with cutoff wall)	DWSE + 1 foot	0.5
		HTOL + 1 foot	0.6
	Landside levee toe with relief wells	DWSE + 1 foot	0.5
		HTOL + 1 foot	0.6
	Bottom of empty ditch at landside toe (without berm)	DWSE + 1 foot	0.5
		HTOL + 1 foot	0.6
	Bottom of empty ditch ≥ 150 feet from toe, with or without berm	DWSE + 1 foot	0.8
		HTOL +1 foot	0.9
	Bottom of ditch between toe and 150 feet from toe, with or without berm	DWSE + 1 foot	linear interpolation between 0.5 and 0.8
		HTOL +1 foot	linear interpolation between 0.6 and 0.9
	Landside levee toe, with berm	DWSE + 1 foot	0.5
		HTOL +1 foot	0.6
	Toe of seepage berm, between landside toe and less than 100 feet from levee toe	DWSE + 1 foot	0.8
		HTOL + 1 foot	does not increase more than 20% from that determined for the DWSE
	Toe of seepage berm between 100 feet and 300 feet from landside levee toe	DWSE + 1 foot	0.8
		HTOL + 1 foot	use engineering judgment
	Toe of seepage berm at or greater than 300 feet from landside levee toe	DWSE + 1 foot	use engineering judgment
		HTOL +1 foot	use engineering judgment

WSE = water surface elevation
DWSE = design water surface elevation
HTOL = hydraulic top of levee

TABLE 5-1A: CHARACTERIZATION OF REACH 1
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="3">1</td><td rowspan="3">SBFCA 10+00
[DWR 2167+00]</td><td rowspan="3">SBFCA 129+66
[DWR 2286+00]</td><td>2.25</td><td>Toe
CPT – 4

Crown
CPT – 9
Borings – 6</td><td>Crown Width
• 17 to 20 feet

Landside Slope
• 1.3H:1V to flatter than 2H:1V

Waterside Slope
• 2.2H:1V to flatter than 3H:1V</td><td>• 10+93 (2167+13) 1997: Light seepage observed.
• 29+13 (2185+34) 1997: Light seepage observed.
• 35+85 to 60+50 (2192+06 to 2216+73) 1997: Severe sloughing due to wave erosion along approximately 1800 feet of waterside slope.
• 40+52 to 43+88 (2196+79 to 2200+03) 1997: Erosion along the waterside levee slope with a vertical face of 6 to 12 inches, half way up the levee.
• 48+00 to 53+21 (2204+22 to 2209+44) 1955; Levee cut to dewater the flooded area in December 1955.
• 69+94 (2226+15) 1997: An area where levee turns sharply to an east-west direction was eroded for about 150 feet.
• 73+02 (2229+26) 1986: A sinkhole approximately 30 feet long, ten feet wide, and ten feet deep right at waterside toe of the levee. This hole was discovered after the 1986 high water receded. There was no landside evidence of this sinkhole during high water.
• 98+06 (2254+30) 2006: Waterside erosion.
• Based on DWR's Seepage Area map for the Feather River West Levee area, 100% of Reach 1 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 100% of Reach 1 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.</td><td>• 10+03 to 280+77 (2166+00 to 2437+00) 1963: Levee stabilization construction (USACE Specification 2783, File Drawing 4-4-531).
• 73+02 (2229+26) 1986: Sinkhole at waterside toe filled with gravel.
• 98+06 (2254+30) 2006: Waterside erosion, emergency repair made.</td><td>• Land Side: Between Stations 10+93 and 54+13 (2167+13 and 2210+33) Geomorphologic consists of Ra with isolated fingers of Rdc. Station 54+13 (2210+33) to end of reach consist mainly of Ra with fingers of Hch, Rofc, and Rob.
• Water Side: Mainly Ra along the levee toe with meandering Rch parallel with waterside ditch.</td><td>• 10+93 to 45+80 (2167+13 to 2202+00): Low to moderate resistivity within upper 15-20', low to very low resistivity to depth
• 45+80 to 106+80 (2202+00 to 2263+00): High to very high resistivity (occasionally moderate) within upper 15-25' at 45+80 (2202+00) increasing in depth to greater than 50' between 93+80 (2250+00) and 106+80 (2263+00), possible anomalous deep high resistivity readings at depth from 68+80 (2225+00) to 178+80 (2335+00) on waterside due to bend in levee.
• 106+80 to 129+76 (2263+00 to 2286+00): Moderate resistivity (occasionally high) within upper 15-25', high to very high below, possible anomalous deep high resistivity readings from 118+80 (2275+00) to 129+80 (2286+00) due to irrigation pipe crossings.</td></tr>
<tr><th>GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th>DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th>NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]
SBFCA STATION
[DWR ULE STATION]</th><th>EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><td>• Levee Fill: Consists of variable lenses of clay, silt, sandy silt, silt sand and silty sand.
• Blanket: Relatively consistent silt, clay, sandy silt and sandy clay blanket with thicknesses ranging from 45 to 60 ft. Isolated lenses (between 1 and 10 ft. thick) of sand and silty sand are shown within the blanket area at different elevations. Below an elevation of -20 ft. MSL, the logs show alternative thin layers of GM/SP/SM/CL.</td><td>1) DWR 2262+50 (SBFCA 106+10) = 100-yr, 52.1 ft
2) Average Head = 13.7 ft</td><td>Parallel landside and waterside unlined ditches are present. Near Stations 45+80, 57+80, and 67+80 (2202+00, 2214+00, and 2224+00) unlined perpendicular ditches tie into parallel ditch. Land side ditch shifts further away from levee toe near Station 57+80 (2214+00). Landside ditches ends near Station 67+80 (2224+00). Pump Station located in landside ditch near Station 51+80 (2208+00). High voltage power pole and line located near Station 72+80 (2229+00) on waterside. Hwy 99 crosses levee at Station 97+80 (2254+00).
Sacramento Ave intersects levee road at Station 106+80 (2263+00).</td><td>SBFCA Station 106+10.50
[DWR Station 2262]</td><td>1) WM0003_012B, SM0003_004C, CM0003_004H</td><td colspan="2">Location of potential underseepage problem due to presence of deep sand layers. A ditch is also present at the landside toe, which increases underseepage potential.</td></tr>
</table>

TABLE 5-1B: ANALYSIS RESULTS FOR REACH 1
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 1
Analysis Station: 106+11

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.30 to 0.46		0.33 to 0.52	7 feet above toe	No	The high phreatic breakout point and granular levee fill indicate potential for through-seepage. Past performance records also indicate underseepage.
100-year + 3 feet	0.35 to 0.55		0.38 to 0.61	8.5 feet above toe	No	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
100-year	1.36				No	
100-year + 3 feet	1.26				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	
100-year + 1 foot	typ. winter WSE					
Rationale for Selection of Two Alternatives for Analysis						
For Alternative Analysis, two different models were created using Station 106+11 surface geometry to represent the reach. One model was generated with the existing berm and irrigation ditch and another model was created without the berm or ditch. The analyses results for the cross-section without the berm and ditch are presented, because with the berm and ditch in place the toe gradients move to the bottom of the ditch due to the close proximity of the toe and ditch. Alternative 1 for Reach 1 is a soil-bentonite cutoff wall extending below relatively shallow silty sand layers into a less permeable clay layer. Alternative 2 for Reach 1 is a drained stability berm and an undrained seepage berm. These alternatives were selected to mitigate through-seepage and underseepage.						
Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):	Soil-Bentonite Cutoff Wall.					
Dimensions of Primary Features:	**10+00 to 58+80** Cutoff Wall Tip Elevation 20 feet, **58+80 to 83+00** Cutoff Wall Tip Elevation 18 feet, **83+00 to 114+00** Cutoff Wall Tip Elevation 24 feet, **114+00 to 129+66** Cutoff Wall Tip Elevation 27 feet. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade.					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	0.31 to 0.49	N/A	0.38 to 0.62	No breakout above toe	Yes	
100-year + 4 feet	0.37 to 0.57	N/A		No breakout above toe	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	1.93				Yes	
100-year + 4 feet	1.92				Yes	
Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):	Drained Stability Berm with Undrained Seepage Berm.					
Dimensions of Primary Features:	**10+00 to 129+66**, 8 feet tall Drained Stability Berm with 88 feet wide Undrained Seepage Berm. Assume 25% of reach will have an undrained seepage berm width extending to 100 feet due to taller levee at some locations. Stability berm is 10 feet wide at top, 2 feet thick drain, and 3H:1V slope. Seepage Berm thickness - 5 feet at levee toe and 3 feet at berm toe.					
Seepage						
WSE	Exit Gradient Levee Toe	Stability Berm Toe	Seepage Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot		0.16 to 0.23	0.39 to 0.62	Bottom of stability berm	Yes	
100-year + 4 feet		0.21 to 0.30	0.44 to 0.72	Bottom of stability berm	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	1.90				Yes	
100-year + 4 feet	1.80				Yes	

TABLE 5-2A: CHARACTERIZATION OF REACH 2
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
2	SBFCA 129+66 [DWR 2286+00]	SBFCA 218+66 [DWR 2375+00]	1.69	**Toe** CPT – 9 Borings – 7 **Crown** CPT – 8 Borings – 15 **Field** CPT – 3 Borings – 5	**Crown Width** • 15 to 20 feet **Landside Slope** • 1.5H:1V to flatter than 2H:1V **Waterside Slope** • 2H:1V to flatter than 3H:1V	• **130+27 to 181+83 (2286+46 to 2338+04) Previous Flood Events:** Excessive seepage occurs during high water between Sacramento Avenue and Laurel Avenue; • **132+87 (2289+07) 1997:** Heavy seepage running clear; • **139+46 (2295+67) 1997:** Heavy seepage running clear; • **144+63 to 146+73 (2300+84 to 2302+92) 1997:** Heavy seepage running clear; • **163+88 (2320+07) 1997:** Heavy seepage running clear; • **175+60 (2331+79) 1997:** Heavy seepage running clear; • **178+72 (2334+92) 1997:** Heavy seepage around tree, running clear; • **183+40 (2339+60) 1997:** Heavy seepage running clear; • **193+16 to 194+70 (2349+37 to 2350+92) 1997:** Heavy seepage running clear; • **197+97 (2354+07) 1986:** Boil 200 feet from landward toe, water was clear and did not carry material; • **201+92 (2358+13) 1997:** Heavy seepage running clear; • **198+42 (2354+62) 1986:** A landside boil occurred in a landside drainage ditch near the levee toe during the 1986 flood, an area of boils appeared at the landside toe and away from the toe, there appears to be a swale or old slough at this location, ditches were reported to be excavated on both sides of the levee to explore for the cause of the boils but no explanation found; • **206+15 to 242+00 (2362+13 to 2398+27) 1997:** Heavy seepage into existing ditch caused sloughing of the bank and the levee slope, clear seepage was entering the ditch. Particularly heavy seepage and sloughing at toe of levee at 2370 to 2374. • **203+77 to 218+79 (2360+00 to 2375+00):** Per B. Hampton of LD-1, this is area of old levee breach and reconstruction. Date unknown. • Based on DWR's Seepage Area map for the Feather River West Levee area, 100% of Reach 2 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 100% of Reach 2 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• **129+76 to 190+41 (2286+00 to 2346+55) 1962/1963**: Landside toe berm with 15'-deep drain present (constructed as part of 1962-63 USACE levee stabilization project); • **190+34 to 198+79 (2346+55 to 2355+00 and 2355+41 to 2366+00) 1998**: Toe drain and berm constructed under the Marysville/Yuba City Levee Reconstruction Project (F.R. Site B), 41-foot gap in berm at Laurel Avenue levee ramp access. Also, this is Site 11 (toe drain and berm planned) on project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3" (As-builts?); • **206+29 to 241+93 (2362+13 to 2398+27) 1998**: During 1997 flood, emergency stability berm constructed using sandbags placed on geotextile fabric in drainage ditch, later that year this was converted to a pervious toe drain under the Marysville/Yuba City Levee Reconstruction Project (F.R. Site B).	• **Landside:** Between Stations **129+76 to 190+41 (2286+00 and 2355+50)** Geomorphology consists of alternating Ra and Rofc. Between Stations **198+79 (2355+00)** and the end of the reach the Geomorphology consists predominantly of Ha with fingers of Rdc. • **Waterside:** Consists of Ra along reach.	• **129+76 to 173+80 (2286+00 to 2330+00)**: Moderate resistivity to depths of 20-40' followed by high resistivity to depths of 100-120'. Moderate resistivity at depths greater than 100-120'; • **173+80 to 218+31 (2330+00 to 2375+00):** Moderate resistivity to depths of 15 to 20 feet (high resistivity zones from **173+80 to 188+80 (2230+00 to 2345+00)** on land/water side), thick very high resistivity zone beneath to depths over 100'.

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
2	SBFCA 129+66 [DWR 2286+00]	SBFCA 218+66 [DWR 2375+00]	• **Levee Fill:** Consists of variable lenses of clay, silt, sandy silt, silt sand and silty sand. • **Blanket:** With the exception of two borings at Stations 151+80 and 170+05 (2308+00 and 2326+25), the Reach contains a relatively consistent clay and silty clay blanket between 10 and 25 ft thick. The blanket is overlain and underlain by silty sand and sandy silt of varying thicknesses. At Stations 151+80 and 170+05 (2308+00 and 2326+25) borings indicate levee is underlain by sandy silt to 25 feet below the levee toe elevation. Below an elevation of approximately 0 to 10 ft the borings show interbedded layers of CL/GW-GP/SP/SM/ML to the depths explored.	1) DWR 2337+50 (SBFCA 181+13) = **100-yr, 54.1 ft** 2) Average Head = 16.2 ft	Seepage berm with toe drain located along land side levee toe between Stations **129+76 and 210+30 (2286+00 and 2366+50)**. Irrigation pipe through levee crown at Station **199+80 (2365+00)**. Concrete ditch near end of Reach starting at Station **216+80 (2373+00)** continuing into Reach 3. Audubon Society dirt parking lot and metal structure located near the landslide toe at Station **199+80 (2365+00)**. Water side ditch along entire alignment.	SBFCA Station 181+13 [DWR 2337+50]	1) WM0003_034C, WM0003_035C, WM0003_003A, and WM0003_018C	Presence of sandy layers both in embankment and foundation. Potential thin blanket condition.

TABLE 5-2B: ANALYSIS RESULTS FOR REACH 2
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 2
Analysis Station: 181+13

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.51 to 1.83			4' above toe	No	Results are consistent with past performance of the reach, which consists of significant boils and sloughing at the toe of the levee.
100-year + 3 feet						
Landside Stability						
WSE		Performance			Meets Criteria?	
100-year						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	
100-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 2 is a soil-bentonite cutoff wall extending below sand/silty sand layers into a less permeable clay/silt layer. This alternative was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 2 includes the use of both stability berm with seepage berm and a shallow cutoff wall with a seepage berm. The seepage berm material type is lean clay, which is similar to the shallow near surface layers. The shallow cutoff wall reduces the potential for seepage through shallow near surface sandy layers and unacceptable gradients through the berm near the levee toe.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall.

Dimensions of Primary Features: **129+66 to 181+00** Cutoff Wall Tip Elevation 20 ft, **181+00 to 191+00** Cutoff Wall Tip Elevation (-10 feet), **191+00 to 218+66** Cutoff Wall Tip Elevation (-73 feet) with full levee de-grade/re-grade. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade except between station **191+00 and 218+66** where a full levee de-grade/re-grade is required.

Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	0.09 to 0.12	N/A		Toe	Yes	
100-year + 4 feet	0.15 to 0.13			Toe	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	1.78				Yes	
100-year + 4 feet	1.75				Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Drained Stability Berm with Undrained Seepage Berm and Cutoff Wall with Undrained Seepage Berm

Dimensions of Primary Features: **129+66 to 181+00**, 8 feet tall Drained Stability Berm with 100 feet wide Undrained Seepage Berm, **181+00 to 218+66** Cutoff Wall Tip Elevation 30 feet with 100 feet wide Undrained Seepage Berm. Stability berm is 10 feet wide at top, 2 feet thick drain, and 3H:1V slope. Seepage Berm thickness - 5 feet at levee toe and 3 feet at berm toe. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade.

Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	0.15	0.57		At the toe of the slope/top of the seepage berm	Yes	Remove thin CL Blanket and underlying SM (4 feet by 200 feet) and replace with Undrained Stability Berm material (CL).
100-year + 4 feet	0.22	0.66			Yes	Remove thin CL Blanket and underlying SM (4 feet by 200 feet) and replace with Undrained Stability Berm material (CL).
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	1.86				Yes	
100-year + 4 feet	1.81				Yes	

TABLE 5-3A: CHARACTERIZATION OF REACH 3
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
3	SBFCA 218+66 [DWR 2375+00]	SBFCA 300+66 [DWR 2457+00]	1.55	**Toe** CPT – 5 Borings – 9 **Crown** CPT – 9 Borings – 13 **Field** CPT – 3 Borings – 1	**Crown Width** • 17 to 20 feet **Landside Slope** • 1.5H:1V to flatter than 2H:1V **Waterside Slope** • 2H:1V to flatter than 3H:1V	• **206+15 to 242+00 (2362+13 to 2398+27) 1997**: Heavy seepage into existing ditch caused sloughing of the bank and the levee slope, clear seepage was entering the ditch. • **253+70 to 257+92 (2410+00 to 2414+00):** Per B. Hampton with LD-1, water level in landside pond rises with increasing water level behind levee, which leads to bank erosion in the pond. • Based on DWR's Seepage Area map for the Feather River West Levee area, 95% of Reach 3 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 90% of Reach 3 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• **206+29 to 241+93 (2362+13 to 2398+27) 1998**: During 1997 flood, emergency stability berm constructed using sandbags placed on geotextile fabric in drainage ditch, later that year this was converted to a pervious toe drain under the Marysville/Yuba City Levee Reconstruction Project (F.R. Site B). • **218+31 to 298+97 (2375+00 to 2455):** Drain consists of fabric-wrapped gravel placed in existing ditch and then backfilled. Concrete lined landside ditch on slightly elevated berm.	• **Landside:** Geomorphology along levee toe consists of alternating Ha and Rob units. One section of Rcs is shown against the levee toe between Stations **291+05 and 292+80 (2447+25 and 2449+00)**. • **Waterside:** Geomorphology consists of Ra.	• **218+31 to 241+80 (2375+00 to 2398+00)**: Low to moderate resistance throughout profile (low within upper 10-20' of this segment); • **241+80 to 288+80 (2398+00 to 2445+00)**: Alternating high to moderate resistivity zones within the upper 10-20' (waterside mainly high resistivity near-surface) followed by high to very high resistivity to depths of about 100', moderate resistivity below 100'; • **288+80 to 300+80 (2445+00 to 2457+00)**: Primarily high to very high resistivity to a depth of about 50', occasional moderate resistivity zones within the upper 10-20'.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
3	• **Levee Fill:** Consists of variable lenses of clay, silt, sandy silt, silt sand and silty sand. • **Blanket:** ML and CL blanket is present below the levee toe at thicknesses ranging from 10 and 30 ft. The blanket is overlain by thin lenses (less than 5 ft) of silty sand and sand at a few of the boring locations. The blanket is underlain by 10 to 40 ft layers of sand and silty sand. Below an elevation of approximately -20 the borings show interbedded layers of CL/ML/GW/GP//SM to the depths explored.	1) DWR Station 2396+00 (SBFCA Station 239+78) = **100-yr 54.6 ft**, **100-yr + 3ft = 57.5 ft** 2) Average Head 100 yr = 10.6 ft, Average Head 100 yr + 3ft = 13.6 ft	Concrete lined ditch located on land side along entire reach. Pond located on land side near Station **255+80 (2412+00)**. Irrigation pump structure located on water side near Station **298+97 (2455+00)**. Irrigation piping from pump to farm travels through Levee crown near Station **298+97 (2455+00)**. Water side borrow ditch appears to end at Station **265+30 (2421+50)**.	SBFCA Station 239+78 [DWR Station 2396+00]	1) WM0003_022B, WM0003_008C, SM0003_008H	Presence of thin fine-grained blanket layer. The shallow sand and silty layer is about 50 feet thick.

TABLE 5-3B: ANALYSIS RESULTS FOR REACH 3
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 3
Analysis Station: 239+78

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.5	1.7	1.34	Toe	No	Results are consistent with past performance of significant seepage and sloughing at the levee toe.
100-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
100-year						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	
100-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative 1 for Reach 3 is a soil-bentonite cutoff wall extending below sand/silty sand layers into a less permeable clay/silt layer. This alternative was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 3 consists of a stability berm with a seepage berm. The seepage berm material type is silty sand, which is similar to the shallow near surface layers. Alternative 2 mitigates underseepage and through seepage. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):	Soil-Bentonite Cutoff Wall					
Dimensions of Primary Features:	**218+66 to 220+00** Cutoff Wall Tip Elevation (-73 feet), **220+00 to 230+00** Cutoff Wall Tip Elevation 20 feet, **230+00 to 250+00** Cutoff Wall Tip Elevation (-35 feet), **250+00 to 289+00** Cutoff Wall Tip Elevation (-20 feet), **289+00 to 300+66** Cutoff Wall Tip Elevation 15 feet. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade.					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	0.08	0.48		At toe of the levee	Yes	
100-year + 4 feet	0.11	0.56		At toe of the levee	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	1.4				Yes	
100-year + 4 feet	1.4				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):	Drained Stability Berm with Undrained Seepage Berm.					
Dimensions of Primary Features:	**218+66 to 300+66** 8 feet tall Drained Stability Berm with 300 feet Wide Undrained Seepage Berm with monitoring for seepage at the toe of the Undrained Seepage Berm. Drained Stability Berm is 10 feet wide at top, 2 feet thick drain, and 3H:1V slope. Undrained Seepage Berm thickness - 5 feet at levee toe and 3 feet at berm toe					
Seepage						
WSE	Exit Gradient Levee Toe	Stability Berm Toe	Seepage Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot		0.17	1.31	Bottom of stability berm/top of seepage	Yes	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
100-year + 4 feet		0.29	1.52		Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
100-year + 1 foot	2.01				Yes	
100-year + 4 feet	2.00				Yes	

TABLE 5-4A: CHARACTERIZATION OF REACH 4
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
4	SBFCA 300+67 [DWR 2457+00]	SBFCA 410+67 [DWR 2567+00]	2.1	**Toe** CPT – 5 Borings – 1 **Crown** CPT – 11 Borings – 5 **Field** CPT - 1	**Crown Width** • 15 to 20 feet **Landside Slope** • 1.7H:1V to flatter than 2H:1V **Waterside Slope** • 1.5H:1V to flatter than 3H:1V	• **359+42 (2515+65) 1986**: During the 1986 flood, a crack formed in the levee. (F.R. Site C) This site is located 2.4 miles south of Star Bend (LM 1.5). • **362+10 to 370+02 (2518+29 to 2526+22) 1997**: During the 1997 flood, seepage occurred at the site (same location of 1986 crack). (F.R. Site C) • Based on DWR's Seepage Area map for the Feather River West Levee area, 85% of Reach 4 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 80% of Reach 4 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• **354+50 to 364+44 (2510+73 to 2520+67) 1998**: Toe Drain and Seepage/Stability Berm constructed. A pervious toe drain and seepage/stability berm have been constructed at this site under a PL 8499 action (F.R. Site C)	• **Landside:** Between **300+80 and 349+80** (**2457+00 and 2506+00)** Geomorphology consists of alternating Ra, Ha and Rcs. Between **349+80 and 385+80** (**2506+00 and 2542+00)**- Geomorphology consists of alternating Hob and Rcs. Between **385+80** (**2542+00) to the end of the reach** the Geomorphology consists of alternating layers of Rob and Qmu. • **Waterside**: Geomorphology consists of Ra from the **start of the reach to 311+80** (**2468+00)** and consists of Rb from **311+80 (2468+00) to the end of the reach**, with Rofc between Ra and Rb and the water.	• **0 - 10' bgs:** Generally low to moderate resistance crest and land side, high resistance water side. Exceptions: At crest: low resistance from 313+80 to 327+80, 351+80, 353+80, from 408+30 to 411+80 (**2470+00 to 2484+00, 2508+00, 2510+00, from 2564+50 to 2568+00)**; land side: low resistance from 312+80 to 315+30, 312+80 to 410+80 (**2469+00 to 2471+50, 2469+00 to 2567+00)**; water side - low resistance between 323+80 to 334+80 (**2480+00 to 2491+00)** and in between moderate zones at 320+30 to 337+80, 377+80 to 379+80 (**2476+50 to 2494+00, 2534+00 to 2536+00)**. Moderate resistance water side**:** 350+30 (**2506+50) to end of reach** with some high resistance near 393+80 and 406+80 (**2550+00 and 2563+00)**. • **10' - 40' (50' water side):** High resistance from 300+80 to 317+80 (**2457+00 to about 2474+00)** for water, land and crest. 317+80 (**2474+00) to end of reach**, moderate resistance crest and water side, with areas of low resistance along the crest, mainly low resistance land side. • **40' to 150':** moderate resistance for crest, land and water side except areas of low resistance land side from 319+80 to 337+80 and 406+80 to 410+80 (**2476+00 to 2494+00 and 2563+00 to 2567+00)**.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
4	• **Levee** materials generally consist of sandy silt and silty sand with lesser sand and clay layers between 300+80 and 363+80 (2457+00 and 2520+00), then predominately silt and clay between 363+80 and 410+80 (2520+00 and 2567+00). • **Blanket** generally consists of silt and clay that extends 25 to 40 feet below the base of the levee. Local soft layers are present 6 to 10 feet below the base of the levee. Isolated silty sand layers are present within the blanket between approximate 372+80 and 391+80 (2529+00 and 2548+00). Below about elev. 12 ft MSL, soil generally consists of alternating layers/lenses of silty sand, sand, gravel, silt, and clay of varying thickness.	1) DWR 2462+50 (SBFCA 306+33) **100-yr = 54.7 ft, 100 yr + 3 ft = 57.7 ft** 2) Average Head 100 yr = 12.5 ft, Average Head 100 yr + 3ft = 15.5 ft	At about **409+80 (2566+00)**, sump pond located at landside toe and pump station located at riverside toe with associated pipes crossing through levee. Otherwise the area adjacent to the landside levee is used for agricultural purposes.	SBFCA 306+33 [DWR 2462+50]	1) WL0001_023C: SL0001_001C: SL0001_001H	Shallow thin previous layers are present at this section, which is a good representation of this reach.

*SBFCA station numbers based on 2156+20 subtracted from older DWR stations.
**SBFCA station numbers provided by WoodRodgers.

TABLE 5-4B: ANALYSIS RESULTS FOR REACH 4
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 4
Analysis Station: 306+33

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.17 to 0.46	N/A	0.2 to 0.52	5.8 feet above Toe	No	The high phreatic surface breakout point and granular fill indicate potential for through seepage. Past performance records also indicate underseepage.
100-year + 3 feet	0.22 to 0.59		0.26 to 0.64	7 feet above Toe	No	

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
100-year				
100-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	
100-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 4 is a soil-bentonite cutoff wall extending below sand/silty sand layers into a less permeable clay/silt layer. Alternative 2 for Reach 4 is a stability berm with a seepage berm. Both alternatives were selected to mitigate through-seepage and underseepage.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall.

Dimensions of Primary Features: **300+66 to 349+00** Cutoff Wall Tip Elevation 15 feet, **349+00 to 368+00** Cutoff Wall Tip Elevation 10 feet, **368+00 to 410+67** Cutoff Wall Tip Elevation 20 feet. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade.

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	See Reach 1				Yes	Subsurface conditions, past performance and Problem Identification seepage results are similar to Reach 1 and therefore mitigation is consistent with Reach 1 results.
100-year + 4 feet	See Reach 1				Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
100-year + 1 foot	See Reach 1		Yes	
100-year + 4 feet	See Reach 1		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Drained Stability Berm with Undrained Seepage Berm

Dimensions of Primary Features: **300+66 to 410+67**, 8 feet tall Drained Stability Berm and 100 feet wide Undrained Seepage Berm. Stability berm is 10 feet wide at top, 2 feet thick drain, and 3H:1V slope. Seepage Berm thickness - 5 feet at levee toe and 3 feet at berm toe

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year + 1 foot	See Reach 1			Bottom of stability berm/top of seepage berm	Yes	Subsurface conditions, past performance and Problem Identification seepage results are similar to Reach 1 and therefore mitigation is consistent with Reach 1 results.
100-year + 4 feet	See Reach 1				Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
100-year + 1 foot	See Reach 1		Yes	
100-year + 4 feet	See Reach 1		Yes	

TABLE 5-5A: CHARACTERIZATION OF REACH 5
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA)* [DWR ULE STATION]	 END STATION (SBFCA)* [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
5	SBFCA 410+67 [DWR 2567+00]	SBFCA 478+67 [DWR 2635+00]	1.3	**Toe** CPT – 9 Borings – 3 **Crown** CPT – 7 Borings – 4 **Field** CPT - 2	**Crown Width** • 15 to 20 feet **Landside Slope** • 1.5H:1V to flatter than 2H:1V **Waterside Slope** • 2H:1V to flatter than 3H:1V	• **425+33 to 478+80** (**2581+57 to 2635+00) 1997**: Four boils at landside toe of levee. • **467+47 to 495+72 (2623+92 to 2650+22) 1986**: During the 1986 flood, boils carrying soil formed near the landside toe of the levee. The ground within approximately 100 feet of landside toe was very soft and wet. The peak floodwater was 5-6 feet below the top of the levee at this location. LDI personnel constructed sandbag rings around three of the worst boils. (F.R. Site D) • Based on DWR's Seepage Area map for the Feather River West Levee area, 100% of Reach 7 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 100% of Reach 7 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation.	• **242+80 to 478+80** (**2581+00 to 2635+00) unknown date**: An undrained stability berm was constructed at landside toe of levee.	• **Landside:** Between **410+80 (2567+00)** to the end of the reach Geomorphology consists predominantly of Rob with a finger of Qmu. • **Waterside**: Consists of Rb with Rofc between Rb and the water.	• Crest, LS, and WS Profiles similar. **0 - 150' bgs**: Generally alternating moderate to high resistance for entire reach to about 100' depth, high resistance depths greater than 100'. Exceptions: Low resistance noted from **410+80 to 420+80** (**2567+00 to 2577+00**), and from **435+80 to 439+80** (**2592+00 to 2596+00**) at about 40' to 70' depth landside, and from **443+80 to 457+80** (**2600+00 to 2614+00**) from the ground surface to 10' and dipping to about 50' near **447+80 (2604+00)** landside.

REACH ID [1]	START STATION (SBFCA)* [DWR ULE STATION]	END STATION (SBFCA)* [DWR ULE STATION]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] DWR ULE STATION (SBFCA STATION)	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
5	SBFCA 410+67 [DWR 2567+00]	SBFCA 478+67 [DWR 2635+00]	• **Levee** materials generally consist of silt and clay with lenses of sandy silt at about Sta. 438+80 (2595+00). • **Blanket** generally consists of silt and clay that extends 5 to 20 feet below the base of the levee. A hardpan layer is generally present at a depth of 5 to 9 feet except at 416+80 (2573+00), 427+34 (2583+54), and 466+40 (2622+60). Sand and silty sand layers/lenses 4 to 10 feet thick are present at a depth of 5 to 20 feet below the base of the levee. Sand and gravel layer present at about 30 to 35 feet below the base of the levee that extends to the bottom of 110'- deep explorations.	1) DWR 2633+00 (SBFCA 477+01) **200-yr 64.0 ft** 2) Average Head 200yr = 13.4 ft	No significant constraints or features identified within this Reach. No landside ditches present. The area adjacent to the landside levee is used for agricultural purposes. Significant water-side excavations present along the entire reach.	1) DWR 2633+00 (SBFCA 477+01)	1) WL0001_020C: WL001-059C: WL0001_036C: B5-07 (BCI): B1-06 (BCI): SL0001_003C: SL0001- 003H	A relatively thick layer of sand and silty sand present at about 20 feet below the landside levee toe. The thickness of this layer increases landside based on the toe and field explorations.

TABLE 5-5B: ANALYSIS RESULTS FOR REACH 5
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 5
Analysis Station: 477+01

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.40 to 0.46	0.99 to 1.75		At Toe	No	Results are consistent with historic significant seepage and boils at the toe of the levee.
Landside Stability						
WSE		Performance			Meets Criteria?	
200-year						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	
200-year + 1 foot	typ. winter WSE					
Rationale for Selection of Two Alternatives for Analysis						
Alternative 1 for Reach 5 is a soil-bentonite cutoff wall and a soil-bentonite cutoff wall with a 200 feet seepage berm. Alternative 2 for Reach 5 is a seepage berm. The seepage berm material type is lean clay, which is similar to the shallow near surface layers. The alternative reduces the potential for seepage through shallow near surface sandy layers and unacceptable gradients. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.						
Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):	Soil-Bentonite Cutoff Wall and Soil-Bentonite Cutoff Wall with Seepage Berm					
Dimensions of Primary Features:	**410+67 to 417+00** Cutoff Wall Tip Elevation 20 feet, **417+00 to 425+00** Cutoff Wall Tip Elevation 10 feet, **425+00 to 456+00** Cutoff Wall Tip Elevation 15 feet, **456+00 to 478+68** Cutoff Wall Tip Elevation 15 feet with 200 feet Wide Undrained Seepage Berm. Cutoff wall is 3 feet wide and requires 1/2 levee de-grade/re-grade. Undrained Seepage Berm thickness: 5 feet at levee toe and 3 feet at berm toe.					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	See Reach 3				Yes	Subsurface conditions, past performance and Problem Identification seepage results are similar to Reach 3 and therefore mitigation is consistent with Reach 3 results.
200-year + 4 feet	See Reach 3				Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	See Reach 3				Yes	
200-year + 4 feet	See Reach 3				Yes	
Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):	Undrained Seepage Berm					
Dimensions of Primary Features:	300 feet Wide Undrained Seepage Berm. Undrained Seepage Berm thickness: 5 feet at levee toe and 3 feet at berm toe.					
Seepage						
WSE	Exit Gradient Levee Toe	Seepage Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	See Reach 3				Yes	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	See Reach 3				Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	See Reach 3				Yes	
200-year + 4 feet	See Reach 3				Yes	

TABLE 5-6A: CHARACTERIZATION OF REACH 6
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2]		LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
	START STATION (SBFCA)* [DWR ULE STATION]	END STATION (SBFCA)* [DWR ULE STATION]							
6	SBFCA 478+67 [DWR 2635+00]	SBFCA 510+37 [DWR 2676+00]	0.78		3:1 land-side and water-side slopes with minimum 20-foot-wide crown.	• Significant seepage prior to construction of setback levee and slurry wall. • Based on DWR's Seepage Area map for the Feather River West Levee area, 95% of Reach 6 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 95% of Reach 6 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• Setback levee with SB cutoff wall constructed in 2009 within this entire reach. Cutoff wall tie-ins are SCB and extend 125' north into the existing levee and 150' south into the existing levee. The SB cutoff wall is 40 to 45 feet deep at the north tie-in and 62 to 67 feet deep at the south tie-in. • The new levee was constructed of levee fill meeting USACE classification and compaction criteria. • The wall depths and cutoff clay layer depths were confirmed during construction. • Seepage, stability and settlement analyses were performed during design of the setback levee by Blackburn Consulting consistent with USACE criteria and results indicate all criteria was met with regards to seepage gradients, slope stability factors of safety and settlement magnitude.		
			GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] DWR ULE STATION (SBFCA STATION)	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]	
								New setback levee and SB cutoff wall along entire reach constructed in 2009 in accordance with USACE criteria. This reach will be "certified" under the Star Bend Setback Levee project close-out documentation, which will be referenced in the final FRWL design-level report.	

TABLE 5-7A: CHARACTERIZATION OF REACH 7
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT – TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA)* [DWR ULE STATION]	 END STATION (SBFCA)* [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION – CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
7	SBFCA 510+37 [DWR 2676+00]	SBFCA 598+87 [DWR 2765+00]	1.68	**Toe** CPT – 23 Borings – 11 **Crown** CPT – 30 Borings – 14 **Field** CPT – 4	**Crown Width** • 15 to 20 feet **Landside Slope** • 2H:1V to flatter than 2H:1V **Waterside Slope** • 3H:1V to flatter than 3H:1V	• **510+69 to 556+25 (2677+06 to 2722+50) 1995:** Seepage -- the site is located between Star Bend Road and Abott Road at approximate River Mile 18.1 and 19.0 (LM 4.1 to 5.0). During the 1995 flood, clear seepage exited the levee toe and the ground beyond the levee toe while the river level was approximately 12 to 15 feet below the top of levee. (F.R. Site E). • **510+69 to 556+25 (2677+06 to 2722+50) 1997:** Boils and seepage. During the 1997 flood, numerous boils occurred in a 200 linear foot stretch. Sandbag rings were constructed around the boils that were moving material. The following day, the sandbagged boils were flowing clear. (F.R. Site E) • **521+50 (2687+62) 1997:** Boil at landside levee toe. • **532+60 (2698+72) 1997:** Two boils at landside levee toe. • **534+71 (2700+83) 1997:** Three boils at landside levee toe. • **540+02 (2706+13) 1997:** Three boils at landside levee toe. • **543+20 to 547+58 (2709+28 to 2713+51) 1997:** Seventeen (17) boils at landside levee toe. • **563+00 to 568+74 (2729+00 to 2735+00) 1997:** Waterside slope instability due to rapid drawdown when levee breach occurred on east bank of Feather river. • **592+34 (2758+48) 1986:** Erosion, site located at LM 6.1 near Abbott Lake, north of O'Banion Road at approximately River Mile 19.7. During the 1986 flood, holes appeared at the top of waterside levee berm. It is believed that the holes were result of small trees growing on the berm. The berm slope down to natural ground was eroded. The water was within 5 feet of the top of the levee (F.R. Site F) 2006 - Boils reported in 2006 adjacent to and north of the relief well field. • Based on DWR's Seepage Area map for the Feather River West Levee area, 95% of Reach 7 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 95% of Reach 7 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• **510+87 to 555+88 (2677+00 to 2722+00) date unknown**: Toe Berm constructed from the north end of Star Bend Road to the south end of Abbott Lake Road. Project Plans entitled "PL 84-99 Phase III, Relief Wells - LD1, Feather River at Star Bend, dated 8/18/1997. USACE File No. 04-04-617" • **526+72 to 542+10 (2692+50 to 2708+20) 1997**: In 1997, the Corps of Engineers installed relief wells in this area to reduce seepage and instability of the levee under PL 84-99 contract. (F.R. Site E). A total of 25 relief wells from 1550 feet north of north end of Star Bend Road to 1380 feet south of the south end of Abbott Lake Road. Project Plans entitled "PL 84-99 Phase III, Relief Wells - LD1, Feather River at Star Bend, dated 8/18/1997. USACE File No. 04-04-617". Significant seepage continued (heavy in 2006) after wells were constructed. Top of wells and collection ditch appear to be too high to mitigate seepage and/or no confining layer present to force near-surface water into wells. • **563+05 to 568+76 (2729+00 to 2735+00 with an approximately 120 feet gap):** Waterside slope repair by USACE after the 1997 flood. A gap of about 120 feet was left unrepaired.	• **Landside:** Between **510.25 to 568+38** (**2676+00 to 2734+50)** Geomorphology consists of alternating Rcs, Rob, Qmu. From **568+38** (**2734+50) to the end of the reach**, Geomorphology consists of Qmu. • **Wateside**: Geomorphology consists of Rb from the beginning **of the reach to 517+88** (**2684+00)** and then consists of Ra to the end of the reach.	• **0 - 10' bgs:** Generally low resistance from **509+88 to 529+88** (**2676+00 to 2696+00)** with some moderate resistance interspersed. Alternating moderate and high resistance **529+88** (**2696+00) to the end of the reach**. • **10' - 40':** Moderate resistance land and crest, moderate to high resistance water side **509+88 to 535+88** (**2676+00 to 2702+00)**. Exception: low resistance from about **509+88 to 526+88** (**2676+00 to 2693+00)** land side. High resistance **526+88 to 580+88** (**2693+00 to 2747+00) 583+88 (2750+00 water side)**. Moderate resistance to end of reach. • **40' to 150':** Generally moderate resistance land and water side from **509+88 to 583+88** (**2676+00 to 2750+00)** except areas of low resistance land side from **555+88 to 558+38** (**2722+50 to 2724+50)** at about 120' to 150' depths and water side from **548+88 to 555+88** (**2715+00 to 2722+00)** at about 100' to 150' depths. High resistance water side from **beginning of reach to 528+88 (2695+00)** to depths of 60' and to **519+88** (**2686+00)** at greater depths. Crest - moderate resistance 40' to about 100', low resistance 100' to 150' (moderate resistance from **566+88 to 575+88** (**2733+00 to 2742+00**). From **583+88 to 598+88** (**2750+00 to 2765+00)**, moderate resistance from 40' to about 100' depths, low resistance from 100' to 150' depths.

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] DWR ULE STATION (SBFCA STATION)	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
• **Levee** materials generally consist of silt and clay with lenses of sandy silt. • **Blanket** generally consists of silt and clay that extends 7 to 10 feet below the base of the levee. A hardpan layer is locally present below the silt and clay. Locally, 1 to 2 foot thick lenses of sand and silty sand are present near the base of the levee. Sand and silty sand layers/lenses 5 to 35 feet thick are present at a depth of 7 to 10 feet below the base of the levee. 1' to 2' thick lenses of very soft, fine grained soil (sometimes organic) within the upper 5' of soil below levee and at the toe. This is likely old lakebed material.	1) DWR 2705+00 (SBFCA 539+30) **200-yr 65.9 ft** 2) Average Head = 18 ft	Landside lined ditch present at levee toe between approximate 511+88 and 555+88 (2678+00 and 2722+00) at levee toe. Relief wells present at levee toe between 526+38 to 542+08 (2692+50 to 2708+20). The area adjacent to the landside levee is used for agricultural purposes. Significant water-side excavation present within 50 to 100 feet of levee toe. Levee was constructed over old lakebed.	1) SBFCA 539+30 (DWR 2705+00)	1) WL0001_084C: WL0001_088B: WL0001_089B: SL0001_004C: SL0001_004H	Blanket layer is approximately 10 feet thick and underlain by sand and silty sand layers.

TABLE 5-7B: ANALYSIS RESULTS FOR REACH 7
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Results for Reach 7
Analysis Station: 539+30

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	1.55	N/A	2.7	11.5 feet above Toe	No	Results are consistent with significant historic seepage and boils.
Landside Stability						
WSE		Performance			Meets Criteria?	
200-year						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	
200-year + 1 foot	typ. winter WSE					
Rationale for Selection of Two Alternatives for Analysis						
Alternative 1 for Reach 7 is a soil-bentonite cutoff wall extending below sand/silty sand layers into a less permeable clay/silt layer. This alternative was selected as an in-place alternative to mitigate both through-seepage and underseepage. Alternative 2 for Reach 7 consists of a stability berm with a seepage berm. The seepage berm material type is lean clay, which is similar to the shallow near surface layers. Alternative 2 mitigates underseepage and through seepage. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.						
Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):	Soil-Bentonite Cutoff Wall					
Dimensions of Primary Features:	**510+37 to 528+00** Cutoff Wall Tip Elevation 15 ft, **528+00 to 546+00** Cutoff Wall Tip Elevation (-10 ft), **546+00 to 565+00** Cutoff Wall Tip Elevation (-65 ft) with Full Levee Degrade, **565+00 to 576+00** Cutoff Wall Tip Elevation (-50 ft), **576+00 to 584+00** Cutoff Wall Tip Elevation (-10 ft), **584+00 to 598+87** Cutoff Wall Tip Elevation 20 ft. Cutoff wall is 3 ft wide and requires 1/2 levee de-grade/re-grade except between station **546+00 to 565+00** where a full levee de-grade/re-grade is required.					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.05	N/A		Toe	Yes	
HTOL	0.05			Toe	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.09				Yes	
HTOL	2.09				Yes	
Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):	Drained Stability Berm with Undrained Seepage Berm					
Dimensions of Primary Features:	**510+37 to 598+87**, 9.5 feet tall, Drained Stability Berm with 300 feet Wide Undrained Seepage Berm with monitoring for seepage at the toe of the berm. Drained Stability Berm is 13 feet wide at top, 2 feet thick drain, and 3H:1V slope. Height of the Undrained Seepage Berm at Levee Toe is 7 feet tall due to high gradients across the seepage berm/blanket, 3 feet tall at berm toe.					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.47	1.39		Stability Berm Toe	No	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
HTOL	0.57	1.53		Stability Berm Toe	No	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.77				Yes	
HTOL	1.57				Yes	

TABLE 5-8A: CHARACTERIZATION OF REACH 8
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
8	598+84 [2765+00]	654+75 [2821+00]	1.11	Total Number of Explorations = 19; Crest Explorations = 11 (SPT - 5, CPT - 6); Landside Toe Explorations = 6 (SPT - 3, CPT - 3); Landside Field Explorations = 2 (CPT - 2)	**Crown Width :** approximately 20 feet **Landside slope** approximately 2H:1V to flatter than 2H:1V **Waterside Slope** approximately 2H:1V to flatter than 3H:1V	602+88 [2769+01] to 640+32 [2806+45]: Twenty-six (26) sinkholes on waterside berm of levee (1997). According to Bill Hampton of LD1, these sinkholes were 3 feet to 6 feet deep and 2 feet to 4 feet in diameter and formed on the waterside berm, which was the original levee before it was raised. Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 2/3 rd of Reach 8 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 2/3 rd of Reach 8 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations.	602+88 [2769+01] to 640+32 [2806+45]: After the flood, 26 sinkholes were repaired by compacting soil (1997).	R_{SL} (Recent Slough deposits) R_{a} (Recent Alluvium) R_{ob} (Recent Overbank)	High resistivity soil layers present at approximately 10 feet below the embankment. The highly resistive layers extends to the ground surface occasionally.

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: Sand (SP, SM), Silt and Clay **Foundation**: Near Surface layers include Sand (SP), Clay (CL). Landside toe and field explorations indicate shallow Sand layers. Hardpan and soft layers were encountered within shallow foundation layers.	**200 yr WSE** Head = approximately 18.8 feet at the analysis section	Mostly Agricultural. A pond is located on the landside of the levee between Sta. 646+86 [2813+00] and Sta. 648+86 [2815+00].	623+86 [2790+00]	WL0001_049B, WL0001_049C, 2F-91-10 (CREST), WL0001_074C, WL0001_061C, SL0001_005C (LANDSIDE TOE), WL0001_117C, 2F-91-10A, WL0001_110C (FIELD)	Presence of sand in levee embankment may indicate potential for through seepage; shallow sand layers in foundation and relatively thin fine-grained blanket may indicate potential for underseepage; deep sand layers encountered at approximately similar depths as other explorations within this reach.

TABLE 5-8B: ANALYSIS RESULTS FOR REACH 8
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 623+86

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.77	N/A	N/A	1.0	No	The existing levee embankment material is silty sand underlain by approximately 5.2 feet of silty sand and 4.3 feet of clay blanket.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 8 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through seepage and underseepage potential. Alternative 2 for Reach 8 is a seepage berm with a shallow cutoff wall. The seepage berm material type is silty sand, which is similar to the shallow near surface layer. The shallow cutoff wall reduces the potential for seepage through shallow near surface sandy layers.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 3 feet wide and 45 feet deep cutoff wall starting at Elevation 60 (half levee degrade)

Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No breakout	Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	<0.1	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.90				Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	1.89				Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm and Shallow Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: Seepage Berm - 130 feet wide berm, 3 feet thick at berm toe and 5 feet thick at levee toe, berm soil type - silty sand
Shallow Cutoff Wall - 3 feet wide and 21.5 feet deep cutoff wall starting at Elevation 60 (half levee degrade)

Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.27	0.63	N/A	No Breakout	Yes	Seepage berm and shallow cutoff wall alternative meets criteria both at levee toe and berm toe.
200-year + 4 feet	0.34	0.70	N/A	No Breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.86				Yes	Seepage berm and shallow cutoff wall alternative meets criteria both at levee toe and berm toe.
200-year + 4 feet	1.81				Yes	

TABLE 5-9A: CHARACTERIZATION OF REACH 9
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
9	654+75 [2821+00]	706+50 [2872+75]	0.98	Total Number of Explorations = 17; Crest Explorations = 10 (SPT - 5, CPT - 5); Landside Toe Explorations = 6 (SPT - 4, CPT - 2); Landside Field Explorations = 1 (CPT - 1)	**Crown Width :** approximately 10 feet to 20 feet **Landside slope** Approximately 2H:1V or flatter than 2H:1V **Waterside Slope** Approximately 3H:1V to flatter than 3H:1V	• 655+57 [2821+72] to 279+10 [2845+25]: Waterside bank erosion encroaching on the levee section. (F.R. Site G) (Unknown Year). The site is located near Messick Road at approximate River Mile 21.2 (LM 7.3). • 677+72 [2843+88]: Scour on waterside berm of levee (1997) • 692+50 [2858+65]: This site is at the Boyd Pump Boat Ramp at LM 8, between Messick Road and Oswald Avenue at approximate RM 21.7. During the 1986 flood, portions of the boat ramp parking lot and subgrade and portions of the levee toe were eroded. (F.R. Site H) (1986) • 699+35 [2865+51]: Bank erosion at pump structure at waterside levee (1997) Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 1/4 th of Reach 9 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 1/2 of Reach 9 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations.	655+83 [2820+78] to 706+50 [2872+50]: In 1998, the Corps raised the levee 1-foot and installed a pervious toe drain and seepage/stability berm at the site under the Marysville/Yuba City Levee Reconstruction Project. (F.R. Site H). Site 9: Toe drain and berm from LM 7.28 to LM 8.26. Project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3 (Sites 1, 2, 3, 8, 9, 10, 11, & 12)," dated 7/4/1997. USACE Design File No. 50-04-6001.	R_{SL} (Recent Slough deposits) R_a (Recent Alluvium) R_{ofc} (Recent Overflow Channels) BP (Borrow Pit present in 1937) SP (Spoils present in 1937)	High resistivity soil layers present at approximately 10 ft below the embankment. The resistive layer extends to the ground surface occasionally.

REACH ID [1]	START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
9	654+75 [2821+00]	706+50 [2872+75]	**Levee Embankment**: Sand (SP, SM) and Silt (ML). Sand encountered in the entire levee embankment (from levee crown to levee toe) in several explorations. **Foundation**: Near Surface layers include Sand (SP-SM), Clay (CL), and Silt (ML). Thick porous layers are present in multiple layers.	**200 yr WSE** Head = approximately 17.6 feet at the analysis section	Mostly Agricultural. Scattered houses and Sheds observed from aerial maps.	705+84 [2872+00]	WL0001_010S, 2F-91-14 (CREST), WL0001_079C, WL0001_004B, 2F-91-14A (TOE), WL0001_111C (FIELD)	Sand and silty sand in levee embankment may indicate potential for through seepage; sandy foundation layer present near the surface indicate potential for shallow seepage (leaking layer); fine-grained layer below surface sandy layer create potential blanket condition for underseepage; deep sand/gravel layers also present.

TABLE 5-9B: ANALYSIS RESULTS FOR REACH 9
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 705+84

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.59	N/A	N/A	3.7	No	High breakout point, which is an indication of through seepage potential is due to sandy materials in the embankment.
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 9 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both throughseepage and underseepage potential. Alternative 2 for Reach 9 is seepage berm with shallow cutoff wall. Seepage berm material type is silty sand, which is similar to shallow near surface layers. The shallow cutoff wall reduces the potential for seepage through shallow near surface sandy layers.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 3 feet wide and 45.5 feet deep cutoff wall starting at Elevation 64.5 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.08	N/A	N/A	At the landside levee toe	Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	0.10	N/A	N/A	At the landside levee toe	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.37		Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	2.28		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm and Shallow Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: Seepage Berm - 110 feet wide berm, 3 feet thick at berm toe and 5 feet thick at levee toe, and berm soil type - silty sand
Shallow Cutoff Wall - 3 feet wide and 29.5 feet deep cutoff wall starting at Elevation 64.5 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.15	0.49	N/A	At the landside levee toe	Yes	Seepage berm and shallow cutoff wall alternative meets criteria both at levee toe and berm toe.
200-year + 4 feet	0.21	0.54	N/A	At the landside levee toe	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.45		Yes	Seepage berm and shallow cutoff wall alternative meets criteria both at levee toe and berm toe.
200-year + 4 feet	2.39		Yes	

TABLE 5-10A: CHARACTERIZATION OF REACH 10
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
10	706+50 [2872+75]	774+00 [2940+25]	1.28	Total Number of Explorations = 20; Crest Explorations = 11 (SPT - 6, CPT - 5); Landside Toe Explorations = 8 (SPT - 5, CPT - 3); Landside Field Explorations = 1 (CPT - 1)	**Crown Width :** approximately 18 feet to 20 feet **Landside Slope** approximately 2H:1V to flatter 2H:1V **Waterside Slope** approximately 3H:1V to flatter than 3H:1V	No documentation of past performance problems were found in the reviewed documents or site walk with LD1 representative. However, based on DWR's Seepage Area map for the Feather River West Levee area, approximately 7/10 th of Reach 10 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 3/5 th of Reach 10 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations.	No documentation of existing levee improvements were found in the reviewed documents or site walk with LD1 representative.	R_{ob} (Recent Overbank) H_a (Holocene Alluvium) H_{ofc} (Holocene Overflow Channels)	Approximately 60 feet thick, continuous, high resistivity soil layer is present at approximately 10 feet below the embankment from 706+65 [2872+65] to 739+92 [2906+00]. The high resistive layer is not observed beyond Sta. 739+92 [2906+00] to the end of the Reach.

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: Sand (SP, SP-SM, SC). Foundation: Near Surface layers include SM, CL, ML. Thick (~ 25 feet) pervious zones was encountered at about 25 feet below the embankment in crest explorations. Blanket thins at the toe explorations.	**200 yr WSE** Head = approximately 17.6 feet at the analysis section	Garden highway runs parallel along 1200 feet of this Reach.	733+84 [2900+00]	2F-91-15, CF-88-9, WL0001_057C, WL0001_058C (CREST), 2F-91-15A and SL0001_001B (LANDSIDE TOE)	Sandy layers in levee embankment may indicate potential for through seepage; Location coincide with high resistivity soils shown in HEM differential resistivity profiles; Moderately thick fine-grained blanket layer (about 30 feet thick) encountered in the explorations within this reach in the crest explorations, however blanket thins based on the landside toe explorations.

TABLE 5-10B: ANALYSIS RESULTS FOR REACH 10
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 733+84

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.86	N/A	N/A	5.8	No	High breakout point due to sandy materials in the embankment.
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 10 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both throughseepage and underseepage potential. Alternative 2 for Reach 10 is seepage berm with shallow cutoff wall. Seepage berm material type is silt, which is similar to shallow near surface layers. The shallow cutoff wall reduces the potential for seepage through shallow near surface sandy layers. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 3 feet wide and 68.5 feet deep cutoff wall starting at Elevation 64.9 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No Breakout	Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	<0.1	N/A	N/A	No Breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.99		Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	1.99		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm and Shallow Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: Seepage Berm - 300 feet wide berm, 3 feet thick at berm toe and 7.2 feet thick at levee toe, and berm soil type - silt
Shallow Cutoff Wall - 3 feet wide and 35 feet deep cutoff wall starting at Elevation 65 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.29	0.89	N/A	0	Meets criteria at levee toe. See Comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	0.38	1.0	N/A	0		

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.00		Yes	
200-year + 4 feet	1.99		Yes	

TABLE 5-11A: CHARACTERIZATION OF REACH 11
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
11	774+00 [2940+25]	830+00 [2996+25]	1.06	Total Number of Explorations = 15; Crest Explorations = 9 (SPT - 6, CPT - 3); Landside Toe Explorations = 4 (SPT - 2, CPT - 2); Landside Field Explorations = 2 (CPT - 2)	**Crown Width :** approximately 15 feet to 20 feet **Landside slope** approximately 2H:1V to flatter than 2H:1V **Waterside Slope** approximately 3H:1V to flatter than 3H:1V	812+50 [2978+75]: During 1997 flood, a boil developed around a tree. Clean water was coming out. Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 2/3 rd of Reach 11 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 2/3 rd of Reach 11 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations.	No documentation of existing levee improvements were found in the reviewed documents or site walk with LD1 representative.	R_{ob} (Recent Overbank) R_a (Recent Alluvium)	Approximately 50 feet thick, continuous, high resistivity soil layer is present immediately below the embankment in this Reach.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
11	**Levee Embankment**: SP, SM, ML, CL **Foundation**: Fine-grained blanket less than 10 feet thick underlain by approximately 10 to 30 feet thick pervious layers.	**200 yr WSE** Head = approximately 14.8 feet at the analysis section	Mostly agricultural with scattered residential houses. Mostly urban in the northern 700 feet (approximately) of the reach (Yuba City)	808+85 [2975+00]	2F-07-01 (CREST), SL0001_008C (TOE), CPT-11 (FIELD)	Thin fine-grained blanket encountered in the boring and CPT underlain by thick pervious layers may indicate potential for underseepage; Approximately 30 feet thick pervious zone (SP layer) below the fine-grained blanket was encountered in both crest and landside toe explorations approximately at the same elevations.

TABLE 5-11B: ANALYSIS RESULTS FOR REACH 11
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 808+85

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	1.38	N/A	N/A	8.4 feet above LS toe	No	High breakout point due to presence of silt layer in the levee.
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 11 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both throughseepage and underseepage potential. Alternative 2 for Reach 11 is seepage berm with drained stability berm. Seepage berm material type is sandy silt, which is similar to shallow near surface layers. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 3 feet wide and 73 feet deep cutoff wall starting at Elevation 67.5 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No Breakout	Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	<0.1	N/A	N/A	No Breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.56		Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	1.54		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm and Drained Stability Berm

Dimensions of Primary Features: Seepage Berm - 300 feet wide berm, 3 feet thick at berm toe and 7.2 feet thick at levee toe, berm soil type - sandy silt
Drained Stability Berm - 7.5 feet tall, 10 feet wide at top, 2 feet thick drain, and 3H:1V landside slope

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.34	1.41	N/A	0	Meets criteria at levee toe. See Comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	0.49	1.67	N/A	0		

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.19		Yes	
200-year + 4 feet	1.91		Yes	

TABLE 5-12A: CHARACTERIZATION OF REACH 12
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
12	830+00 [2996+25]	845+00 [3011+25]	0.28	Total Number of Explorations = 7; Crest Explorations = 5 (SPT - 3, CPT - 2); Waterside Field Explorations = 2 (SPT - 2)	**Crown Width :** approximately 20 feet **Landside slope** approximately 2H:1V to flatter 2H:1V **Waterside Slope** approximately 3H:1V to flatter than 3H:1V	834+39 [3009+39]: The site is located at Shanghai Bend at approximate River Mile 25.0 (LM 10.8). Seepage occurred at this site during the previous flood events. (F.R. Site I). 839+71 [3005+86] to 908+21 [3074+31]: The site is located between Shanghai Bend and Yuba City Airport between approximate River Miles 25.1 and 26.5 (LM 11.0 and 12.4). 842+22 [3009+39]: Seepage and a boil occurred just beyond the berm toe during the 1995 high water. (F.R. Site I) Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 12 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 12 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps.	829+85 [2996+00] to 846+20 [3012+50]: Shanghai Bend setback levee with cutoff wall. Additional site investigation report and Star Bend borrow site report for the Project "PL 84-99 Cost Shared, Basin No. 18, Feather River, Shanghai Bend, dated 1999. USACE Drawing File No. 4-4-620" were available during preparation of this report.	R_{ob} (Recent Overbank)	High resistivity soil layer is not present for most of this Reach. The high resistivity layer starts towards the northern end 843+61 [3010+00], immediately below the ground surface.
			GENERALIZED SUBSURFACE CONDITIONS [10]	**DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]**	**NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]**	**TRANSVERSE SECTION FOR EVALUATION [13]** SBFCA STATION [DWR ULE STATION]	**EXPLORATIONS FOR TRANSVERSE SECTION [14]**	**RATIONALE FOR SECTION SELECTION [15]**	
			Levee Embankment: ML, SC, CL **Foundation**: Shallow Sand layer (potentially cutoff by slurry wall). Deep thick sand layer present approximately 30 feet below embankment.	**200 yr WSE** Head = approximately 16.9 feet at the analysis section	Urban Area (Yuba City)	830+59 [2996+70]	WL0001_064B, WL0001_065C (CREST), SL0001_009C, (LANDSIDE TOE), CPT-11 (FIELD)	Recently built setback levee includes a shallow cutoff wall. Available explorations in the area indicate that the cutoff wall tip embedded within fine-grained soil (fully penetrating cutoff wall); Analysis to evaluate effect of the deep sand and gravel layers.	

TABLE 5-12B: ANALYSIS RESULTS FOR REACH 12
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 830+59

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	< 0.1	N/A	N/A	No Breakout	Yes	Shanghai Bend setback levee with cutoff wall. Available explorations indicate that the cutoff wall toe embedded in fine-grained soil layer.
200-year + 3 feet	< 0.1	N/A	N/A	No Breakout	Yes	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	1.78				Yes	Shanghai Bend setback levee with cutoff wall. Available explorations indicate that the cutoff wall toe embedded in fine-grained soil layer.
200-year + 3 feet	1.78				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-13A: CHARACTERIZATION OF REACH 13
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
13	845+00 [3011+25]	927+00 [3093+25]	1.55	Total Number of Explorations = 70; Crest Explorations = 19 (SPT - 10, CPT - 9); Landside Toe Explorations = 2 (SPT - 1, CPT - 1); Landside Field Explorations = 45 (SPT - 7, CPT - 38); Waterside Field Explorations = 4 (SPT - 4)	**Crown Width** approximately 15 feet to 20 feet **Landside Slope** 1.8 H:1V to flatter than 2H:1V **Waterside Slope** 2.5H:1V to flatter than 3H:1V	839+71 [3005+86] to 908+21 [3074+31]: The site is located between Shanghai Bend and Yuba City Airport between approximate River Miles 25.1 and 26.5 (LM 11.0 and 12.4). The levee broke in this area during the 1909, 1911, and 1955. (F.R. Site J). Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 13 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 13 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps.	845+71 [3012+00] to 926+51 [3092+65]: In 1957, the USACE reconstructed the levee to the LS of it's previous loc. and installed a row of relief wells near the LS levee toe. Water from the relief wells is pumped to the Feather River (FR). (F.R. Site J). Note: The actual LMs for this improv. meas. are from LM 11.3 to LM 12.8. Relief wells are spaced at 200 feet intervals. Plans are available in the doc. entitled "Emergency Levee Repairs, Relief Wells - Right Bank FR, Shanghai Bend, Sutter County, Ca, USACE File 4-4-435". 845+71 [3012+00] to 926+15 [3092+65]: In 1990, the City of Yuba City installed a seepage interceptor system in the southern part of this site. The interceptor system consists of a perforated pipeline and filter 12-18 feet below ground surface to extract shallow seepage, and relief wells placed between the 1957 relief wells to extract deeper seepage. All water collected is pumped into the Feather River separately from the water collected by the 1957 relief wells. (F.R. Site J). Note: According to Bill Hampton of LD1, this interceptor system cont. up to 3092+65.	R_{ch} (Recent Channels) Ra (Recent Alluvium) R_{ms} (Channel meander scroll deposits) W_{37} (Water)	Approximately 50 feet thick, continuous, high resistivity soil layer is present immediately below the embankment from 845+71 [3012+00] to 893+90 [3060+00]. Beyond 893+90 [3060+00], the layer is present 10 ft below the foundation.
						845+71 [3012+00] to 926+15 [3092+65]: During the 1986 flood, volunteers sandbagged several boils in this area. (F.R. Site J). Site 3: At the downstream of Shanghai Bend, an area of concentrated seepage produced boils during the 1986 high water. These boils were controlled with sack rings. Site 3: According to LD 1 Engineer Mr. Von Geldern, Relief wells in Shanghai Bend area produced 3 gallons per minute during the 1986 high water. Additional seepage occurred in fields adjacent to the levee. Seepage appeared in this area up to several hundred feet away from the levee toe (1986). 893+89 [3060+16] to 902+77 [3069+06]: Heavy seepage near landside toe of levee south of Burns Drive (1997)	845+71 [3012+00] to 926+15 [3092+65]: In 1993, and inspection of the shallow drain perforated pipeline discovered unacceptable deflections over large portions of the pipeline. The deformed plastic pipeline was removed and replaced with a perforated clay pipeline. (F.R. Site J). 829+44 [3058+59] to 926+50 [3092+65]: In 2001, the Corps of Engineers rehabilitated the original 1957 relief wells under a PL 84-99 contract. (F.R. Site J). Project plans entitled "PL 84-99 Cost Shared Added Sites Sacramento Basin No. 18 Relief Well Improvements," dated 7/31/2000. USACE Design File No. 4-04-625. **During the site visit in November 2010, relief wells at a spacing of 100 feet were observed from station 845+61 [3012+00] to 926+15 [3092+65]**		

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: Sand (SP, SP-SM, SM) **Foundation**: Near Surface Layers SM, ML, SC. Thick layers of pervious zones (sand and gravel) present at shallow and deep layers.	**200 yr WSE** Head = approximately 22.3 feet at the analysis section	Urban Area (Yuba City)	861+33 [3027+50]	WL0001_067B, WL0001_067C, B-40 (CREST); CPT 22, CPT 23, SL0001_002B (LANDSIDE TOE); B-33 and B-57 (LANDSIDE FIELD); B-51 and B-52 (WATERSIDE TOE AND FIELD)	Near surface layer consists of sand or sandy layers. Shallow silty sand and sandy silt layer overlying a thick porous zone may create a thin blanket condition and potential for underseepage. HEM differential resistivity profiles indicate presence of thick layer of high resistivity soils from ground surface; Sandy layers in levee embankment may indicate potential for through seepage. Levee embankment at this section have SP-SM materials.

TABLE 5-13B: ANALYSIS RESULTS FOR REACH 13
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 861+33

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	1.21	N/A	1.38	9.2 feet above LS toe	No	High breakout point due to presence of sand layers (<10% fine contents) in the levee embankment.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 13 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both throughseepage and underseepage potential. Alternative 2 for Reach 13 is relief wells with shallow cutoff wall.
Existing relief wells have 100 feet spacing and alternatively 20 feet (installed after 1955 flood) and 50 feet deep (installed in 1990s). Analysis indicated 100 feet wide spacing, however considering presence of existing deep (50 feet deep) relief wells, recommended spacing is 200 feet.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features:	3 feet wide and 97 feet deep cutoff wall starting at Elevation 68.8 (half levee degrade)					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No Breakout	Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	<0.1	N/A	N/A	No Breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.69				Yes	Fully penetrating (i.e. tip within fine-grained soil) cutoff wall reduces seepage potential.
200-year + 4 feet	1.68				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Shallow Soil-Bentonite Cutoff Wall and Relief Well						
Dimensions of Primary Features:	Shallow Cutoff Wall - 3 feet wide and 32 feet deep cutoff wall starting at Elevation 68.8 (half levee degrade) Relief Well - 200 feet Spacing and 65 feet Deep. Existing relief wells have 100 feet spacing and alternatively 20 feet (installed after 1955 flood) and 50 feet deep (installed in 1990s). Analysis indicated 100 feet wide spacing, however considering presence of existing deep (50 feet deep) relief wells, recommended spacing is 200 feet.					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No Breakout	Yes	Shallow cutoff wall and relief well system meet criteria
200-year + 4 feet	<0.1	N/A	N/A	No Breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.69				Yes	Shallow cutoff wall and relief well system meet criteria
200-year + 4 feet	1.69				Yes	

TABLE 5-14A: CHARACTERIZATION OF REACH 14
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
14	927+00 [3093+25]	954+40 [3120+25]	0.52	Total Number of Explorations = 10; Crest Explorations = 9 (SPT - 2, CPT - 2, TP - 5); Landside Toe Explorations = 1 (CPT - 1)	**Crown Width :** approximately 18 feet to 20 feet **Landside slope** approximately 2H:1V to flatter then 2H:1V **Waterside Slope** approximately 3H:1V to flatter than 3H:1V	No documentation of past performance problems were found in the reviewed documents or site walk with LD1 representative. However, based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 14 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 14 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps.	Site 8: Slurry walls from LM 12.76 to LM 13.28. Project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3 (Sites 1, 2, 3, 8, 9, 10, 11, & 12)," dated 7/4/1997. USACE Design File No. 50-04-6001.	R_a (Recent Alluvium)	Approximately 40 feet thick, continuous, high resistivity soil layer is present immediately below the embankment in this Reach.

REACH ID [1]	START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
14	927+00 [3093+25]	954+40 [3120+25]	**Levee Embankment**: Silty sand and silt (SM, ML) **Foundation**: Near Surface layer silty sand and silt (SM, ML). Thick layers of sand and silt sand (SP, SM) and Gravel (GP).	**200 yr WSE** Head = approximately 13.2 feet at the analysis section	Urban Area, Airport (Yuba City)	939+33 [3105+50]	2F-07-02, WL0001_073C (CREST) and SL0001_011C (LANDSIDE TOE)	Based on the design drawings, slurry wall tip in SP/SM layer (potential partially penetrating cutoff wall). Silty sand encountered at toe location at ground surface. However, based on information provided by Bill Hampton of LD1 during a November 2010 site visit the cutoff wall depth varied to embed tipin fine-grained soil. The model is developed using a deeper cutoff wall than depth shown on design drawings. However, specification requirements and as-built documents should be evaluated or excavation should be performed to investigate the depths and conditions of the cutoff walls in this Reach.

TABLE 5-14B: ANALYSIS RESULTS FOR REACH 14
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 939+33

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.07	N/A	0.3 (approximately 55 feet from landside toe)	No Breakout	Yes	The design drawings for this section indicates that the proposed cutoff wall depth is 40 feet. A sensitivity analysis using 40 feet cutoff wall depth did not meet seepage criteria for 200 year and 200+3 feet WSE as the tip of the cutoff wall was in coarse-grained soil layer. Based on discussion with Bill Hampton of LD1 during a site visit in November, 2010, the cutoff wall depths in this area were varied to embed the tip in fine-grained soil layers. Therefore, the analysis model was developed with an assumed cutoff wall depth of approximately 50 feet i.e. cutoff wall tip embedded in fine-grained soil. Specification requirement and as-built drawings should be evaluated or excavation should be performed to investigate the conditions of the cutoff wall. Moreover, a sensitivity study using a $K_h = 10^{-5}$ cm/sec in the top 15 feet of the cutoff wall (for 50 feet deep cutoff wall section) was performed to evaluate effect of potential cracking in the upper portion of the wall. The sensitivity analyses meet criteria.
200-year + 3 feet	0.16	N/A	0.4 (approximately 55 feet from landside toe)	No Breakout	Yes	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	2.0				Yes	
200-year + 3 feet	1.93				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-15A: CHARACTERIZATION OF REACH 15
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
15	954+40 [3120+25]	968+50 [3134+50]	0.27	Total Number of Explorations = 8; Crest Explorations = 3 (SPT - 2, CPT - 1); Landside Toe Explorations = 4 (SPT - 2, CPT - 2); Landside Field Explorations = 1 (CPT - 1)	**Crown Width :** approximately 15 feet to 20 feet **Landside slope** 1.5 H:1V to flatter than 2H:1V **Waterside Slope** 1.7H:1V to flatter than 3H:1V	No documentation of past performance problems were found in the reviewed documents or site walk with LD1 representative. However, based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 15 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 15 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps.	Site 8: Toe drain and berm from LM 12.26 to LM 13.57. Project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3 (Sites 1, 2, 3, 8, 9, 10, 11, & 12)," dated 7/4/1997. USACE Design File No. 50-04-6001. **NOTE: DURING A SITE VISIT ON NOVEMBER 2010, NO BERM WAS FOUND. ACCORDING TO BILL HAMPTON OF LD1, A CUTOFF WALL WAS CONSTRUCTED IN THIS AREA.**	R_a (Recent Alluvium) R_{cs} (Recent Crevasse Splay)	Approximately 40 feet thick, high resistivity soil layer is present immediately below the embankment from 953+80 [3120+00] to 958+80 [3125+00] in this Reach. Beyond this station resistive layer was not observed.

REACH ID [1]	START STATION	END STATION	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
15	954+40 [3120+25]	968+50 [3134+50]	**Levee Embankment**: Silty clay, silty sand, and silt (CL, SM, ML) **Foundation**: Near Surface layer consists of silty sand and silt (SM, ML). Sand and Gravel (SP, GM) are present at shallow at deep layers.	**200 yr WSE** Head = approximately 12.4 feet at the analysis section	Urban Area, Airport, Police Station, Baseball Field (Yuba City)	958+83 [3125+00]	WL0001_073C, WL0001_074B, 4F-88-7 (CREST), 2F-88-9A, WL0001_003B, WL0001_103C, WL0001_105C (TOE), and WL0001_108C (LANDSIDE FIELD)	Sand and silty sand in levee embankment may indicate potential for through seepage; Shallow and deep sand/gravel layers may indicate potential for underseepage.

TABLE 5-15B: ANALYSIS RESULTS FOR REACH 15
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 958+83

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.6	N/A	1.15 (at 25 feet from LS toe)	2.4	No	A toe berm and drain was designed for this section by USACE. However, no berm was found during a field visit in November 2010. According to Bill Hampton of LD 1, a cutoff wall was installed in this segment. As-built documents or verification study is needed to investigate the presence and conditions of the cutoff wall.
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year	19 feet drop from 200yr WSE	2.14	A drop of 19 feet was estimated from evaluation of 1986 and 1997 flood levels at gage station "YUB," which is located near Yuba city. The duration of drop considered was 7 days.	Yes	

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 15 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through seepage and underseepage potential. Alternative 2 for Reach 15 is a shallow cutoff wall and relief well system. The seepage berm alternative was not considered because this Reach is located near an urban area.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 3 feet wide and 64 feet deep cutoff wall starting at Elevation 73 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No Breakout	Yes	
200-year + 4 feet	<0.1	N/A	N/A	No Breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.09		Yes	
200-year + 4 feet	2.07		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Shallow Cutoff Wall and Relief Wells

Dimensions of Primary Features: Relief Wells: 23 feet spacing and 51.5 feet deep.
Shallow Cutoff Wall: 3 feet wide and 36 feet deep cutoff wall starting at Elevation 73 (half levee degrade)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot					Yes	Relief wells are modeled to meet an average exit gradient of 0.5 at the center of two relief wells. The head drop at the relief well location and between the relief well and midpoint between relief wells should be higher i.e. gradients should be lower than 0.5. The analyses was performed in accordance with USACE EM 1110-2-1914.
200-year + 4 feet					Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.9		Yes	
200-year + 4 feet	1.87		Yes	

TABLE 5-16A: CHARACTERIZATION OF REACH 16
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
16	968+50 [3134+50]	1080+00 [3246+25]	2.11	Total Number of Explorations = 48; Crest Explorations = 34 (SPT - 24, CPT - 10); Landside Toe Explorations = 4 (SPT - 3, CPT - 1); Landside Field Explorations = 7 (SPT - 7); Waterside Field Explorations = 4 (SPT - 3)	**Crown Width :** approximately 15 feet to 20 feet **Landside slope** 1.6 H:1V to flatter than 2H:1V **Waterside Slope** 1.4H:1V to flatter than 3H:1V	Based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 16 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 16 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps. 971+00 [3137+19] The site is located near the Yuba City Airport at approximate River Mile 27.0 (LM 13.6). Seepage occurs at the site during high water. (F.R. Site K). 991+94 [3158+16] to 1070+74 [3236+83]The site is located in Yuba City from Garden Highway north to the Drive-In Cinema between approximate River Miles 27.4 to 29.3 (LM 14.0 to LM 15.5). During the 1955 flood, seepage was observed near the 10th Street Bridge. (F.R. Site L). 991+94 [3158+16] to 1070+74 [3236+83]: During the 1986 flood, the landside slope became saturated and unstable and bulged slightly in the area of the Corporation Yard. Water also flowed up through cracks in the parking lot pavement and the floor slab of an auto body shop on Teegarden Avenue. (F.R. Site L).	Site 8: 927+08 [3093+19] to 968+31 [3170+25] Slurry walls from LM 12.76 to LM 13.28. Project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3 (Sites 1, 2, 3, 8, 9, 10, 11, & 12)," dated 7/4/1997. USACE Design File No. 50-04-6001. 1019+75 [3186+85] to 1080+25 [3246+40] with gap in HWY-20: Slurry wall from LM 14.56 to LM 15.68. Plans entitled "PL 84-99 Cost Shared/Added Sites Sacramento Basin No. 18 LD1 Slurry Wall, undated, USACE Design File No. 4-25-625." Note: Based on Doc. 3045 and 946, an approximately 135 feet gap between slurry walls from two projects. PL 84-99 calls for an overlap between the slurry walls.	R_a (Recent Alluvium) R_{cs} (Recent Crevasse Splay) H_{ch} (Holocene Channel) R_{ob} (Recent Overbank) R_{ch} (Recent Channel) (along almost the entire reach - Gilsizer Slough)	High Resistivity soil layers are present intermittently along the Reach, immediately below the embankment.
						991+96 [3158+16] to 1070+69 [3236+83]: Site 1: This is a reach of levee extending 4000 feet through downtown Yuba City. During the 1986 flood, this area showed considerable wetness on landside slope. (1986). 1041+99 [3208+07] to 1073+59 [3239+44]: During the 1997 flood, heavy seepage north of 10th Street Bridge. Heavy seepage near two City drainage pipes. 1005+95 [3172+25] to 1021+87 [3188+10]: During the 1986 flood, Erosion of the waterside levee toe occurred in the areas immediately upstream and downstream of the Fifth Street Bridge. (F.R. Site L). 1005+95 [3172+25] to 1021+87 [3188+10]: During the 1997 flood - this is an area of waterside erosion which extends for several hundred feet upstream and downstream of the Fifth Avenue Bridge (Site 2). This erosion occurred mostly low down on the slope and had cut into the levee section causing a steepening of the lower levee slope. Bank and levee erosion south of the 5th Street Bridge. Bank erosion between 5th Street Bridge and 10th Street Bridge (1997).	991+96 [3158+16] to 1025+84 [3224+00]: Yuba City constructed a berm along the landside toe as an emergency action. A permanent seepage/stability berm was later constructed from the Fifth Street Bridge extending northward to about 2500 feet beyond the 10th Street Bridge. An impermeable cutoff wall was also constructed in this area (See F.R. Site K). (F.R. Site L). 991+96 [3158+16] to 1025+84 [3224+00]: Temporary toe drain/berm: Site 1: In 1986, this area was upgraded by placing a landside berm on the slope with an internal drain protected by filter fabric which extends into the toe. Drainage from this system can be monitored at drain pipes.		

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: Silty sand and silt (SM, ML), thin layers of silty clay (CL) **Foundation**: Shallow layers of CL, SM, ML, SC. Sand layers are approximately 10 feet thick and interlayered with CL, ML.	**200 yr WSE** Head = approximately 17.1 feet at the analysis section 993+80 and approximately 21.3 feet at the analysis section 1043+88	Urban Area (Yuba City)	993+80 [3160+00] and 1043+88 [3210+00]	SECTION 993+80: 2F-07-03 (CREST), SL0001_012C (LANDSIDE TOE) SECTION 1043+88: WL0001_081B, WL0001_081C, DH-4C, DH-4A (CREST); DH-4D, DH-4B (LANDSIDE TOE)	Location of mapped old Gilsizer Slough. Location of existing slurry wall.

TABLE 5-16B: ANALYSIS RESULTS FOR REACH 16 (ANALYSIS SECTION 1]
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 993+80

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	<0.1	N/A	N/A	No Breakout	Yes	A sensitivity study using a $K_h = 10^{-5}$ cm/sec in the upper 20 feet of the cutoff wall was performed to evaluate effect of potential cracking in the upper portion of the wall. The sensitivity analyses meet criteria. Specification requirement and as-built drawings should be evaluated or excavation should be performed to investigate the conditions of the existing cutoff walls.
200-year + 3 feet	<0.1	N/A	N/A	No Breakout	Yes	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	1.48				Yes	
200-year + 3 feet	1.47				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year	19 feet drop from 200yr WSE	0.94	A drop of 19 feet was estimated from evaluation of 1986 and 1997 flood levels at gage station "YUB," which is located near Yuba city. The duration of drop considered was 7 days.		No	This section does not meet Rapid Drawdown criteria due to a steep waterside slope (~1.6H:1V).

Rationale for Selection of Two Alternatives for Analysis

Typical alternative section with 3H:1V waterside slope and clay embankment satisfies the criteria. Typical section includes 20 feet wide crest, and 3H:1V waterside slope.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-16C: ANALYSIS RESULTS FOR REACH 16 (ANALYSIS SECTION 2)
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1043+88

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	<0.1	N/A	0.4 (280 feet from LS toe)	No Breakout	Yes	A sensitivity study using a $K_h = 10^{-5}$ cm/sec in the upper 20 feet of the cutoff wall was performed to evaluate effect of potential cracking in the upper portion of the wall. The sensitivity analyses meet criteria. Specification requirement and as-built drawings should be evaluated or excavation should be performed to investigate the conditions of the existing cutoff walls.
200-year + 3 feet	<0.1	N/A	0.42 (280 feet from LS toe)	No Breakout	Yes	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	2.17				Yes	
200-year + 3 feet	2.14				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-17A: CHARACTERIZATION OF REACH 17
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
17	1080+00 [3235+00]	1130+86 [3297+00]	0.96	Total Number of Explorations = 17; Crest Explorations = 8 (SPT - 6, CPT - 2); Landside Toe Explorations = 2 (SPT - 1, CPT - 1); Landside Field Explorations = 5 (SPT - 4, CPT - 1); Waterside Toe Explorations = 1 (CPT - 1); Waterside Field Explorations = 1 (SPT - 1)	**Crown Width :** approximately 15 feet to 20 feet **Landside slope** approximately 1.8H:1V to flatter than 2H:1V **Waterside Slope** approximately 2.5H:1V to flatter than 3H:1V	No documentation of past performance problems were found in the reviewed documents or site walk with LD1 representative. However, based on DWR's Seepage Area map for the Feather River West Levee area, approximately 100% of Reach 17 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observations. Based on the same map, approximately 100% of Reach 17 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observations. However, the setback levee was placed after these maps.	No documentation of existing levee improvements were found in the reviewed documents or site walk with LD1 representative.	R_a (Recent Alluvium) R_{ob} (Recent Overbank) BP (Borrow Pit Present in 1937)	High resistivity soil layers are present immediately below the embankment, occasionally.

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: Silty clay, silty sand, and silt (CL, SM, ML) **Foundation**: Near Surface layer consists of silty sand and silt (SM, ML). Silty sand and silt (SM, ML) are present at shallow at deep layers.	**200 yr WSE** Head = approximately 17.7 feet at the analysis section	Urban Area (Yuba City)	1108+86 [3275+00]	2F-07-05 (CREST), WL0001_003C (WS TOE), WL0001_002C, WL0001_002B (LS TOE), AND WL0001_106C (LANDSIDE FIELD)	A swale or depression is present near the landside toe. Interbedding of ML and CL layers in the levee embankment and foundation. Boring 2F-07-05 encountered sand in foundation.

TABLE 5-17B: ANALYSIS RESULTS FOR REACH 17
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1108+86

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	1.13	N/A	N/A	5.9	No	
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 17 is soil-bentonite cutoff wall and fill-in of the landside ditch/swale, which was selected as an in-place alternative to mitigate both through seepage and underseepage potential. Alternative 2 for Reach 17 is shallow cutoff wall with relief well system. The seepage berm alternative was not considered because this Reach is located near an urban area.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall and fill-in ditch						
Dimensions of Primary Features:	3 feet wide and 39 feet deep cutoff wall starting at Elevation 74 (half levee degrade) Soil type of the ditch/swale fill in - sandy silt					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.46	0.29 (soil filled-in ditch)	N/A	3.0	Yes	Considering the cutoff wall and levee degrade and rebuild criteria, the section meets through seepage criteria.
200-year + 4 feet	0.58	0.33 (soil filled-in ditch)	N/A	4.1	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.43					
200-year + 4 feet	1.33					

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Relief Wells and Shallow Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features:	Relief wells - 45 feet spacing and 40 feet deep. Shallow Cutoff Wall - 3 feet wide and 16 feet deep cutoff wall starting at Elevation 74 (half levee degrade)					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot					Yes	Shallow cutoff wall and relief well system meet criteria. Relief wells are modeled to meet an average exit gradient of 0.5 at the center of two relief wells. The head drop at the relief well location and between the relief well and midpoint between relief wells should be higher i.e. gradients should be lower than 0.5. The analyses was performed in accordance with USACE EM 1110-2-1914.
200-year + 4 feet					Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.69				Yes	Shallow cutoff wall and relief well system meet criteria
200-year + 4 feet	1.69				Yes	

TABLE 5-18A : CHARACTERIZATION FOR REACH 18
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
18	1130+86 [3297+00]	1213+85 [3380+00]	1.6	Crest:8 Borings 7 CPTs Landside: 2 Borings 6 CPTs 2 Hand augers Waterside: Not available	Levee Height: 20 feet Crown Width: 22 feet Landside Slope: 2.0H:1V Waterside Slope: 3.0H:1V at analysis section	• Seepage reported during storms of late 1996 from 1135+00 to 1214+24 [3299+00 to 3380+44]. • Sinkholes observed on riverside toe of levee in Winter of 1966 from 1136+32 to 1153+14 [3302+46 to 3319+28], repaired in Spring of 1966. • Sinkhole in 1986 at 1173+78 [3339+92]. • Heavy seepage reported in 1997 from 1131+08 to 1199+60 [3297+21 to 3365+74]. • Based on DWR's Seepage Area map for the Feather River West Levee area, 20% of Reach 18 was mapped as seepage area as indicated on aerial photographs of 4-24-63 in conjunction with field observation. Based on the same map, 25% of Reach 18 was mapped as seepage area as indicated on aerial photographs of 2-10-65 in conjunction with field observation.	• Repair of sinkholes on riverside toe of levee in Spring 1966. Note: As-Constructed Drawings not available.	Rob (Recent Overbank) Qmu (Pleistocene Modesto Formation - Upper) Rofc (Recent Overflow Channel) Ra (Recent Alluvium) Rch (Recent Channel)	Shallow higher resistivity zone near downstream end of reach

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
18	• Levee: 19 to 20 feet of sandy silt to lean clay with minor silty sand. • Foundation: 2 to 20 feet of sandy silt to lean clay. • 4 to 27 feet of sand to silty sand with localized silt. • 10 to 40 feet of silty sand to fat clay with minor silty sand and clayey sand. • 3 to 16 feet of poorly-graded gravel to poorly-graded sand with lean clay. • 4 to 23 feet of silty sand to fat clay.	200 Year 13 feet	LS and WS orchards along most of reach, Agricultural buildings at 1179+86 [3346+00]	1138+86 [3305+00]	WL0009_001S (crest), WL0009_008C (crest), WL0009_005C (landside toe), WL0009_004C (landside toe), WL0009_003H (landside), WL009_007A (landside toe)	Thinner blanket over sand layer more susceptible to underseepage.

TABLE 5-18B ANALYSIS RESULTS FOR REACH 18
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1138+86

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	1.32	N/A	N/A	8.0	No	The reach has reported seepage deficiencies including seepage reported in 1965 and 1996, and heavy seepage reported in 1997.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 18 is undrained seepage berm with relief wells. Seepage berm material type is silt having similar hydraulic conductivity as the clay blanket. Relief wells and/or a drainage system could be used to reduce high gradient. Alternative 2 for Reach 18 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate underseepage potential. The cutoff wall will block the flow through the existing thick aquifer, and effectively control the underseepage.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with Relief Wells						
Dimensions of Primary Features: 300 feet wide seepage berm, seepage berm thickness at the levee toe = 7.0 feet, thickness at the berm toe = 3 feet; Relief wells: depth = 30 feet and spacing = 100 feet.						
Seepage						
WSE	Exit Gradient Levee Toe	Berm (near berm toe)	Berm Toe	Phreatic Surface Breakout Point above Berm	Meets Criteria?	Comments
200-year + 1 foot	0.38	0.69	1.31	3.2	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Levee prism mainly consists of clayey soils; Drainage berm not required for through seepage.
200-year + 4 feet	0.55	0.89	1.58	6.0	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Levee prism mainly consists of clayey soils; Drainage berm not required for through seepage.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.59				Yes	
200-year + 4 feet	1.23				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Cutoff Wall						
Dimensions of Primary Features: 1130+86 to 1149+50 Cutoff Wall Tip Elevation 0, 1149+50 to 1190+00 Cutoff Wall Tip Elevation 30, 1190+00 to 1213+85 Cutoff Wall Tip Elevation 40.						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis performed; Alternative 2 of Reach 19 was used in the evaluation; for details see Figures B19-A5 through B19-A8.
200-year + 4 feet					Yes	No separate analysis performed; Alternative 2 of Reach 19 was used in the evaluation; for details see Figures B19-A5 through B19-A8.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis performed; Alternative 2 of Reach 19 was used in the evaluation; for details see Figures B19-A5 through B19-A8.
200-year + 4 feet					Yes	No separate analysis performed; Alternative 2 of Reach 19 was used in the evaluation; for details see Figures B19-A5 through B19-A8.

TABLE 5-19A : CHARACTERIZATION FOR REACH 19
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
19	1213+85 [3380+00]	1297+83 [3464+00]	1.6	Crest:14 Borings 4 CPTs Landside: 5 Borings 6 CPTs 1 Hand auger Waterside: 1 Boring	Levee Height: 20 feet Crown Width: 18 feet Landside Slope: 2.1H:1V Waterside Slope: 2.9H:1V at analysis section	• Boil and piping observed on landside of levee at 1242+86 [3409+01], landward half of levee excavated and recompacted with gravel filter in 1955. • Heavy levee saturation reported by Levee District 9 from 1259+63 to 1267+66 [3425+73 to 3433+88] (year unknown, awaiting information). • Pinhole boils observed on landside of levee in 1997 from 1288+24 to 1296+34 [3454+40 to 3462+52]. • Heavy seepage and boils on landside at 1278+82 to 1293+07 [3444+99 to 3459+25] in 1997 with river near 3 feet freeboard.	• Landward half of levee excavated and recompacted with gravel filter at 1242+86 [3409+01] in 1955. • Bank protection using rock was placed at 1266+45 to 1269+18 [3432+60 to 3435+35] in 1974. • Levee rock slope protection place at 1271+00 [3437+17] in 1974 • Drained toe berm repair was proposed near 1293+82 [3460+00] in 1998. Note: As-Constructed Drawings not available.	Ra (Recent Alluvium) Rofc (Recent Overflow Channel) Rob (Recent Overbank) Rcs (Recent Crevasse Splay) Qmu (Pleistocene Modesto Formation - Upper)	Shallow higher resistivity zone upstream of 1228+85 [3395+00]. Higher resistivity zone thickest from 1228+85 to 1238+85 [3395+00 to 3405+00].

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
• Levee: 18 to 20 feet of sandy silt to lean clay with minor well-graded gravel and clayey sand lenses. • Foundation: 14 to 46 feet of silty sand to fat clay with minor well-graded sand with silt to clayey sand. • 17 to 63 feet of poorly-graded gravel to poorly-graded sand with silt with minor silty sand and clayey sand. • 6 to 58 feet of silty sand to fat clay.	200 Year 13 feet	LS and WS orchards along much of reach, River at levee toe near 1267+84 [3434+00]	1238+85 [3405+00]	WL009_003S (crest), 2F-88-7 (crest), WL009_016C (landside toe), WL009_009A (landside toe), WL009_014C (landside toe), WL009_008A (landside toe)	Thin blanket over sand layer with underlying gravel susceptible to underseepage, near 1955 seepage area

TABLE 5-19B ANALYSIS RESULTS FOR REACH 19
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1238+85

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	2.32	N/A	N/A	7.0	No	The reach has reported seepage deficiencies including heavy seepage and boils reported in 1997.
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 19 is undrained seepage berm with relief wells and drained stability berm. Seepage berm material type is silt having similar hydraulic conductivity as the clay blanket. Relief wells provided at the berm toe to reduce high gradient. The drained stability berm is to control through seepage. Alternative 2 for Reach 19 is soil-bentonite cutoff wall, which was selected as an in-place alternative to effectively mitigate both through seepage and underseepage.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with Relief Wells, and Drained Stability Berm

Dimensions of Primary Features: 300 feet wide seepage berm, thickness at the levee toe = 7.0 feet, thickness at the berm toe = 3 feet; Relief wells: depth = 50 feet and spacing = 100 feet; stability berm height = 10 feet above seepage berm.

Seepage

WSE	Exit Gradient Levee Toe	Berm (near berm toe)	Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.36	0.76	2.13	No breakout	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Drainage stability berm controls through seepage.
200-year + 4 feet	0.57	1.03	2.59	No breakout	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Drainage stability berm controls through seepage.

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.56		Yes	
200-year + 4 feet	2.15		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Cutoff Wall

Dimensions of Primary Features: 1213+85 to 1224+00 Cutoff Wall Tip Elevation 40, 1224+00 to 1240+00 Cutoff Wall Tip Elevation -27 , 1240+00 to 1269+00 Cutoff Wall Tip Elevation 5, 1269+00 to 1297+83 Cutoff Wall Tip Elevation 35.

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	< 0.05	N/A	N/A	No breakout	Yes	
200-year + 4 feet	< 0.05	N/A	N/A	No breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.88		Yes	
200-year + 4 feet	1.86		Yes	

TABLE 5-20A : CHARACTERIZATION FOR REACH 20
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
20	1297+83 [3464+00]	1374+33 [3540+50]	1.4	Crest:9 Borings 9 CPTs Landside: 2 Borings 8 CPTs Waterside: 1 CPT	Levee Height: 16 feet Crown Width: 15 feet Landside Slope: 2.1H:1V Waterside Slope: 3.3H:1V at analysis section	• Heavy levee saturation reported by Levee District 9 from 1316+46 to 1320+70 [3482+64 to 3486+88] (year unknown). • Boil and piping observed on landside of levee at 1321+07 [3487+25], landward half of levee excavated and recompacted with fill in 1955. • Seepage and boil observed on landside of levee at 1328+73 [3495+06] in 1986. • Heavy seepage and boils about 20 feet from levee toe in 1997 at 1368+41 to 1410+63 [3534+58 to 3576+80] • Seepage and boils occur near the landside levee toe during high water at 1336+73 to 1410+70 [3502+94 to 3576+80]	• Landward half of levee excavated and recompacted with fill in 1955 at 1321+07 [3487+25].	Rcs (Recent Crevasse Splay) Rofc (Recent Overflow Channel) Qmu (Pleistocene Modesto Formation - Upper) Rob (Recent Overbank)	Shallow higher resistivity zone upstream of 1348+83 [3515+00].

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
20	• Levee: 12 to 18 feet of silty sand to lean clay. • Foundation: 4 to 13 feet of sandy lean clay to lean clay. • 1 to 10 feet of poorly-graded sand with silt to silty sand. • 20 to 43 feet of sandy silt to fat clay with minor poorly-graded sand with silt and silty sand. • 4 to 30 feet of poorly-graded sand with silt to silty sand with minor clayey sand to silt. • 44 to 74 feet of sandy silt to lean clay with minor silty sand and clayey sand.	200 Year 10 feet	Orchards in portions of reach, Agricultural buildings at 1348+33 [3514+50]	1338+83 [3505+00]	WL009_025C (crest), WL009_003A (landside toe), WL009_030C (200 ft landside)	10 feet of silty clayey sand (SC-SM) at toe overlying poorly graded sand with silt (SP-SM) susceptible to underseepage

TABLE 5-20B ANALYSIS RESULTS FOR REACH 20
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1338+83

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.72	NA	NA	3.0	No	The reach has reported seepage deficiencies including heavy seepage and boils reported in 1986 and 1997, and seepage and boils reported during high water.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 20 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through seepage and underseepage potential. Alternative 2 for Reach 20 is undrained seepage berm with drained stability berm on Top. Seepage berm material type is SC-SM, which is similar to shallow near surface layer. The drained stability berm is to control through seepage.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Cutoff Wall						
Dimensions of Primary Features: 1297+83 to 1359+00 Cutoff Wall Tip Elevation 50, 1359+00 to 1369+00 Cutoff Wall Tip Elevation 40, 1369+00 to 1374+33 Cutoff Wall Tip Elevation 50.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	< 0.05	N/A	N/A	No breakout	Yes	
200-year + 4 feet	< 0.05	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.64				Yes	
200-year + 4 feet	1.64				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with Drained Stability Berm on Top						
Dimensions of Primary Features: 70 feet wide seepage berm, seepage berm thickness at the levee toe = 5 feet, thickness at the berm toe = 3 feet; 1297+83 to 1309+00 stability berm height = 8.5 feet above seepage berm, 1320+00 to 1374+33 stability berm height = 8.5 feet above seepage berm.						
Seepage						
WSE	Exit Gradient Levee Toe	Berm (near berm toe)	Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.14	0.09	0.65	No breakout	Yes	
200-year + 4 feet	0.30	0.20	0.79	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.09				Yes	
200-year + 4 feet	1.90				Yes	

TABLE 5-21A : CHARACTERIZATION FOR REACH 21
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	 END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
21	1374+33 [3540+50]	1433+83 [3600+00]	1.1	Crest:5 Borings 9 CPTs Landside: 4 Borings 5 CPTs 2 Hand augers Waterside: 3 Borings	Levee Height: 15 feet Crown Width: 20 feet Landside Slope: 2.1H:1V Waterside Slope: 2.5H:1V at analysis section	• "Seasonal seepage" noted in 1957 by Levee District 9 from 1375+34 to 1390+17 [3541+54 to 3556+37]. • Boil observed on landside of levee in 1986 at 1401+58 [3567+75]. • Landside embankment distress in 1997. • Heavy seepage and boils in 1997 at [3444+99 to 3559+25] • Heavy seepage and sloughing in 1997 at [3584+13 to 3595+20] • Heavy seepage and boils in 1997 at [3534+58 to 3576+80] • Boils in 1955 at 1385+01 [3551+17] • Seepage and boils occur near the landside levee toe during high water at 1336+73 to 1410+70 [3502+94 to 3576+80]	• Corps plans indicate relocation of an open drainage ditch away from the toe of the levee. Plans call for 5374 feet of landside irrigation ditch back-filled from1375+58 to 1429+32 [3541+75 to 3595+49] (Site 3, completion unknown - awaiting construction documents, USACE Plans dated July, 1997). • Relief well on the landward side of the levee at 1385+83 [3552+00]. • Backfilling of irrigation canal with crushed rock and gravel in 1997 as emergency measure at 1417+95 to 1429+03 [3584+13 to 3595+20]. Note: As-Constructed Drawings not available. Canal observed in field at the landside toe of the levee in November 2010.	Ra (Recent Alluvium) Rch (Recent Channel) Qmu (Pleistocene Modesto Formation - Upper) Rcs (Recent Crevasse Splay) Qml (Pleistocene Modesto Formation - Lower)	Shallow higher resistivity in most of reach

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
• Levee: 10 to 12 feet of silty sand to lean clay with minor silty gravel. • Foundation: 9 to 14 feet of silty sand to lean clay. • 10 to 24 feet of well-graded sand to poorly-graded sand with minor silty and lean clay. • 23 to 36 feet of silty sand to fat clay.	200 Year 10 feet	Orchards on landside, unlined landside ditches	1378+83 [3545+00]	WL009_28C (crest), WL009_006A (landside toe), WL009_004H (260 ft landside), B-3 (crest)	8.5 feet of clay (CL) and silt (ML) at toe over silty sand (SM) and poorly graded sand with silt (SP-SM) susceptible to underseepage

TABLE 5-21B ANALYSIS RESULTS FOR REACH 21
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1378+83

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.17	2.79	1.0	NA	No	The reach has reported seepage deficiencies including heavy seepage and boils reported in 1986 and 1997, and seepage and boils reported during high water.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 21 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through seepage and underseepage potential. Alternative 2 for Reach 21 is undrained seepage berm with relief wells and drained stability berm. Seepage berm material type is silt, which is similar to shallow near surface layer. The drained stability berm is to control through seepage.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Cutoff Wall						
Dimensions of Primary Features: 1374+33 to 1379+00 Cutoff Wall Tip Elevation 40, 1379+00 to 1389+00 Cutoff Wall Tip Elevation 50, 1389+00 to 1409+00 Cutoff Wall Tip Elevation 60, 1409+00 to 1433+83 Cutoff Wall Tip Elevation 40.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	0.46	N/A	No breakout	Yes	
200-year + 4 feet	No exit of water	0.46	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.33				Yes	
200-year + 4 feet	2.33				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with Relief Wells, and Drained Stability Berm						
Dimensions of Primary Features: 300 feet wide seepage berm, seepage berm thickness at the levee toe = 6-1/2 feet, thickness at the berm toe = 3 feet; Relief wells: depth = 20 feet and spacing = 100 feet; stability berm height = 8 feet above seepage berm.						
Seepage						
WSE	Exit Gradient Levee Toe	Berm (near berm toe)	Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.12	0.23	0.96	No breakout	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Drainage stability berm controls through seepage.
200-year + 4 feet	0.32	0.36	1.18	No breakout	No at berm toe; will meet with Relief Wells	The calculated gradient at the berm toe does not meet criteria; Need to provide relief wells to reduce the high gradient at the berm toe; Drainage stability berm controls through seepage.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.88				Yes	
200-year + 4 feet	2.66				Yes	

TABLE 5-22A : CHARACTERIZATION FOR REACH 22
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
22	1433+83 [3600+00]	1503+83 [3670+00]	1.3	Crest:4 Borings 8 CPTs Landside: 1 Boring 2 CPTs Waterside: Not available	Levee Height: 12 feet Crown Width: 15 feet Landside Slope: 2.0H:1V Waterside Slope: 2.7H:1V at analysis section	Severe animal burrows in levee at 1433+83 to 1459+77 [3600+00 to 3626+00]; extremely severe burrowing at north end of LD-9 observed in 2010	• No known mitigation measures.	Qml (Pleistocene Modesto Formation - Lower) Rob (Recent Overbank) Ra (Recent Alluvium) Rch (Recent Channel) Rcs (Recent Crevasse Splay) Rofc (Recent Overflow Channel)	Shallow higher resistivity in most of reach
			GENERALIZED SUBSURFACE CONDITIONS [10]	**DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]**	**NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]**	**TRANSVERSE SECTION FOR EVALUATION [13]** SBFCA STATION [DWR ULE STATION]	**EXPLORATIONS FOR TRANSVERSE SECTION [14]**	**RATIONALE FOR SECTION SELECTION [15]**	
			• Levee: 7 to 10 feet of silty sand to lean clay. • Foundation: 0 to 15 feet of silt to lean clay. • 5 to 15 feet of poorly-graded sand with silt to silty sand. • 10 to 15 feet of sandy silt to lean clay. • 25 to 50 feet of silty sand to lean clay.	200 Year 7 feet 10 feet in ditch	Orchards on landside, unlined landside ditch	1468+83 [3635+00]	WM0016_001A (crest), WM0016_008C (crest), WM0016011C (landside toe)	Thin blanket over thick sand layer susceptible to underseepage	

TABLE 5-22B: ANALYSIS RESULTS FOR REACH 22
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1468+83

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.27	1.34	NA	1.5	No	The reach has extremely severe burrowing at north end of LD-9 observed in 2010; No other reported deficiencies have been identified; For empty ditch condition, the gradient at the bottom of the ditch did not meet criteria.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 22 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both throughseepage and underseepage potential. Alternative 2 for Reach 22 is fill canal and drained stability berm/levee reconstruction. Fill canal option would mitigate the estimated high gradient at the ditch. The drained stability berm/levee reconstruction is to control through seepage.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Cutoff Wall						
Dimensions of Primary Features: 1433+83 to 1449+00 Cutoff Wall Tip Elevation 40, 1449+00 to 1469+00 Cutoff Wall Tip Elevation 50, 1469+00 to 1503+83 Cutoff Wall Tip Elevation 55 plus full degrade/reconstruct from 1433+83 to 1459+77 due to severe burrowing.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	0.18	NA	No breakout	Yes	
200-year + 4 feet	No exit of water	0.24	NA	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.23				Yes	
200-year + 4 feet	2.23				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Fill Canal and Drained Stability Berm						
Dimensions of Primary Features: Fill canal to top (adj grade); stability berm from sta 1459+77 to 1503+83, height = 10 feet; full degrade/reconstruct from sta 1433+83 to 1459+77 due to severe burrowing.						
Seepage						
WSE	Exit Gradient Levee Toe	Filled Ditch	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.34	0.30	N/A	No breakout	Yes	
200-year + 4 feet	0.56	0.53	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.15				Yes	
200-year + 4 feet	1.79				Yes	

TABLE 5-23A : CHARACTERIZATION FOR REACH 23
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
23	1503+83 [3670+00]	1609.37 [3775+50]	2.0	Crest:7 Borings 7 CPTs Landside: 1 Boring 1 CPT Waterside: Not available	Levee Height: 11 feet Crown Width: 18 feet Landside Slope: 2.2H:1V Waterside Slope: 3.0H:1V at analysis section	• "Levee erosion" noted by citizens at 1521+05 to 1532+55 [3687+24 to 3698+82] in 1991.	• Landside drainage ditch constructed by USACE from 1521+03 to 1532+83 [3687+20 to 3699+00] in 1968. • Grouted rip-rap placed from 1515+63 to 1527+32 [3681+80 to 3693+49] by USACE in 1968 adjacent to levee and in the river channel for bank protection. • Bank protection placed at 1516+02 to 1534+67 [3682+19 to 3700+84] in 1965. Note: As-Constructed Drawings not available.	Qml (Pleistocene Modesto Formation - Lower) Rofc (Recent Overflow Channel) Rob (Recent Overbank) Rcs (Recent Crevasse Splay) Rch (Recent Channel)	Shallow higher resistivity in most of the reach
			GENERALIZED SUBSURFACE CONDITIONS [10]	**DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]**	**NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]**	**TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]**	**EXPLORATIONS FOR TRANSVERSE SECTION [14]**	**RATIONALE FOR SECTION SELECTION [15]**	
			• Levee: 7 to 10 feet of silty sand to sandy lean clay. • Foundation: 0 to 25 feet of sandy silt to lean clay. • 5 to 60 feet of poorly-graded sand to silty and clayey sand with significant localized silt to clay.	200 Year 4 feet 10 feet in ditch @ ~250 ft LS of toe	Orchards on both sides, unlined ditch about 200 to 600 feet landside of levee, buildings near 1558+83 [3725+00]	1508+33 [3674+50]	WM0016_011B (crest) SM0016_002C (landside)	Silt to clay levee may be susceptible to through seepage; Thin blanket over sand layer susceptible to underseepage.	

TABLE 5-23B: ANALYSIS RESULTS FOR REACH 23
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1508+33

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.32	NA	NA	0.9	Yes	
200-year	0.60	NA	NA	1.8	No	No reported seepage or landside stability deficiencies have been identified; However, the reach has thin clay blanket underlain by shallow sand aquifer (high potential for underseepage); Sand aquifer is exposed on the riverside.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
100-year	2.05				Yes	
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 23 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate underseepage potential. Alternative 2 for Reach 23 is undrained seepage berm. Seepage berm material type is silt, which has similar hydraulic conductivity as the shallow near surface clay blanket layer. Seepage analysis indicate high exit gradients at the toe of the seepage berm at HTOL WSE. Therefore, monitoring at the toe of the berm for seepage is recommended during the hig water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Cutoff Wall						
Dimensions of Primary Features: 1503+83 to 1509+00 Cutoff Wall Tip Elevation 55, 1509+00 to 1529+00 Cutoff Wall Tip Elevation 60, 1529+00 to 1566+00 Cutoff Wall Tip Elevation 55, 1566+00 to 1589+00 Cutoff Wall Tip Elevation 60, 1589+00 to 1609+37 Cutoff Wall Tip Elevation 40.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	< 0.05	N/A	N/A	No breakout	Yes	
200-year + 4 feet	< 0.05	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.25				Yes	
200-year + 4 feet	2.23				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm						
Dimensions of Primary Features: 100 feet wide seepage berm, seepage berm thickness at the levee toe = 5 feet, thickness at the berm toe = 3 feet.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Berm Toe	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	0.52	No breakout	Yes	As the berm width is 100 feet and the exit gradient meets criteria at DWSE, judgment was used for HTOL. Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	0.20	N/A	0.89	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.83				Yes	
200-year + 4 feet	1.47				Yes	

TABLE 5-24A : CHARACTERIZATION FOR REACH 24
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">24</td><td rowspan="4">1609.37
[3775+50]</td><td rowspan="4">1623+86
[3790+00]</td><td>0.3</td><td>Crest:1 Boring
1 CPTs
Landside: 1 CPT
Waterside: Not available</td><td>Levee Height: 9 feet
Crown Width: 20 feet
Landside Slope: 3.2H:1V
Waterside Slope: 2.7H:1V
at analysis section</td><td>• No reported deficiencies have been identified.</td><td>• No known mitigation measures.</td><td>Rch (Recent Channel)
Rob (Recent Overbank)
Rcs (Recent Crevasse Splay)
Qml (Pleistocene Modesto Formation - Lower)</td><td>Shallow higher resistivity zone</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>• Levee: 7 to 10 feet sandy silt to lean silty clay.
• Foundation: 5 to 10 feet of sandy lean clay with silt.
• 10 to 15 feet of well-graded with silt.
• 5 to 10 feet of sandy silt to lean clay.
• 20 to 25 feet of clayey sand to silty sand.</td><td>200 Year
2 feet
7 feet in ditch</td><td>LS and WS orchards along most of reach, building at 1610+36 [3776+50]</td><td>1615+62
[3781+75]</td><td>WL0016_020B (crest)
WL0016_006C (landside)</td><td colspan="2">Silt to clay levee; waterside levee toe drops to low elevation; shallow SW-SM acquifer exposed at the bottom of the ditch; wide landside toe ditch, susceptible to through seepage and underseepage</td></tr>
</table>

TABLE 5-24B: ANALYSIS RESULTS FOR REACH 24
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1615+62

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.20	4.14	NA	NA	No	No reported seepage or landside stability deficiencies have been identified; For empty ditch condition, the gradient at the bottom of the ditch did not meet criteria.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 24 is fill canal. Fill canal option would mitigate the estimated high gradient at the ditch. Alternative 2 for Reach 24 is cutoff wall plus excavate and place 4.5-foot thick compacted clay fill at the bottom of ditch. The cutoff wall and clayey fill below the bottom of the canal will significantly reduce the flow into the canal and thus reduce the exit gradient at the canal bottom.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Fill Canal						
Dimensions of Primary Features: Fill canal to top (elev 90ft, 40ft wide x 10ft deep).						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	0.38	No breakout	Yes	
200-year + 4 feet	0.27	N/A	0.69	1.0	Yes	Levee prism mainly consists of clayey soils; Drainage berm not required for through seepage.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.95				Yes	
200-year + 4 feet	2.21				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): 1609+37 to 1623+86 Cutoff Wall Tip Elevation 64, plus excavate and place 4.5-foot thick compacted clay fill at the bottom of ditch.						
Dimensions of Primary Features: 30ft-deep cutoff wall.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	0.44	N/A	No breakout	Yes	
200-year + 4 feet	No exit of water	0.58	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.82				Yes	
200-year + 4 feet	1.70				Yes	

TABLE 5-25A : CHARACTERIZATION FOR REACH 25
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
25	1623+86 [3790+00]	1674+37 [3840+50]	1.0	Crest:4 Borings 2 CPTs Landside: Not available Waterside: Not available	Levee Height: 9 feet Crown Width: 23 feet Landside Slope: 1.6H:1V Waterside Slope: 2.5H:1V at analysis section	• No reported deficiencies have been identified.	• No known mitigation measures.	Qml (Pleistocene Modesto Formation - Lower) Rch (Recent Channel) Rcs (Recent Crevasse Splay)	Shallow higher resistivity zone

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
25	• Levee: 5 to 10 feet of sandy silt to lean clay. • Foundation: 10 to 20 feet of sandy silt to lean clay with localized clayey sand. • 0 to 20 feet of poorly-graded sand with silt. • 10 to 20 feet of clayey sand to clayey silty sand. • 0 to 15 feet of well-graded gravel with silt to silty sand.	200 Year 1 foot	Orchards on both LS and WS, building at 1664+37 [3830+50]	1645+86 [3812+00]	WL0016_022S (crest) WL0016_021B (crest)	Silt to clay levee; waterside levee toe drops to low elevation more susceptible to underseepage

TABLE 5-25B: ANALYSIS RESULTS FOR REACH 25
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1645+86

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.15	NA	NA	No breakout	Yes	No reported deficiencies have been identified.
200-year + 3 feet	0.43	NA	NA	1.6	Yes	No reported deficiencies have been identified.; Portion of levee prism below the water surface elevation mainly consists of clay; Drainage berm not required for through seepage.
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	2.25				Yes	No reported deficiencies have been identified.
200-year + 3 feet	1.93				Yes	No reported deficiencies have been identified.
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
N/A

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE					Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-26A : CHARACTERIZATION FOR REACH 26
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
26	1674+37 [3840+50]	1707+11 [3873+25]	0.6	Crest:1 Boring 3 CPTs Landside: 1 CPT Waterside: Not available	Levee Height: 7 feet Crown Width: 19 feet Landside Slope: 1.5H:1V Waterside Slope: 2.6H:1V at analysis section	• Waterside seepage observed during irrigation season from 1677+84 to 1721+90 [3843+98 to 3888+07]. • Sloughing and oversteepening of canal slope at landside toe of levee observed in 2010 at 1689+87 [3856+00]	• No known mitigation measures.	Rob (Recent Overbank) Ra (Recent Alluvium) Rcs (Recent Crevasse Splay) Rch (Recent Channel) Qml (Pleistocene Modesto Formation - Lower)	Shallow higher resistivity zone

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
• Levee: 5 to 8 feet of sandy silt to lean clay. • Foundation: 5 to 20 feet of sandy silt to lean clay. • 5 to 20 feet of poorly-graded sand to clayey silty sand with localized silt to clay. • 5 to 15 feet of gravel with silt to silty sand.	200 Year 4 feet in ditch	Orchards on both LS and WS, Unlined landside ditch	1698+85 [3865+00]	WM0007_018C (crest)	Silt to clay levee, shallow SP-SM acquifer exposed at the bottom of the ditch; wide landside toe ditch susceptible to underseepage

TABLE 5-26B: ANALYSIS RESULTS FOR REACH 26
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1698+85

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	No exit of water	N/A	N/A	NA	No	Waterside seepage observed during irrigation season; Sloughing and oversteepening of canal slope at landside toe of levee observed in 2010; Based on interpreted soil profile, the section has shallow foundation sand layer which leads to through seepage into the canal.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 26 is fill canal, which would mitigate the through seepage into the canal. Alternative 2 for Reach 26 is cutoff wall and reconstruct the landside slope. The cutoff wall is to block the through seepage into the canal. The reconstruction of the landside slope is to mitigate the steep slopes observed along the canal and thus improve slope instability.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Fill Canal						
Dimensions of Primary Features: Fill canal to top (adj grade, 30ft wide x 5ft deep).						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis was performed; Alternative 1 of Reach 27 was used for evaluation; for details see Figures B27-A1 through B27-A4.
200-year + 4 feet					Yes	No separate analysis was performed; Alternative 1 of Reach 27 was used for evaluation; for details see Figures B27-A1 through B27-A4.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis was performed; Alternative 1 of Reach 27 was used for evaluation; for details see Figures B27-A1 through B27-A4.
200-year + 4 feet					Yes	No separate analysis was performed; Alternative 1 of Reach 27 was used for evaluation; for details see Figures B27-A1 through B27-A4.

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Cutoff Wall and Reconstruct the Landside Slope						
Dimensions of Primary Features: 1674+37 to 1686+00 Cutoff Wall Tip Elevation 75, 1686+00 to 1707+11 Cutoff Wall Tip Elevation 65; plus Reconstruct the Landside Slope						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis was performed; Alternative 2 of Reach 27 was used for evaluation; for details see Figures B27-A5 through B27-A8.
200-year + 4 feet					Yes	No separate analysis was performed; Alternative 2 of Reach 27 was used for evaluation; for details see Figures B27-A5 through B27-A8.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot					Yes	No separate analysis was performed; Alternative 2 of Reach 27 was used for evaluation; for details see Figures B27-A5 through B27-A8.
200-year + 4 feet					Yes	No separate analysis was performed; Alternative 2 of Reach 27 was used for evaluation; for details see Figures B28-A5 through B27-A7.

TABLE 5-27A : CHARACTERIZATION FOR REACH 27
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	 END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
27	1707+11 [3873+25]	1721+60 [3887+75]	0.3	Crest:1 Boring 2 CPTs Landside: Not available Waterside: Not available	Levee Height: 6 feet Crown Width: 20 feet Landside Slope: 1.3H:1V Waterside Slope: 3.0H:1V at analysis section	• Waterside seepage observed during irrigation season from 1677+84 to 1721+90 [3843+98 to 3888+07]. • Large voids behind the apron at the Campbell Weir. • Voids in levee during construction of bentonite-tire slurry wall in 1999 from 1707+11 to 1721+62 [3873+25 to 3887+75]	• Bentonite-tire slurry wall installed in 1999 from 1707+11 to 1721+62 [3873+25 to 3887+75] • Waterside monitoring piezometers installed in 2005. Piezometers indicate continuing seepage after installation of wall. Note: As-Constructed Drawings for Slurry Wall not available. Construction report available.	Rob (Recent Overbank) Ra (Recent Alluvium) Rch (Recent Channel)	Shallow higher resistivity zone

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
• Levee: 4 to 7 feet of lean clay. • Foundation: 10 to 15 feet of lean clay. • 5 to 10 feet of poorly-graded sand to silt. • 25 to 30 feet of clayey sand to fat clay with localized sand and silt.	200 Year 4 feet in ditch	Orchards on both LS and WS, Unlined landside ditch	1708+87 [3875+00]	WM0007_019B (crest), WM0007_019C (crest) WM0007_020C (crest)	Clayey levee; thin CL blanket underlain by SP; wide landside toe ditch; Bentonite-tire slurry wall susceptible to underseepage if wall is not performing well

TABLE 5-27B: ANALYSIS RESULTS FOR REACH 27
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1708+87

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	No exit of water	0.57	N/A	No breakout	No	Waterside seepage observed during irrigation season; Voids in levee were observed during construction of cutoff wall in 1999; Based on interpreted soil profile, levee is underlain by soft clay; Note that the existing bentonite-tire slurry cutoff wall was not considered as effective in controlling the flow, and was not modeled in the analyses; For empty ditch condition, the gradient at the bottom of the ditch did not meet criteria.
200-year + 3 feet						
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative1 for Reach 27 is fill canal, which would mitigate the high exit gradient at the canal bottom. Alternative 2 for Reach 27 is cutoff wall and reconstruct the landside slope. The cutoff wall is to block the flow into the canal, and reduce the gradient at ditch bottom. The reconstruction of the landside slope is to improve landside slope instability.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Fill Canal						
Dimensions of Primary Features: Fill canal to top (adj grade, 30ft wide x 5ft deep).						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.01	N/A	N/A	No breakout	Yes	
200-year + 4 feet	0.16	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.40				Yes	
200-year + 4 feet	1.94				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Cutoff Wall and Reconstruct the Landside Slope						
Dimensions of Primary Features: 1707+11 to 1721+60 Cutoff Wall Tip Elevation 65; plus Reconstruct the Landside Slope.						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.22	N/A	N/A	No breakout	Yes	
200-year + 4 feet	0.35	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.76				Yes	
200-year + 4 feet	1.68				Yes	

TABLE 5-28A : CHARACTERIZATION FOR REACH 28
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
28	1721+60 [3887+75]	1769+31 [3935+50]	0.9	Crest:3 Borings 5 CPTs Landside: Not available Waterside: Not available	Levee Height: 6 feet Crown Width: 23 feet Landside Slope: 1.4H:1V Waterside Slope: 2.4H:1V at analysis section	• No reported deficiencies have been identified.	• No known mitigation measures.	Ra (Recent Alluvium) Qml (Pleistocene Modesto Formation - Lower) Rob (Recent Overbank) Rcs (Recent Crevasse Splay)	Localized shallow higher resistivity zones

REACH ID [1]	START STATION	END STATION	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
28	1721+60 [3887+75]	1769+31 [3935+50]	• Levee: 4 to 8 feet of silt to lean clay. • Foundation: 0 to 20 feet of sandy silt to lean clay with localized clayey sand to silty sand. • 0 to 10 feet of sand to sand with silt. • 10 to 40 feet of silt to lean clay.	200 Year 5 feet in ditch	Orchards on both LS and WS along much of reach, Agricultural buildings at 1738+87 and 1744+87 [3905+00 and 3911+00], Unlined landside ditch	1760+87 [3927+00]	WM0007_024B (crest) WM0007_025C (crest)	Clayey levee; CL blanket underlain by SC/SC-SM; wide landside toe ditch susceptible to underseepage

TABLE 5-28B: ANALYSIS RESULTS FOR REACH 28
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1760+87

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	No exit of water	0.38	N/A	No breakout	Yes	No reported deficiencies have been identified; Based on interpreted soil profile, levee is underlain by soft clay; The canal is located at the landside levee toe; For empty ditch condition, the gradient at the bottom of the ditch did not meet criteria.
200-year + 3 feet	No exit of water	0.61	N/A	No breakout	No	
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	1.10				No	
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative1 for Reach 28 is fill canal, which would mitigate the high exit gradient at the canal bottom. Alternative 2 for Reach 28 is cutoff wall and reconstruct the landside slope. The cutoff wall is to block the flow into the canal, and reduce the gradient at ditch bottom. The reconstruction of the landside slope is to improve landside slope instability.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Fill Canal						
Dimensions of Primary Features: Fill canal to top (adj grade, 30ft wide x 5ft deep).						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	N/A	No breakout	Yes	
200-year + 4 feet	0.15	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.94				Yes	
200-year + 4 feet	1.67				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Cutoff Wall and Reconstruct the Landside Slope						
Dimensions of Primary Features: 1721+60 to 1728+00 Cutoff Wall Tip Elevation 65, 1728+00 to 1749+00 Cutoff Wall Tip Elevation 80, 1749+00 to 1769+31 Cutoff Wall Tip Elevation 45; Plus Reconstruct the Landside Slope.						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	0.16	NA	No breakout	Yes	
200-year + 4 feet	No exit of water	0.23	NA	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.85				Yes	
200-year + 4 feet	1.80				Yes	

TABLE 5-29A : CHARACTERIZATION FOR REACH 29
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">29</td><td rowspan="4">1769+31
[3935+50]</td><td rowspan="4">1813+33
[3979+50]</td><td>0.8</td><td>Crest:5 Borings
1 CPT
Landside: Not available
Waterside: Not available</td><td>Levee Height: 4 feet
Crown Width: 20 feet
Landside Slope: 3.5H:1V
Waterside Slope: 2.7H:1V
at analysis section</td><td>• No reported deficiencies have been identified.</td><td>• No known mitigation measures.</td><td>Ra (Recent Alluvium)
Rcu (Recent Crevasse Splay Cutoff Channel)
Rob (Recent Overbank)</td><td>~ 50 feet thick low resistivity zone below the ground surface</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th rowspan="2" colspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>• Levee: 4 to 7 feet of silty sand to lean clay.
• Foundation: 0 to 20 feet of sandy silt to lean clay with localized surficial clayey sand to silty sand.
• 0 to 25 feet of fat clay.
• 0 to 10 feet of clayey sand to silty sand with localized clayey gravel to silty gravel.
• 10 to 40 feet of lean sandy silt to lean clay.</td><td>200 Year
N/A</td><td>Orchards on both LS and WS along much of reach,
Agricultural buildings at 1782+81 and 1786+81 [3949+00 and 3953+00]</td><td>1788+81
[3955+00]</td><td>WM0007_026B (crest)
WM0007_027S (crest)</td><td colspan="2">Clayey levee; low blowcount SC-SM shallow foundation layer susceptible to underseepage</td></tr>
</table>

TABLE 5-29B: ANALYSIS RESULTS FOR REACH 29
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TO-1

Analysis Station: 1788+81

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	N/A	N/A	N/A	N/A	Yes	Water surface elevation is lower than the landside toe elevation; Seepage analysis performed for pore pressure input to slope stability; No reported deficiencies have been identified.
200-year + 3 feet	N/A	N/A	N/A	N/A	Yes	Water surface elevation is lower than the landside toe elevation; Seepage analysis performed for pore pressure input to slope stability; No reported deficiencies have been identified.
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	4.73				Yes	Short levee with flatter landside slope; No reported deficiencies have been identified.
200-year + 3 feet	3.80				Yes	Short levee with flatter landside slope; No reported deficiencies have been identified.
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
N/A

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-30A: CHARACTERIZATION OF REACH 30
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">30</td><td rowspan="4">1813+33
[3979+50]</td><td rowspan="4">1902+00
[4068+00]</td><td>1.68</td><td>Total Number of Explorations = 18;
Crown Explorations = 12 (Borings-10, CPT-2);
Landside Toe Explorations = 4 (Borings-3, CPT-1)
Waterside Toe Explorations = 2 (Borings-2, CPT-0)</td><td>At analysis section:
Crown width = 15 feet
Landside Slope = 2.8H:1V
Waterside Slope = 2.4H:1V</td><td>At 1871+91 [4037+06], in 1986, a boil appeared on the landslide slope of levee.</td><td>After high water receded, approximately 300 feet of levee at this location was excavated to surrounding grade and rebuilt; no evidence of the boil was found during excavation.</td><td>Old levee fill mapped meandering along current levee alignment. Ra waterside and landside, with Rcu along landside of old levee fill (which at some locations is just to waterside of and underneath current levee alignment).</td><td>High resistivity present, fair correspondence with logs.</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>Levee silty sand to silt. Silty sand and silt from native grade to about 20 to 25 feet bgs, underlain by clean gravel to about 40 to 60 feet bgs, underlain by clay, clayey sand, and/or clayey gravel >5 feet thick.</td><td>200 yr WSE
Head = approximately 5.6 feet at analysis section</td><td>Predominantly agricultural on landside and on waterside where levee set back from river. From approximately sta 1896+00 [4060+20] to 4068+00 [1902+00] there are homes on the landside of the levee (observed from the aerial map).</td><td>1826+94
[3993+10]</td><td>WM0007_004S</td><td colspan="2">Section selection based on location that appears most likely to have highest gradient (i.e. underseepage potential).</td></tr>
</table>

TABLE 5-30B: ANALYSIS RESULTS FOR REACH 30
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 1826+94

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
100-year	0.39	NA	NA	0.6	Yes	
200-year	0.48	NA	NA	1.2	Yes	Thin waterside blanket (if extrapolate crown boring) removed for analysis. Toe gradient marginally passes; through-seepage marginal*.
200-year + 3 feet	0.78	NA	NA	2.2	No	Toe gradient clearly does not meet criterion. Through-seepage more clearly not meeting criterion (daylights on slope, erodible material)

Landside Stability — * Consistent with past performance, report of one "boil" on the landside slope during 1986 high water event.

WSE	FS	Performance	Meets Criteria?	Comments
100-year	1.61		Yes	
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 30 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 30 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silt, which is similar to shallow near surface layers. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 1813+33 to 1816+40 Cutoff Wall Tip Elevation 80 , 1816+40 to 1865+90 Cutoff Wall Tip Elevation 40 , 1865+90 to 1877+90 Cutoff Wall Tip Elevation 50, 1877+90 to 1902+00 Cutoff Wall Tip Elevation 30

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	N/A	No breakout	Yes	Moderate depth wall ties in to fine-grained layer, mitigates through-seepage and underseepage. Fine-grained layer varies in depth
200-year + 4 feet	No exit of water	N/A	N/A	No breakout	Yes	over length of reach, so wall depth varies over length of reach, as indicated above

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.81		Yes	
200-year + 4 feet	1.78		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm (ML) with stability berm

Dimensions of Primary Features: 1813+33 to 1831+00 seepage berm 300 feet wide, 6.5 feet thick at levee toe, 1831+00 to 1888+00 seepage berm 100 feet wide, 5 feet thick at levee toe,
1888+00 to 1895+00 seepage berm 300 feet wide, 6.5 feet thick at levee toe, 1895+00 to 1902+00 seepage berm 100 feet wide, 5 feet thick at levee toe,
for each, thickness at seepage berm toe = 3 feet and stability berm height = 4 feet (on top of seepage berm)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No Exit of Water	0.86	NA	1.3	Meets criteria at levee toe. See comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	No Exit of Water	1.14	NA	Top of berm	Meets criteria at levee toe. See comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	4.72		Yes	Drained stability berm (for through-seepage) significantly increases stability FS.
200-year + 4 feet	4.15		Yes	

TABLE 5-31A: CHARACTERIZATION OF REACH 31
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
31	1902+00 [4068+00]	1958+00 [4125+00]	1.06	Total Number of Explorations = 11; Crown Explorations = 9 (Borings-8, CPT-1); Landside Toe Explorations = 2 (Borings-2, CPT-0)	At analysis section: Crown width = 16 feet Landside Slope = 1.4H:1V Waterside Slope = 2.5H:1V	none documented	none documented	Old levee fill mapped along current levee alignment for much of length (where river adjacent). Ra landside, with Rcs and Rofc at southern end landside River waterside for much of length, with Hch along waterside where levee set back from current river alignment.	High resistivity present, fair correspondence with logs; gap in HEM at upstream end of reach.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
31	Levee lean clay, with some SC and ML. Foundation profile partly mostly clay, partly 10 to 15 foot clay blanket over clean gravel, sand, and some silt to 40 to 50 feet below ground surface, underlain by >5 feet clay.	**200 yr WSE** Head = approximately 8 ft above base of canal (no head above levee toe)	Butte Main Canal runs along LS toe of levee. The landside and waterside (where the levee is setback from the river) is mainly agricultural. A canal is located approximately 25 feet away from the landside toe. The canal is about 55 feet wide and 7 feet deep.	1907+91 [4074+00]	WM0007_37S SM0007_003A SM0007_004A	If address canal, reach has potential to meet criteria otherwise, so section selection based on location that appears most likely to have highest gradient (after addressing canal). Note that depth of wall (for cost purposes) controlled by other location (036S, at d/s end of segment).

TABLE 5-31B: ANALYSIS RESULTS FOR REACH 31
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 1907+91

Existing Conditions Problem Identification

Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	NA	2.3	0.14	5.1	No	High gradient at canal due to thin blanket (canal bottom close to underlying sand). Breakout point measured from bottom of canal.
200-year + 3 feet						No documentation of performance problems occurring in canal during high water events; unknown if canal was empty or full at time

Landside Stability				
WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown					
WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 31 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. The potential seepage problems, however, are due to the presence of the canal; if the canal were not present, the potential seepage problems would not occur. So Alternative 2 for Reach 31 is filling of the canal (and re-locating the canal a large distance from the levee). Canal fill material type is silt, to be permeability-compatible with underlying material (may use more permeable fill than silt).

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 1902+00 to 1916+90 Cutoff Wall Tip Elevation 30, 1916+90 to 1933+90 Cutoff Wall Tip Elevation 75,
1933+90 to 1958+00 Cutoff Wall Tip Elevation 40

Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No Exit of Water	0.18	NA	No Breakout	Yes	Moderate depth wall ties in to fine-grained layer, mitigates through-seepage and underseepage. Fine-grained layer varies in depth
200-year + 4 feet	No Exit of Water	0.24	NA	No Breakout	Yes	over length of reach, so wall depth varies over length of reach, as indicated above

Landside Stability				
WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.62		Yes	
200-year + 4 feet	2.58		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Filled Canal

Dimensions of Primary Features: Canal filled up to adjacent grade; 1.4 % slope on top of fill for drainage. The filled canal and its existing embankments effectively become a landside berm, about 100 feet wide.

Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No Exit of Water	0.24	NA	No Breakout	Yes	Filled canal and embankments considered a berm, so gradient criterion used at embankment toe is that for berms.
200-year + 4 feet	No Exit of Water	0.51	NA	No Breakout	Yes	

Landside Stability				
WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	9.61		Yes	Berm condition, with fill up against short landside levee slope, significantly increases stability FS.
200-year + 4 feet	8.10		Yes	

TABLE 5-32A: CHARACTERIZATION OF REACH 32
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">32</td><td rowspan="4">1958+00
[4125+00]</td><td rowspan="4">1989+00
[4155+00]</td><td>0.59</td><td>Total Number of Explorations = 7;
Crown Explorations = 5
(Borings-4, CPT-1);
Landside Toe Explorations = 2
(Borings-2, CPT-0)</td><td>At analysis section:
Crown width = 14 feet
Landside Slope = 2.1H:1V
Waterside Slope = 2.1H:1V</td><td>none documented</td><td>none documented</td><td>H_{ch} (Holocene Channel deposits)
R_a (Recent Alluvium)
R_{cu} (Recent Cutoff Channel)
Rob (Recent Overbank)</td><td>Approximately 10ft thick, continuous, High to Moderate Resistive Soil layers observed immediately below the embankment.</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>Levee Embankment: SM, CL, ML, SC
Foundation: Near Surface layers include SM, CL, ML. Thick (~ 20ft) pervious zones present at about 30ft below the embankment (GW).</td><td>200 yr WSE
Head = approximately 12.9 feet at the analysis section</td><td>Predominantly agricultural on the landside and waterside (where levee set back from river). Scattered houses and sheds were observed from the aerial map.</td><td>1965+80
[4132+00]</td><td>WM0007_042B
WM0007_042S
SM0007_002B</td><td colspan="2">Section likely to control gradient, berm design (wall depth ~constant).</td></tr>
</table>

TABLE 5-32B: ANALYSIS RESULTS FOR REACH 32
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 1965+80

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.83	NA	NA	4.6	No	High gradient at toe, due to thin blanket over permeable layers, clearly does not meet criteria. Through-seepage a concern (erodible soils).
200-year + 3 feet						No documented problems during floods. Gradient not FS<1, and analyzed WSE higher than floods; analysis and performance consistent
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 32 is soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and under seepage potential. Alternative 2 for Reach 32 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type is modeled as silty sand, which is similar to there more permeable of the shallow near surface layers (some silty sands on the finer blanket layer materials).

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features:	1958+00 to 1965+80 Cutoff Wall Tip Elevation 40, 1965+80 to 1986+80 Cutoff Wall Tip Elevation 58 1986+80 to 1989+00 Cutoff Wall Tip Elevation 10					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.27	NA	NA	No Breakout	Yes	Moderate depth wall ties in to fine-grained layer, mitigates through-seepage and underseepage.
200-year + 4 feet	0.29	NA	NA	No Breakout	Yes	Fine-grained layer varies in depth over length of reach, so wall depth varies over length of reach, as indicated above
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.65				Yes	
200-year + 4 feet	1.65				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Seepage Berm (SM) with stability berm						
Dimensions of Primary Features:	1958+00 to 1982+00 seepage berm 120 feet wide, 6 feet thick at levee toe, with stability berm height = 6 feet (on top of seepage berm) 1982+00 to 1989+00 seepage berm 50 feet wide, 5 feet thick at levee toe, with stability berm height = 4 feet (on top of seepage berm) for each, thickness at seepage berm toe = 3 feet					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.20	0.76	NA	No Breakout	Yes	
200-year + 4 feet	0.30	0.92	NA	No Breakout	Meets criteria at levee toe. See comments on berm toe.	No criteria of exit gradient at berm toe greater than 100 feet in width. Seepage condition should be monitored during flood. If needed, a relief well or drainage system may be needed at the toe of the berm.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.25				Yes	
200-year + 4 feet	2.16				Yes	

TABLE 5-33A: CHARACTERIZATION OF REACH 33
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
33	1989+00 [4155+00]	2122+00 [4288+00]	2.52	Total Number of Explorations = 22; Crown Explorations = 17 (Borings-14, CPT-3); Landside Toe Explorations = 5 (Borings-2, CPT-3)	At analysis section: Crown width = 16 feet Landside Slope = 2H:1V Waterside Slope = 3.3H:1V	none documented	none documented	Ra (Recent Alluvium) Rcu (Recent Cutoff Channel) DT (Dredge Tailings)	Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment. Low resistivity observed occasionally. Gap in HEM data intermittently.

GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
Levee Embankment: SM, ML **Foundation**: Near Surface layers include ML, SM, SP-SM. Thick (~ 40ft) pervious zones present at about 30ft below the embankment (GP, GW).	**200yr WSE** Head = approximately 4.9 feet at the analysis section	Predominantly agricultural on the landside and waterside (where levee set back from river). Scattered houses and sheds were observed from the aerial map.	2076+90 [4243+00]	WM0007_009S (Crown) WM0007_002C (LS Toe) SM0007_005A (Crown)	Section likely to control gradient, berm design, and wall depth. Previous analyses at this location showed marginal/close to criteria.

TABLE 5-33B: ANALYSIS RESULTS FOR REACH 33
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2076+90

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.55	NA	NA	1.6	No	Gradient at toe does not meet. Dredge tailings (DT) modeled as GM-GP, about 60ft deep. DT has relatively little effect on gradients here.
200-year + 3 feet						No documented performance problems; gradient not much over criterion, though; consistent. Reach has ≤90-degree bends (3D effect).
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative 1 for Reach 33 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 33 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silt, which is similar to shallow near surface layers. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features:	1989+00 to 2000+80 Cutoff Wall Tip Elevation 10, 2000+80 to 2026+80 Cutoff Wall Tip Elevation 90 2026+80 to 2036+90 Cutoff Wall Tip Elevation 20, 2036+90 to 2086+90 Cutoff Wall Tip Elevation 35 2086+90 to 2122+00 Cutoff Wall Tip Elevation 90					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	NA	NA	No Breakout	Yes	Relatively deep wall needed to tie in to fine-grained layer, mitigate through-seepage and underseepage.
200-year + 4 feet	<0.1	NA	NA	No Breakout	Yes	Fine-grained layer varies in depth over length of reach, so wall depth varies over length of reach, as indicated above.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.82				Yes	
200-year + 4 feet	1.81				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Seepage Berm (ML) with stability berm, optionally with relief wells						
Dimensions of Primary Features:	1989+00 to 2020+00 seepage berm 50 feet wide, 5 feet thick at levee toe , 2020+00 to 2028+00 seepage berm 100 feet wide, 5 feet thick at levee toe , 2028+00 to 2037+00 seepage berm 50 feet wide, 5 feet thick at levee toe , 2037+00 to 2050+00 seepage berm 100 feet wide, 6 feet thick at levee toe, 2050+00 to 2065+00 connect berm toe as straight line across bend, 2065+00 to 2087+00 seepage berm 100 feet wide, 6 feet thick at levee toe, 2087+00 to 2102+00 seepage berm 50 feet wide, 5 feet thick at levee toe, 2102+00 to 2106+00 connect berm toe as straight line across bend, 2106+00 to 2122+00 seepage berm 60 feet wide, 5 feet thick at levee toe, for each, thickness at seepage berm toe = 3 feet, stability berm height = 4 feet (on top of seepage berm)					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.02	0.72	NA	Top of Berm	Yes	
200-year + 4 feet	0.22	1.05	NA	Top of Berm	Meets criteria at levee toe. See comments on	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.96				Yes	Drained stability berm (for through-seepage) significantly increases stability FS.
200-year + 4 feet	2.73				Yes	

TABLE 5-34A: CHARACTERIZATION OF FOR REACH 34
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
34	2122+00 [4288+00]	2182+00 [4348+00]	1.14	Total Number of Explorations = 9; Crown Explorations = 8 (Borings-7, CPT-1); Landside Toe Explorations = 1 (Borings-0, CPT-1)	At analysis section: Crown width = 20 feet Landside Slope = 1.6H:1V Waterside Slope = 3.0H:1V	none documented	none documented	Ra (Recent Alluvium) Rcu (Recent Cutoff Channel) DT (Dredge Tailings)	Continuous, High Resistive Soil layers observed immediately below the embankment. Gap in HEM data intermittently.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
34	**Levee Embankment**: SM, ML **Foundation**: Near Surface layers include ML, CL, SM, SP, SW-SM. Thick (~ 20ft) pervious zones present at about 30ft below the embankment (GP, GW).	**200 yr WSE** Head = approximately 1.4 feet at the analysis section	Predominantly agricultural on the landside and waterside (where levee set back from river).	2138+99 [4305+20] (hinged section)	WM0007_055S (crown)	High ground along this reach, but steep levee LS slopes along parts, so section targeted at characterizing levee LS slope stability.

TABLE 5-34B: ANALYSIS RESULTS FOR REACH 34
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2138+99

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.06	NA	NA	No Breakout	Yes*	Very low WSEs, result is low gradients. Dredge tailings (DT) modeled as GM-GP, about 60ft deep; DT has little effect on results here.
200-year + 3 feet	0.19	NA	NA	1.0	Yes*	* See note below in rationale for selection
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	1.62				Yes	
200-year + 3 feet	1.45				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Most of reach does not require rehabilitation. However, downstream end of reach needs rehabilitation for seepage. Rehabilitation measures for reach 33 should extend 1600 feet into reach 34, as described below. See reach 33 for rationale of rehabilitation measures and analysis results. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relie wells or a collection drain system may be needed.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Cutoff Wall

Dimensions of Primary Features: 2122+00 to 2138+00 Cutoff Wall Tip Elevation 90
2138+00 to 2182+00 no wall (no rehabilitation required)

Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	**					** See Reach 33 for representative analysis results.
200-year + 4 feet	**					
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with Drained Stability Berm on Top

Dimensions of Primary Features: 2122+00 to 2138+00 seepage berm 60 feet wide, 5 feet thick at levee toe, with stability berm height = 4 feet (on top of seepage berm)
2138+00 to 2182+00 no berm (no rehabilitation required)

Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	**					** See Reach 33 for representative analysis results. Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	**					
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-35A: CHARACTERIZATION OF REACH 35
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="3">35</td><td rowspan="3">2182+00
[4348+00]</td><td rowspan="3">2224+00
[4390+00]</td><td>0.8</td><td>Total Number of Explorations = 11;
Crown Explorations = 8
(Borings-7, CPT-1);
Landside Toe Explorations = 1
(Borings-1, CPT-0)
Field Explorations = 2
(Borings-2, CPT-0)</td><td>At analysis section:
Crown width = 16 feet
Landside Slope = 2.1H:1V
Waterside Slope = 3H:1V</td><td>Boil 1986 - "The foundation soils are susceptible to seepage and piping".(1986)</td><td>Site 1: Levee raise LM 9.89 to LM 10.39. Project plans entitled "Sacramento River Flood Control Project Phase II, Levee Reconstruction, Contract 3 (Sites 1, 2, 3, 8, 9, 10, 11, & 12)," dated 7/4/1997. USACE Design File No. 50-04-6001. Note: Document 314, which included as-constructed improvement measures did not include this levee raise in the "Sutter County, California Past Problem Sites Map."
"Rebuilt 300' levee section" Note: No detail of this rebuilt work was available during preparation of this SGDR.
NOTE: AS-BUILT DOCUMENT NOT AVAILABLE TO HDR TEAM.</td><td>Ra (Recent Alluvium)
Rcu (Recent Cutoff Channel)
DT (Dredge Tailings)</td><td>Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment. Gap in HEM data intermittently.</td></tr>
<tr><th>GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th>DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th>NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]

SBFCA STATION [DWR ULE STATION]</th><th>EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><td>Levee Embankment: SM, ML, CL
Foundation: Near Surface layers include ML, SM, SP-SM. Thick (~ 15ft) pervious zones present at about 30ft below the embankment (GP, GP-GM).</td><td>200 yr WSE
Head = approximately 8.2 feet at analysis section</td><td>Predominantly agricultural on the landside. A warehouse is located on the landside of the levee at approximately station 2185+00 [4351+25]</td><td>2211+30
[4377+50]</td><td>WM0007_062B (Crown)
WM0007_062S (Crown)
SM0007_004B (Toe)</td><td colspan="2">Section likely to control gradient, berm design, and wall depth. Previous analyses at this location showed marginal/close to criteria.</td></tr>
</table>

TABLE 5-35B: ANALYSIS RESULTS FOR REACH 35
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2211+30

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.36	NA	0.57	1.8	Yes	Dredge tailings (DT) modeled as GM-GP, about 40ft deep. Underseepage gradients meet criteria, through-seepage marginal.
200-year + 3 feet	0.50	NA	0.72	3.6	No	Underseepage does not meet the field criterion, nor through-seepage (daylights on slope, erodible materials). Reach has 90-deg bends. A boil reported in 1986 (different location on same reach); consistent with analysis results showing not meeting criteria.

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 35 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 35 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silty sand, which is similar to shallow near surface layers.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 2182+00 to 2224+00 Cutoff Wall Tip Elevation 55

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	NA	0.12	No Breakout	Yes	Moderate depth wall ties in to fine-grained layer, mitigates through-seepage and underseepage.
200-year + 4 feet	<0.1	NA	0.14	No Breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	1.79		Yes	
200-year + 4 feet	1.79		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Undrained Seepage Berm with and without Drained Stability Berm on Top

Dimensions of Primary Features:
2182+00 to 2199+00 seepage berm 65 feet wide, 5 feet thick at levee toe (no drained stability berm on top)
2199+00 to 2203+00 connect berm toe as straight line across bend (no drained stability berm on top)
2203+00 to 2224+00 seepage berm 65 feet wide, 5 feet thick at levee toe, with drained stability berm height = 5 feet (on top of seepage berm)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.08	0.62	NA	Top of Berm	Yes	Gradients at berm toe same values as in field for ex. conditions, but farther from levee; berm width extended to point where meet criteria.
200-year + 4 feet	0.19	0.78	NA	Top of Berm	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.44		Yes	
200-year + 4 feet	2.15		Yes	

TABLE 5-36A: CHARACTERIZATION OF REACH 36
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
36	2224+00 [4390+00]	2259+00 [4425+00]	0.66	Total Number of Explorations = 7; Crown Explorations = 5 (Borings-4, CPT-1); Landside Toe Explorations = 2 (Borings-2, CPT-0)	At analysis section: Crown width = 8 feet Landside Slope = 3.2H:1V Waterside Slope = 2.9H:1V	none documented	none documented	Generally runs along boundary between DT and Ra, crossing onto and overlying each along part, with Rcu crossing under at several locations. H_{ms} (Holocene meander Scrolls)	Moderate resistivity, gap in HEM in d/s part of reach.

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
36	2224+00 [4390+00]	2259+00 [4425+00]	Silt levee. Silty sand, silt, and clay from native grade to about 15 to 20 feet bgs, underlain by clean sand and gravel to about 35 feet bgs, underlain by clay and/or clayey gravel >5 feet thick.	**200 yr WSE** Head = approximately 7.5 ft at analysis section	Predominantly agricultural on the landside and waterside (where levee set back from river). Scattered houses and sheds on the landside and waterside were observed from the aerial map.	2250+78 [4417+00]	SM0007_005B (TOE)	Section likely to control gradient, berm design, and wall depth.

TABLE 5-36B: ANALYSIS RESULTS FOR REACH 36
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2250+78

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.94	N/A	N/A	3.4	No	Gradient at toe does not meet. Dredge tailings (DT) modeled as GM-GP, about 40ft deep, about ≥300 feet to waterside of levee.
200-year + 3 feet						No documented performance problems; gradient not much over criterion, though; consistent. Reach has 90-degree bends (3D effect).

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 36 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 36 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as clayey sand, which is similar to shallow near surface layers. Clayey sand may be of higher value for other uses on the project; a more permeable material may be used as berm material. Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features: 2224+00 to 2259+00 Cutoff Wall Tip Elevation 75

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	N/A	N/A	No breakout	Yes	Moderate depth wall ties in to fine-grained layer, mitigates through-seepage and underseepage.
200-year + 4 feet	<0.1	N/A	N/A	No breakout	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.03		Yes	
200-year + 4 feet	2.00		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm (SC) with stability berm

Dimensions of Primary Features:
2224+00 to 2227+00 seepage berm 65 feet wide, 5 feet thick at levee toe, with stability berm height = 5 feet (on top of seepage berm)
2227+00 to 2233+00 connect berm toe as straight line across bend, with stability berm height = 5 feet (on top of seepage berm)
2233+00 to 2259+00 seepage berm 300 feet wide, 7.5 feet thick at levee toe, with drained stability berm height = 4 feet (on top of seepage berm)

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	<0.1	0.95	N/A	Top of berm	Meets criteria at levee toe. See comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.
200-year + 4 feet	0.15	1.27	N/A	Top of berm	Meets criteria at levee toe. See comments on berm toe.	Monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	3.19		Yes	Drained stability berm (for through-seepage) significantly increases stability FS.
200-year + 4 feet	2.86		Yes	

TABLE 5-37A: CHARACTERIZATION OF REACH 37
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">37</td><td rowspan="4">2259+00
[4425+00]</td><td rowspan="4">2290+00
[4456+00]</td><td>0.59</td><td>Total Number of Explorations = 7;
Crown Explorations = 5
(Borings-5, CPT-0);
Landside Toe Explorations = 2
(Borings-1, CPT-1)</td><td>At analysis section:
Crown width = 20 feet
Landside Slope = 2.1H:1V
Waterside Slope = 3.1H:1V</td><td>none documented</td><td>none documented</td><td>Rcu (Recent Cutoff Channel)
DT (Dredge Tailings)
H_{ms} (Holocene meander Scrolls)</td><td>Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment.</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>Levee Embankment: ML, CL, CL-ML
Foundation: Near Surface layers include ML, SM, CL-ML. Thick (~ 15ft) pervious zones present at about 20 to 30ft below the embankment (GP, GP).</td><td>200 yr WSE
Head = approximately 6.1 feet at the analysis section</td><td>Predominantly agricultural on the landside and waterside (where levee set back from river). Scattered houses were observed from the aerial map.</td><td>2276+76
[4443+00]</td><td>WM0007_013S
SM0007_006A
SM0007_006B</td><td colspan="2">Section selected for location where gradient most likely to control fix dimensions for berm or other non-wall fix. Note that depth of wall (for cost purposes) controlled by other location (068S, at u/s end of segment).</td></tr>
</table>

TABLE 5-37B: ANALYSIS RESULTS FOR REACH 37
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2276+76

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.49	N/A	N/A	3.6	No	Through-seepage daylights, erodible soil. Underseepage marginal. Dredge tailings modeled on waterside, instead of projecting silt blanket
200-year + 3 feet	0.72	N/A	N/A	4.3	No	At this WSE, underseepage also does not meet criteria. No documented performance problems; consistent with gradients for WSEs
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis
Alternative 1 for Reach 37 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 37 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silt, which is similar to shallow near surface layers.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features: 2259+00 to 2290+00 Cutoff Wall Tip Elevation 45						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.11	N/A	N/A	No breakout	Yes	Relatively deep wall needed to tie in to layer with fine-grained material, mitigate through-seepage and underseepage.
200-year + 4 feet	0.14	N/A	N/A	No breakout	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.83				Yes	
200-year + 4 feet	1.8				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Seepage Berm (ML) with stability berm						
Dimensions of Primary Features: 2259+00 to 2290+00 seepage berm 65 feet wide, 5.5 feet thick at levee toe, with stability berm height = 6 feet (on top of seepage berm)						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.1	0.53	N/A	Top of berm	Yes	
200-year + 4 feet	0.27	0.75	N/A	Top of berm	Yes	
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.54				Yes	Drained stability berm (for through-seepage) significantly increases stability FS.
200-year + 4 feet	2.33				Yes	

TABLE 5-38A: CHARACTERIZATION OF REACH 38
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
38	2290+00 [4456+00]	2303+00 [4469+00]	0.25	Total Number of Explorations = 1; Crown Explorations = 1 (Borings-1, CPT-0)	At analysis section: Crown width = 29 feet Landside Slope = 2H:1V Waterside Slope = 2.6H:1V	None documented on levee reach. Adjacent "old" levee breached in 1955 event. "Old" levee extends southward, along river, from east end of Reach 38 levee (which runs west-east along Vance Ave). Adjacent levee that extends north from east end of Reach 38 levee is Reach 39 levee, which nearly breached in 1955, with multiple boils and sinkholes and significant flood-fighting to save levee.	none documented	DT (Dredge Tailings) H_{ms} (Holocene meander Scrolls)	Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment. Gap in HEM data towards the downstream end of the Reach.

REACH ID [1]	START STATION	END STATION	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
38	2290+00 [4456+00]	2303+00 [4469+00]	**Levee Embankment**: ML **Foundation**: Thick (~ 30ft) pervious zones present below the embankment (GM, GC, GP, GP-GM), thought to be dredge tailings.	**200 yr WSE** Head = approximately 13.1 feet above berm toe at analysis section	Scattered trees on the landside and waterside of the levee.	2299+69 [4466+00] (hinged section)	WM0007_069B WM0007_069S (CREST)	Section selected to flag anticipated critical gradient, controlling berm dimensions. Note that depth of wall (for cost purposes) controlled by other locations (068S and 070S, at either end of segment).

TABLE 5-38B: ANALYSIS RESULTS FOR REACH 38
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2299+69

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.17	N/A	N/A	0.7	No	Gradients meet. Through-seepage marginal. Does not meet based on past performance. At east end, adjacent levee to south breached
200-year + 3 feet	0.23	N/A	N/A	1.3	No	adjacent levee to north nearly breached, and those levee/foundation conditions still present at Reach 38 (silt levee built on dredge tailings)
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	2.68				Yes	
200-year + 3 feet	2.56				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Mechanisms of potential failure thought to be present for the Reach 38 levee are associated with the incompatibility of the silty levee material with the gravelly (dredge tailings) foundation soils. Under high head conditions, seepage through the silt and into the gravels, or along the silt/gravel interface, can lead to internal erosion and piping of the silt as the silt particles are carried into/through the gravel. Alternative 1 for Reach 38 is a soil-bentonite cutoff wall, to decrease the internal gradient at the silt/gravel interface and to inhibit the flow of water through the foundation along the silt/gravel interface. Alternative 2 for Reach 38 is a full de-grade and reconstruction of the levee, and a seepage berm with the newly reconstructed levee.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features:	2290+00 to 2303+00 Cutoff Wall Tip Elevation 45					
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						Existing gradients already meet criteria (exit gradients not salient for this reach); no need to analyze gradients for improved condition.
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Reconstructed Levee, with Seepage Berm						
Dimensions of Primary Features:	2290+00 to 2302+00 De-grade entire levee and reconstruct, with zoned filter at base, and regrade landside to create 300 feet-wide drained seepage berm, 5 feet thick at levee toe, with filter carried out through berm. (2302 to 2303 occupied by berm of Reach 39)					
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						Existing gradients already meet criteria (exit gradients not salient for this reach); no need to analyze gradients for improved condition.
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-39A: CHARACTERIZATION OF REACH 39
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

REACH ID [1]	REACH LIMITS [2] START STATION (SBFCA) [DWR ULE STATION]	REACH LIMITS [2] END STATION (SBFCA) [DWR ULE STATION]	LENGTH OF REACH (MILES) [3]	NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE) [4]	GENERALIZED LEVEE GEOMETRY [5]	DESCRIPTION OF DOCUMENTED PAST PERFORMANCE [6]	SUMMARY OF KNOWN IMPROVEMENT MEASURES [7]	SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS [8]	EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM [9]
39	2303+00 [4469+00]	2319+00 [4486+00]	0.3	Total Number of Explorations = 3; Crown Explorations = 3 (Borings-2, CPT-1)	At analysis section: Crown width = 60 feet Landside Slope = Varies, typically nominally 4H:1V Waterside Slope = Varies (as steep as 1H:1V in some places)	Levee nearly breached in 1955. During flood event, numerous boils and sinkholes, with multiple flood-fight crews and dozers pushing gravel into sinkholes.	Levee raising, setback, and reconstruction. **NOTE: AS-BUILT DOCUMENT NOT AVAILABLE TO HDR TEAM.**	DT (Dredge Tailings) H_{ch} (Holocene Channel deposits)	Continuous, Moderate Resistive Soil layers observed immediately below the embankment. Gap in HEM data towards the upstream end of the Reach.

REACH ID [1]	GENERALIZED SUBSURFACE CONDITIONS [10]	DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE [11]	NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES [12]	TRANSVERSE SECTION FOR EVALUATION [13] SBFCA STATION [DWR ULE STATION]	EXPLORATIONS FOR TRANSVERSE SECTION [14]	RATIONALE FOR SECTION SELECTION [15]
39	**Levee Embankment**: GP, GM **Foundation**: GM, GC, GP, GP-GM (~ 50ft thick below the embankment). Silt and silty sands layers (ML,SM) observed below the Gravel layer.	**200 yr WSE** Head = approximately 21.5 feet at analysis section	Scattered trees on the landside and waterside of the levee.	2314+91 [4481+00]	WM0007_071B (Crown) WM0007_071S (Crown)	Generally flows through, but projection of GW-GC layer (if acts GC-like) in 071S relative to ground surface profile anticipated to control gradient relationship.

TABLE 5-39B: ANALYSIS RESULTS FOR REACH 39
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2314+91

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	No Exit of Water	N/A	0.27 (Local Y-grad)	No Breakout	Yes	Gradients meet. This reach nearly breached in 1955, with numerous boils and sinkholes, with multiple flood-fight crews and
200-year + 3 feet	No Exit of Water	N/A	0.3 (Local Y-grad)	No Breakout	Yes	dozers pushing gravel into sinkholes. Levee built on dredge tailings. No documentation of rehabilitation. Levee appears to have been rehabilitated after poor performance event.
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year	3.06				Yes	
200-year + 3 feet	2.62				Yes	
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s):						
Dimensions of Primary Features:						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot						
200-year + 4 feet						

TABLE 5-40A: CHARACTERIZATION OF REACH 40
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="5">40</td><td rowspan="5">2319+00
[4486+00]</td><td rowspan="5">2359+00
[Old Butte Canal Structure]</td><td>0.76</td><td>Total Number of Explorations = 6;
Crown Explorations = 4
(Borings-4, CPT-0);
Landside Toe Explorations = 1
(Borings-1, CPT-0)
Waterside Toe Explorations = 1
(Borings-0, CPT-1)</td><td>At analysis section:
Crown width = 63 feet
Landside Slope = 2.1H:1V
Waterside Slope = 2.8H:1V</td><td>Numerous boils in 1955 event, with significant flood-fighting.</td><td>Levee raising, setback, and reconstruction.
NOTE: AS-BUILT DOCUMENT NOT AVAILABLE TO HDR TEAM.</td><td>DT (Dredge Tailings)
H_{ch} (Holocene Channel deposits)</td><td>Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment.</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>Levee Embankment: GP, GM, GP-GM, GW-GC
Foundation: GM, GC, GW, GP, GP-GM intermixed thin layers (~2 to 3ft) of ML, SM.</td><td>200 yr WSE
Head = approximately 10 feet at analysis section</td><td>Scattered trees on the landside and waterside of the levee. Some agricultural on the landside of the levee.</td><td>2332+91
[4499+00]</td><td>WM0007_072S
WM0007_014S (CREST)
SM0007_008B (FIELD)</td><td colspan="2">Projection of clay layer in 072S relative to ground surface profile at 4499 anticipated to control gradient relationship, dimensions of berm. Depth of wall (for cost purposes) controlled by other location (073S, at u/s end of segment).</td></tr>
</table>

TABLE 5-40B: ANALYSIS RESULTS FOR REACH 40
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2332+91

Existing Conditions Problem Identification

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.64	N/A	N/A	5.3	No	Does not meet gradient at toe. Past performance: boils/flood-fights. Dredge tailings (DT) modeled as GM-GP, about 40ft deep, waterside
200-year + 3 feet						

Landside Stability

WSE	FS	Performance	Meets Criteria?	Comments
200-year				
200-year + 3 feet				

Rapid Drawdown

WSE - initial	WSE - final	FS	Duration Effect	Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE				

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 40 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 40 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silt, which is similar to shallow near surface layers.

Rehabilitated Levee Alternative 1

Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall

Dimensions of Primary Features:
2319+00 to 2336+90 Cutoff Wall Tip Elevation 50
2336+90 to 2359+00 Cutoff Wall Tip Elevation 20

Seepage

WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	N/A	No breakout	Yes	Deep wall needed to tie in to layer with fine-grained material, mitigate through-seepage and underseepage.
200-year + 4 feet	No exit of water	N/A	N/A	No breakout	Yes	Fine-grained layer varies in depth over length of reach, so wall depth varies over length of reach, as indicated above

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.23		Yes	
200-year + 4 feet	2.18		Yes	

Rehabilitated Levee Alternative 2

Geotechnical Rehabilitation Measure(s): Seepage Berm (ML) with stability berm and landside depressions (pits) filled.

Dimensions of Primary Features:
Fill the large landside pit at 2321+00 to 2332+00 up to el 120,
2321+00 to 2329+00 seepage berm 65 feet wide, 5 feet thick at levee toe, with drained stability berm height = 7 feet (on top of seepage berm)
Fill the large landside pits at 2333+00 to 2343+00
2331+00 to 2346+00 seepage berm 120 feet wide, 5 feet thick at levee toe, with drained stability berm height = 10 feet (on top of seepage berm)
2346+00 to 2359+00 seepage berm 300 feet wide, 5 feet thick at levee toe, with drained stability berm height = 4 feet (on top of seepage berm

Seepage

WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.07	0.78	N/A	Top of berm	Yes	
200-year + 4 feet	0.2	0.82	N/A	Top of berm	Yes	

Landside Stability

WSE	FS		Meets Criteria?	Comments
200-year + 1 foot	2.96		Yes	
200-year + 4 feet	2.68		Yes	

TABLE 5-41A: CHARACTERIZATION OF REACH 41
FEATHER RIVER WEST LEVEE REHABILITATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="3">REACH ID
[1]</th><th colspan="2">REACH LIMITS
[2]</th><th rowspan="3">LENGTH OF REACH (MILES)
[3]</th><th rowspan="3">NUMBER OF EXPLORATIONS (LOCATION - CREST/TOE/FIELD AND TYPE)
[4]</th><th rowspan="3">GENERALIZED LEVEE GEOMETRY
[5]</th><th rowspan="3">DESCRIPTION OF DOCUMENTED PAST PERFORMANCE
[6]</th><th rowspan="3">SUMMARY OF KNOWN IMPROVEMENT MEASURES
[7]</th><th rowspan="3">SUMMARY OF SURFICIAL GEOMORPHOLOGIC UNITS
[8]</th><th rowspan="3">EVALUATION OF DIFFERENTIAL RESISTIVITY PROFILES FROM HEM
[9]</th></tr>
<tr><th>START STATION (SBFCA)</th><th>END STATION (SBFCA)</th></tr>
<tr><th>[DWR ULE STATION]</th><th>[DWR ULE STATION]</th></tr>
<tr><td rowspan="4">41</td><td rowspan="4">2359+00
[Old Butte Canal Structure]</td><td rowspan="4">2368+00
[Thermalito]</td><td>0.17</td><td>Total Number of Explorations = 1;
Crown Explorations = 1
(Borings-1, CPT-0)</td><td>At analysis section:
Crown width = 21 feet
Landside Slope = 2.6H:1V
Waterside Slope = 2H:1V</td><td>none documented</td><td>none documented</td><td>DT (Dredge Tailings)
H_{ch} (Holocene Channel deposits)</td><td>Continuous, High to Moderate Resistive Soil layers observed immediately below the embankment.</td></tr>
<tr><th rowspan="2">GENERALIZED SUBSURFACE CONDITIONS
[10]</th><th rowspan="2">DESIGN WSE BASIS (200 YEAR/100 YEAR) AND AVERAGE HEAD FOR DESIGN WSE
[11]</th><th rowspan="2">NATURAL, PHYSICAL, AND LAND USE CONSTRAINTS OR FEATURES
[12]</th><th>TRANSVERSE SECTION FOR EVALUATION
[13]</th><th rowspan="2">EXPLORATIONS FOR TRANSVERSE SECTION
[14]</th><th colspan="2" rowspan="2">RATIONALE FOR SECTION SELECTION
[15]</th></tr>
<tr><th>SBFCA STATION [DWR ULE STATION]</th></tr>
<tr><td>Levee Embankment: GP, GM, GP-GM, GW-GC
Foundation: GM, GC, GW, GP, GP-GM intermixed thin layers (~2 to 3ft) of ML, SM.</td><td>200 yr WSE
Head = approximately 9.2 feet at analysis section</td><td>Scattered trees on the landside and waterside of the levee. Some agricultural on the landside of the levee.</td><td>2362+70
[None]</td><td>SM0007_010B (CREST)</td><td colspan="2">WSE, geometry fairly consistent, so section at only boring location. Note that depth of wall (for cost purposes) expected to deepen d/s of boring (based on d/s reach characterization).</td></tr>
</table>

TABLE 5-41B: ANALYSIS RESULTS FOR REACH 41
FEATHER RIVER WEST LEVEE EARLY IMPLEMENTATION PROJECT TASK ORDER 1

Analysis Station: 2362+70

Existing Conditions Problem Identification						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year	0.81	N/A	N/A	4	No	Does not meet gradient at toe. Dredge tailings (DT) modeled as GM-GP, about 60ft deep, waterside of levee, instead of extending blanket
200-year + 3 feet						No documented problems. Gradient not FS<1, analyzed WSE higher than this levee experienced; analysis and performance consistent
Landside Stability						
WSE	FS	Performance			Meets Criteria?	Comments
200-year						
200-year + 3 feet						
Rapid Drawdown						
WSE - initial	WSE - final	FS	Duration Effect		Meets Criteria?	Comments
200-year + 1 foot	typ. winter WSE					

Rationale for Selection of Two Alternatives for Analysis

Alternative 1 for Reach 41 is a soil-bentonite cutoff wall, which was selected as an in-place alternative to mitigate both through-seepage and underseepage potential. Alternative 2 for Reach 41 is a seepage berm (for underseepage) with a drained stability berm on top (for through-seepage). Seepage berm material type modeled as silt, which is similar to shallow near surface layers.

Rehabilitated Levee Alternative 1						
Geotechnical Rehabilitation Measure(s): Soil-Bentonite Cutoff Wall						
Dimensions of Primary Features: Cutoff Wall Tip Elevation 20 at 2359+00, constant decrease in depth up to Cutoff Wall Tip Elevation 70 at 2368+00						
Seepage						
WSE	Exit Gradient Levee Toe	Ditch/Canal	Field	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	No exit of water	N/A	N/A	No breakout	Yes	Deep wall needed to tie in to layer with fine-grained material, mitigate underseepage.
200-year + 4 feet	No exit of water	N/A	N/A	No breakout	Yes	Fine-grained layer varies in depth over length of reach, so wall depth varies over length of reach, as indicated above
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	1.92				Yes	
200-year + 4 feet	1.91				Yes	

Rehabilitated Levee Alternative 2						
Geotechnical Rehabilitation Measure(s): Seepage Berm (ML) with stability berm and drainage relief trench at toe.						
Dimensions of Primary Features: 2359+00 to 2368+00 seepage berm 70 feet wide, 5 feet thick at levee toe, with drainage relief trench along berm toe that is 50 feet wide at grade, extending down 12 feet deep, with 1.5:1 backcut and frontcut, filled with drain gravel, with filter zones adjacent in-situ soils						
Seepage						
WSE	Exit Gradient Levee Toe	Berm Toe	Other	Phreatic Surface Breakout Point	Meets Criteria?	Comments
200-year + 1 foot	0.12	0.76	N/A	Top of berm	Yes	
200-year + 4 feet	0.27	0.98	N/A	Top of berm	Meets criteria at levee toe. See comments on berm toe.	Berm toe exit gradient marginally exceed the criteria. Seepage condition should be monitored during flood. If needed, a relief well or drainage system may be needed at the toe of the berm.
Landside Stability						
WSE	FS				Meets Criteria?	Comments
200-year + 1 foot	2.38				Yes	
200-year + 4 feet	2.13				Yes	

Table 5-42: Summary of Geotechnical Analysis for 100 Year WSE

Reach	Design WSE (year)	Rehabilitation Needed for DWSE and or HTOL?	Reach analyzed for 100-year WSE?	Meets geotechnical criteria for 100-year WSE?
1	100	Yes	Yes	No
2	100	Yes	Yes	No
3	100	Yes	Yes	No
4	100	Yes	Yes	No
5	200	Yes	No	No
6	200	No	No	Yes
7	200	Yes	No	No
8	200	Yes	Yes	No
9	200	Yes	Yes	No
10	200	Yes	No	No
11	200	Yes	No	No
12	200	No*	No	Yes*
13	200	Yes	No	No
14	200	No*	No	Yes*
15	200	No*	No	Yes*
16	200	No	No	Yes
17	200	Yes	No	No
18	200	Yes	Yes	No
19	200	Yes	Yes	No
20	200	Yes	Yes	No
21	200	Yes	Yes	No
22	200	Yes	Yes	No
23	200	Yes	Yes	Yes
24	200	Yes	Yes	No
25	200	No	Yes	Yes
26	200	Yes	Yes	No
27	200	Yes	Yes	No
28	200	Yes	Yes	No
29	200	No	Yes	Yes
30	200	Yes	Yes	Yes
31	200	Yes	No	No
32	200	Yes	No	No
33	200	Yes	Yes	No
34	200	Yes	No	No
35	200	Yes	Yes	No
36	200	Yes	No	No
37	200	Yes	Yes	No
38	200	Yes	No	No
39	200	No*	No	Yes*
40	200	Yes	Yes	No
41	200	Yes	Yes	No

NOTE: * = Based on confirmation by receipt of as-constructed drawings or adequate confirmatory investigation.

TABLE 7-1. SUMMARY OF GEOTECHNICAL ANALYSES RESULTS FOR PRE-DESIGN FORMULATION
FEATHER RIVER WEST LEVEE REHABILITATION EARLY IMPLEMENTATION PROJECT - TASK ORDER 1

<table>
<tr><th rowspan="2">Evaluation Reach ID</th><th colspan="2">Reach Limits</th><th rowspan="2">Existing Conditions Meet Analytical Criteria?</th><th rowspan="2">Existing Conditions Meet Performance Criteria?</th><th rowspan="2">Levee Rehabilitation Needed?</th><th rowspan="2">Rehabilitation Alternative</th><th rowspan="2">Approximate Dimensions of Primary Features</th><th rowspan="2">Comments</th></tr>
<tr><th>Start Station</th><th>End Station</th></tr>
<tr><td rowspan="2">1</td><td rowspan="2">10+00</td><td rowspan="2">129+66</td><td rowspan="2">No</td><td rowspan="2">No</td><td rowspan="2">Yes</td><td>Cutoff Wall</td><td>10+00 to 58+80 Cutoff Wall Tip Elevation 20
58+80 to 83+00 Cutoff Wall Tip Elevation 18
83+00 to 114+00 Cutoff Wall Tip Elevation 24
114+00 to 129+66 Cutoff Wall Tip Elevation 27</td><td></td></tr>
<tr><td>Undrained Seepage Berm with Drained Stability Berm</td><td>Drained Stability Berm: 8 feet tall
Undrained Seepage Berm : 88 feet wide and 5 feet thick at levee toe.</td><td>Assume 25% of reach will have a seepage berm width extending to 100ft due to taller levee at some locations.</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2">129+66</td><td rowspan="2">218+66</td><td rowspan="2">No</td><td rowspan="2">No</td><td rowspan="2">Yes</td><td>Cutoff Wall</td><td>129+66 to 181+00 Cutoff Wall Tip Elevation 20
181+00 to 191+00 Cutoff Wall Tip Elevation -10
191+00 to 218+66 Cutoff Wall Tip Elevation -73 with Full Levee Degrade.</td><td>191+00 to 220+00 Full Levee Degrade.</td></tr>
<tr><td>Undrained Seepage Berm with Drained Stability Berm and Cutoff Wall with Seepage Berm</td><td>129+66 to 181+00: 8 feet tall Drained Stability Berm on Seepage Berm; Seepage Berm 100 feet wide and 5 feet thick at levee toe
181+00 to 218+66: Cutoff Wall Tip Elevation 30 feet with 100 feet wide Undrained Seepage Berm. Seepage Berm 5 feet thick at berm toe</td><td></td></tr>
<tr><td rowspan="2">3</td><td rowspan="2">218+66</td><td rowspan="2">300+66</td><td rowspan="2">No</td><td rowspan="2">No</td><td rowspan="2">Yes</td><td>Cutoff Wall</td><td>218+66 to 220+00 Cutoff Wall Tip Elevation -73
220+00 to 230+00 Cutoff Wall Tip Elevation 20
230+00 to 250+00 Cutoff Wall Tip Elevation -35
250+00 to 289+00 Cutoff Wall Tip Elevation -20
289+00 to 300+66 Cutoff Wall Tip Elevation 15</td><td></td></tr>
<tr><td>Undrained Seepage Berm with Drained Stability Berm</td><td>8 feet tall Drained Stability Berm on 300 feet wide Undrained Seepage Berm with monitoring for seepage at the toe of the berm</td><td>Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.</td></tr>
<tr><td rowspan="2">4</td><td rowspan="2">300+66</td><td rowspan="2">410+67</td><td rowspan="2">No</td><td rowspan="2">No</td><td rowspan="2">Yes</td><td>Cutoff Wall</td><td>300+66 to 349+00 Cutoff Wall Tip Elevation 15
349+00 to 368+00 Cutoff Wall Tip Elevation 10
368+00 to 410+67 Cutoff Wall Tip Elevation 20</td><td></td></tr>
<tr><td>Undrained Seepage Berm with Drained Stability Berm</td><td>8 feet tall Drained Stability Berm on 100 feet wide Undrained Seepage Berm;
Seepage berm 5 feet thick at berm toe.</td><td></td></tr>
<tr><td rowspan="2">5</td><td rowspan="2">410+67</td><td rowspan="2">478+68</td><td rowspan="2">No</td><td rowspan="2">No</td><td rowspan="2">Yes</td><td>Cutoff Wall and Cutoff Wall with Seepage Berm</td><td>410+67 to 417+00 Cutoff Wall Tip Elevation 20
417+00 to 425+00 Cutoff Wall Tip Elevation 10
425+00 to 456+00 Cutoff Wall Tip Elevation 15
456+00 to 478+68 Cutoff Wall Tip Elevation 15 with 200 feet wide Undrained Seepage Berm.</td><td></td></tr>
<tr><td>Undrained Seepage Berm</td><td>410+67 to 478+68: 300 feet wide Seepage Berm</td><td>Seepage analysis indicate high exit gradients at the toe of the seepage berm. Therefore, monitoring at the toe of the berm for seepage is recommended during the high water events. If seepage occurs, relief wells or a collection drain system may be needed.</td></tr>
</table>

ENCLOSURE K

SBFCA WITH-PROJECT SEEPAGE/STABILITY RESULTS

Table 8-1: Geotechnical Analyses Results for Preferred Alternatives

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (ft)	Levee Height (1) (ft)	Flood Level Analyzed	Water Surface Elevation (ft)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
						Average Vertical Exit Gradient at Landside Toe, i	Breakout above Landside Toe (ft)	Comments	Minimum Factor of Safety, FS	Comments	Minimum Factor of Safety, FS (2)	Comments
Reach 2												
210+00	Cutoff Wall Tip Elevation 30' with 100' Wide Undrained Seepage Berm	62.4	23.0	HTOL	59.18	Levee Toe = 0.40 (Berm Toe=0.86)	4.4' above Seepage Berm	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria".	1.44	Meets Criteria	1.17	Critical RDD Slope Stability Condition for Reach
				100yr + 1'	56.18	Levee Toe = 0.30 (Berm Toe=0.74)	2.2' above Seepage Berm		1.55			
Reach 3												
239+78	Cutoff Wall Tip Elevation (-)35'	64.3	22.5	HTOL	59.36	Levee Toe <0.05 (Ditch=0.39)	Levee Toe	Meets Criteria	1.52	Meets Criteria	1.18	Critical RDD Slope Stability Condition for Reach
				100yr + 1'	56.36	Levee Toe <0.05 (Ditch=0.34)	Levee Toe		1.55			
255+75 - With Landside Pond	Cutoff Wall Tip Elevation (-)20' Slope flattening	64.6	20.9	HTOL	59.50	Levee Toe <0.05 (Pond=0.78)	Landside Pond	Meets Criteria	1.43	Meets Criteria	n/a	Not Critical RDD Slope Condition
				100yr + 1'	56.50	Levee Toe <0.05 (Pond=0.70)	Landside Pond		1.50			
255+75 - Without Landside Pond (Adjacent Ground Surface)	Cutoff Wall Tip Elevation (-)20' Slope flattening	64.6	20.9	HTOL	59.50	Levee Toe <0.05 (Ditch <0.05)	54' from Levee Toe	Meets Criteria	1.44	Meets Criteria	n/a	Not Critical RDD Slope Condition
				100yr + 1'	56.50	Levee Toe <0.05 (Ditch <0.05)	54' from Levee Toe		1.50			

Table 8-1: Geotechnical Analyses Results for Preferred Alternatives

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (ft)	Levee Height [1] (ft)	Flood Level Analyzed	Water Surface Elevation (ft)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
						Average Vertical Exit Gradient at Landside Toe, i	Breakout above Landside Toe (ft)	Comments	Minimum Factor of Safety, FS	Comments	Minimum Factor of Safety, FS [2]	Comments
299+50	Cutoff Wall Tip Elevation (-)12'	63.3	20.3	HTOL	59.89	Levee Toe <0.05 (Ditch = 0.39)	6.7' above Levee Toe	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria". <u>Sensitivity Analysis</u> No CL/ML (Layer 8) Landside of Cutoff wall. HTOL i<0.05 (CL at Toe) i=0.52 (Ditch) 100yr + 1' i<0.05 (CL at Toe) i = 0.46 (Ditch)	1.63	Meets Criteria	n/a	Not Critical RDD Slope Condition
				100yr + 1'	56.89	Levee Toe <0.05 (Ditch = 0.34)	5.8' above Levee Toe		1.67			
Reach 4												
319+00	Cutoff Wall Tip Elevation 15'	64.4	22.5	HTOL	60.36	Levee Toe in CL/ML Blanket = 0.30, Levee Toe in CL/ML and CL Blanket = 0.25	2.0' above Levee Toe	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria". <u>Sensitivity Analysis</u> Truncated Waterside Blanket HTOL i=0.31 (Toe CL/ML) i=0.25 (Toe CL/ML, CL) 100yr + 1' i=0.25 (Toe CL/ML) i=0.20 (Toe CL/ML, CL)	1.80	Meets Criteria	1.17	Critical RDD Slope Stability Condition for Reach
				100yr + 1'	57.36	Levee Toe in CL/ML Blanket = 0.25, Levee Toe in CL/ML and CL Blanket = 0.19	1.0' above Levee Toe		1.85			

Table 8-1: Geotechnical Analyses Results for Preferred Alternatives

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (ft)	Levee Height [1] (ft)	Flood Level Analyzed	Water Surface Elevation (ft)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
						Average Vertical Exit Gradient at Landside Toe, i	Breakout above Landside Toe (ft)	Comments	Minimum Factor of Safety, FS	Comments	Minimum Factor of Safety, FS [2]	Comments
408+80	Cutoff Wall Tip Elevation 20'	66.7	22.5	HTOL	63.04	Levee Toe = 0.27	2.3' above Levee Toe	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria". <u>Sensitivity Analysis</u> Truncated Waterside Blanket HTOL i=0.28 (Toe CL, CL) 100yr + 1' i=0.19 (Toe CL, CL)	1.50	Meets Criteria	n/a	Not Critical Slope Condition
				100yr + 1'	60.04	Levee Toe = 0.18	1.3' above Levee Toe		1.63			
Reach 5												
435+00	Cutoff Wall Tip Elevation 20'	66.8	22.0	HTOL	63.96	Levee Toe in Thin CL Blanket = 1.18 Levee Toe in CL, SM and CL Blanket = 0.32	1.3' above levee toe	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria". <u>Sensitivity Analysis</u> Truncated Waterside Blanket HTOL i=1.34 (Toe CL) i=0.33 (Toe CL, SM, CL) 100yr+1' i=1.04 (Toe CL) i=0.27 (Toe CL, SM, CL)	1.88	Meets Criteria	1.08	Critical RDD Slope Stability Condition for Reach
				100yr + 1'	60.96	Levee Toe in Thin CL Blanket = 1.00 Levee Toe in CL, SM and CL Blanket = 0.26	1.0' above levee toe		1.89			

Table 8-1: Geotechnical Analyses Results for Preferred Alternatives

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (ft)	Levee Height [1] (ft)	Flood Level Analyzed	Water Surface Elevation (ft)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
						Average Vertical Exit Gradient at Landside Toe, i	Breakout above Landside Toe (ft)	Comments	Minimum Factor of Safety, FS	Comments	Minimum Factor of Safety, FS [2]	Comments
466+25	Cutoff Wall Tip Elevation 15' with 300' Wide Undrained Seepage Berm	67.3	18.0	Physical Top of Levee	67.30	Levee Toe = 0.33 (Berm Toe=0.88)	10.0' above levee toe	Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria". <u>Sensitivity Analysis</u> Extend Cutoff Wall to Elev. 15' and Add Layer of ML between Layers 3 and 4, similar to actual conditions HTOL i=0.08 (Berm and CL at Levee Toe) i=0.60 (CL at Berm Toe) 100yr + 1' i<0.05 (Berm and CL at Levee Toe) i=0.55 (CL at Berm Toe)	1.20	Meets Criteria	n/a	Not Critical RDD Slope Condition
				200yr + 1'	65.25	Levee Toe = 0.26 (Berm Toe=0.80)	9.1' above levee toe		1.71			
477+00	Star Bend Setback Levee Cutoff Wall Tip Elevation (-) 20'	69.9	19.9	HTOL	68.56	Levee Toe <0.05	76' from Levee Toe	No rehabilitation measure under the FRWL Project. Existing conditions meet criteria.	1.76	Meets Criteria	n/a	Not Critical RDD Slope Condition
				200yr + 1'	65.56	Levee Toe <0.05	76' from Levee Toe		1.82			

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 7													
Station 539+30	Cutoff Wall Tip Elev. -5'	69.8	48	21.8	Physical Top of Levee (PTOL)	69.8	<0.10 [Toe, blanket = 7.8'] 0.54 [50' from toe, blanket=5.2']	0.0	Existing relief well did not perform adequately during previous flood event. Accordingly, they are not included in the model. Exit gradients are calculated at the toe and 50 feet from the toe. Blanket includes Layer 2 (CL).	1.54 (shallow) 2.36 (deep)		-	
					200yr + 1'	67.4	<0.10 [Toe, blanket = 7.8'] 0.50 [50' from toe, blanket=5.2']	0.0		1.65 (shallow) 2.42 (deep)		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 541+00	Cutoff Wall Tip Elev. -5'	69.8	47.1	22.7	Physical Top of Levee (PTOL)	69.7			Sensitivity Analysis Only Responding DWR review comment ID # 48, a set of sensitivity analyses were performed with adjusted stratigraphy. Steady State Seepage and Stability Analyses Results are as below; (Exit gradients are calculated at approximately 40 feet from toe through both thin and thick blankets) PTOL i=0.24 [Near toe, thin blanket=7'] i=0.19 [Near toe, thick blanket=12.6'] Breakout above toe =1.2' FS = 2.00 (Slope stability) 200yr+1' i=0.23 [Near toe, thin blanket=7'] i=0.18 [Near toe, thick blanket=12.6'] Breakout above toe =1.2' FS = 2.02 (Slope stability) Sensitivity analysis performed with sand to elev. -8' results in; PTOL i=0.24 [Near toe, thin blanket=7'] i=0.20 [Near toe, thick blanket=12.6'] Breakout above toe =1.2' FS = 1.99 (Slope stability) 200yr+1' i=0.24 [Near toe, thin blanket=7'] i=0.18 [Near toe, thick blanket=12.6'] Breakout above toe =1.2' FS = 2.02 (Slope stability) Sensitivity analysis performed with aquiclude truncated on landside results in; PTOL i=0.39 [Near toe, thin blanket=7'] i=0.33 [Near toe, thick blanket=12.6'] Breakout above toe =2' FS = 1.86 (Slope stability) 200yr+1' i=0.37 [Near toe, thin blanket=7'] i=0.30 [Near toe, thick blanket=12.6'] Breakout above toe =2' FS = 1.90 (Slope stability)	RDD Analysis Only		-	This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
					200yr + 1'	67.4						1.30	Waterside Levee Slope: 2.7H to flatter than 3.0H:1V Levee Slope Height = 22 feet Bench Width to Bank Slope = 15 feet Bank Slope: 1.4H to 1.7H:1V Bank Slope Height = 36 feet

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 545+50	Cutoff Wall Tip Elev. -5' Relief Wells: 50 feet deep, spaced 60 feet apart.	70.3	44.9	25.4	Physical Top of Levee (PTOL)	70.3	Sensitivity Analysis Only Responding to USACE review comment ID #1, sensitivity analysis was performed with and without the aquiclude to see if only cutoff wall option for this subreach could work or not. The following sensitivity results showed that the only cutoff wall option would work with the presence of aquiclude. <u>Sensitivity analysis</u> with hanging wall (tip elev. -54.5') without the aquiclude (SP instead of CL for Layer 6 results in; 200yr+4' :i=0.85 200yr+1': i=0.76 <u>Sensitivity analysis</u> with shallow cutoff wall (tip elev. -0.7') with the aquiclude (5.2' thick Elev. -0.7' to 4.5') results in; 200yr+4' :i=0.30 200yr+1': i=0.27			RDD Analysis Only		-	This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
					200yr + 1'	67.5						1.29	Waterside Levee Slope: 2.1H to flatter than 3.0H:1V Levee Slope Height = 22 feet Bench Width to Bank Slope = 30 feet Bank Slope: 1.0H to 1.2H:1V Bank Slope Height = 20 feet
Station 565+50	Cutoff Wall Tip Elev. -5' Relief Wells: 50 feet deep, spaced 60 feet apart.	70.8	47.1	23.7	200yr + 4'	70.7	0.58 [Toe, blanket=20.6'] 0.83 [50' from toe, blanket=18.7']	2.4	Relief wells modeled with fixed total head boundary condition at the bottom of the blanket. Fixed total head values chosen to represent conditions Mid-way between wells (i=0.3 for 200yr+1' at the toe due to thinning blanket conditions away from the toe; i=0.6 for 200yr+4' at the toe) Exit gradients are calculated at the toe and 50 feet from the toe. Blanket includes Layer 3 (ML).	1.52		-	Levee Slope: 2.1H to flatter than 3.0H:1V Levee Slope Height = 24 feet Bench Width to Bank Slope = 8 feet Bank Slope: 2.1H:1V Bank Slope Height = 20 feet Past performance records state instability associated with rapid drawdown was experienced between stations 563+00 and 568+74 when the east bank of the levee was breached during the 1997 flood event.
					200yr + 1'	67.7	0.30 [Toe, blanket=20.6'] 0.58 [50' from toe, blanket=18.7']	2.2		1.80		1.27	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 585+00	Cutoff Wall Tip Elev. -10'	70.6	46.5	24.1	Physical Top of Levee (PTOL)	70.6	<0.10 [Toe, blanket=9.1']	11.4	Exit gradient is calculated at the toe. Blanket includes Layer 2 (CL). Responding to DWR Review Comments ID #54, <u>sensitivity analysis</u> was performed with truncated waterside blanket to validate the design. The results are as below; <u>PTOL</u> i<0.10 [Toe, blanket=7.9'] Breakout above toe = 11' FS = 1.73 (slope stability) <u>200yr+1'</u> i<0.10 [Toe, blanket=7.9'] Breakout above toe =0' FS = 1.76 (slope stability) Responding to DWR Review Comments ID #55, <u>sensitivity analysis</u> was performed with truncated landside aquiclude to validate the design. The results are as below; <u>PTOL</u> i=0.37 [Toe, blanket=7.9'] i=0.20 [Toe, blanket=25.4'] Breakout above toe = 3.1' FS = 1.58 (slope stability) <u>200yr+1'</u> i=0.32 [Toe, blanket=7.9'] i=0.17 [Toe, blanket=25.4'] Breakout above toe =1.8' FS = 1.62 (slope stability) Responding to DWR Review Comments ID #55, <u>sensitivity analysis</u> was performed with truncated landside aquiclude and blanket thinned to elev +40 to validate the design. The results are as below; <u>PTOL</u> i=0.38 [Toe, blanket=7.9'] Breakout above toe = 11' FS = 1.59 (slope stability) <u>200yr+1'</u> i=0.35 [Toe, blanket=7.9'] Breakout above toe =1.8' FS = 1.61 (slope stability)	1.73		-	
					200yr + 1'	67.8	<0.10 [Toe, blanket=9.1']	0.0		1.74		1.28	Waterside Levee Slope: 4.8H:1V Levee Slope Height = 13 feet Bench Width to Bank Slope = 25 feet Bank Slope: 1.1H:XV Bank Slope Height = 17 feet

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 8													
Station 601+00	Cutoff Wall Tip Elev. +15'	72.1	48	24.1	200yr + 1'	68.0	RDD Analysis Only			RDD Analysis Only		1.98	Waterside Levee Slope: 2.7H to flatter than 3.0H:1V Levee Slope Height = 23 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 1.4H:1V Bank Slope Height = 31 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 623+86	Cutoff Wall Tip Elev. +15'	72.1	48	24.1	200yr + 4'	71.3	<0.10 [Toe, thin blanket=10.6'] 0.57 [Toe, thick blanket =35.1'] 0.10 [20' from toe, thin blanket =9.7'] 0.59 [20' from toe, thick blanket =33.9']	0.0	Exit gradients are calculated at the toe and 20 feet from the toe. Thin blanket includes Layer 3 (CL); thick blanket includes Layer 3 (CL), Layer 4 (SP-SM) and Layer 5 (CL). Responding to DWR Review Comments ID #113, a <u>sensitivity analysis</u> was performed with SC instead of SP for Layer 2 and truncated waterside blanket. The results are as below; <u>200yr+4'</u> i=0.12 [Toe, thin blanket=10.6'] i=0.56 [Toe, thick blanket=35.1'] <u>200yr+1'</u> i<0.1 [Toe, thin blanket=10.6'] i=0.48 [Toe, thick blanket=35.1'] Responding to DWR Review Comments ID #61, a <u>sensitivity analysis</u> was performed with truncated waterside blanket. The results are as below; <u>200yr+4'</u> i=0.08 [Toe, thin blanket=10.6'] i=0.57 [Toe, thick blanket=35.1'] i=0.10 [20' from toe, thin blanket=9.7'] i=0.59 [20' from toe, thick blanket=33.9'] Breakout above toe = 0' FS = 1.89 (Slope stability) <u>200yr+1'</u> i=0.08 [Toe, thin blanket=10.6'] i=0.50 [Toe, thick blanket=35.1'] i=0.09 [20' from toe, thin blanket=9.7'] i=0.52 [20' from toe, thick blanket=33.9'] Breakout above toe = 0' FS = 1.90 (Slope stability) The above sensitivity results are identical to those of the primary model presented. Accordingly, truncated WS has no effect on the results.	1.89		-	
					200yr + 1'	68.3	<0.10 [Toe, thin blanket =10.6'] 0.50 [Toe, thick blanket =35.1'] <0.10 [20' from toe, thin blanket =9.7'] 0.52 [20' from toe, thick blanket=33.9']	0.0		1.90		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 636+00	Cutoff Wall Tip Elev. +15'	72.5	48.2	24.3	200yr + 4'	71.4	0.21 [Toe, thin blanket=9.6'] 0.60 [Toe, thick blanket=32.2']	0.0	Exit gradient is calculated at the toe. Thin blanket includes Layer 2 (CL); thick blanket includes Layer 2 (CL), Layer 3 (SP), and Layer 4 (CL).	1.68		-	
					200yr + 1'	68.4	0.19 [Toe, thin blanket=9.6'] 0.52 [Toe, thick blanket=32.2']	0.0	<u>Sensitivity analysis</u> was performed with boundary condition changed from the fixed total head to no flow along the water side of model (at the middle of river). The results are as below; <u>200yr+4'</u> i=0.13 [Toe, thin blanket] i=0.28 [Toe, thick blanket] <u>200yr+1'</u> i<0.10 [Toe, thin blanket] i<0.10 [Toe, thick blanket] Responding to DWR Review Comments ID # 63, the following explanation is added. The average exit gradient is 0.52 across a 32 feet thick blanket at 200yr + 1 ft WSE. The lower CL layer and thin blanket are separated by a permeable sand layer (SP), which will likely release some pore water pressure across SP layer and 3-dimensional directions in reality, thereby reducing the exit gradient. Furthermore, the gradient criteria (i=0.5 equivalent to FS=0.8/0.5=1.6) is based on an assumed saturated unit weight of 112.5 pcf for the blanket materials. The average estimated saturated unit weight for the 32 ft thick combined blanket layer is nearer 125 pcf. The critical exit gradient, i_{cr} and FS computations are as follows; i_{cr}=(125-62.4)/(62.4)=1.0 and FS =i_{cr}/i = 1.0/0.52=1.9 > 1.6 Given the above it is considered that the section meets criteria.	1.69		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 9													
Station 656+00	Cutoff Wall Tip Elev. +15'	74.2	50.8	23.4	200yr + 1'	68.7	Sensitivity Analysis Only Responding to DWR Review Comment ID #64, sensitivity analysis was performed with SM instead of CL for Layer 2 to validate the design. The results are as below; 200yr+4' i=0.33 [Toe, blanket=24.6'] Breakout above toe = 0.3' FS = 2.21 (Slope stability) 200yr+1' i=0.28 [Toe, blanket=24.6'] Breakout above toe = 0.3' FS = 2.31 (Slope stability)			RDD Analysis Only		0.84 1.00 (deep slip surface crossing the levee prism)	Waterside Levee Slope: 2.7H to flatter than 3.0H:1V Levee Slope Height = 23 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 1.4H:1V Bank Slope Height = 31 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Possible rapid drawdown stability remediation extent may be between station 654+00 and 667+00 Rapid drawdown (RDD) analysis show that slip surfaces with a factor of safety less than or equal to 1 occur within the steep channel slopes and that some of these encroach into the theoretical levee prism. However, slip surfaces that encroach into the existing levee profile and that could impact the global stability of the levee have a factor of safety greater than 1. Therefore, the levee is considered to have an adequate FOS to meet RDD criteria. Wide levee crown and/or waterside bench in this area. Slope maintenance to address sloughing of steep channel bank slopes may be required in the future..

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 683+00	Cutoff Wall Tip Elev. +20'	75.8	49.9	25.9	200yr + 4'	72.4	0.29 [Toe, thin blanket =12.4'] 0.48 [Toe, thick blanket =31.7']	0.0	Exit gradient is calculated at the toe. Thin blanket includes Layer 2 (SP-SM) and Layer 3 (ML); thick blanket includes Layer 2 (SP-SM), Layer 3 (ML) and Layer 5 (ML). <u>Sensitivity analysis</u> was performed with ML instead of SP-SM for Layer 2. The results are as below; <u>200yr+4'</u> i=0.46 (Toe, thin blanket) i=0.49 (Toe, thick blanket) <u>200yr+1'</u> i=0.40 (Toe, thin blanket) i=0.43 (Toe, thick blanket) Responding to DWR review comment ID#66, <u>Sensitivity analysis</u> with adjusted layer 6 (SP) referring to CPT SL0001_007C results in; <u>200yr+4'</u> i=0.38 (Toe, thin blanket) i=0.55 (Toe, thick blanket) FS=2.18 (Slope stability) <u>200yr+1'</u> i=0.32 (Toe, thin blanket) i=0.48 (Toe, thick blanket) FS=2.21 (Slope stability)	2.22		-	
					200yr + 1'	69.4	0.24 [Toe, thin blanket =12.4'] 0.41 [Toe, thick blanket =31.7']	0.0		2.24		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 705+84	Cutoff Wall Tip Elev.-10'	76.6	50.9	25.7	200yr + 4'	72.9	0.30 [Toe, thin blanket =11.3'] 0.52 [Toe, thick blanket =28.6']	0.0	Exit gradient is calculated at the toe. Thin blanket includes Layer 3 (SM) and Layer 4 (CL); thick blanket includes Layer 3 through Layer 6 (CL). Sensitivity Analysis With cutoff wall tip elev. +20' results in; 200yr+4' i=0.30 (Toe, thin blanket) i=0.63 (Toe, thick blanket) 200yr+1' i=0.26 (Toe, thin blanket) i=0.54 (Toe, thick blanket) Accordingly, results do not meet criteria. Sensitivity Analysis With the kh=1.0E-05 cm/s instead of kh=1.0E-6 cm/s for Layer 6 (CL) results in; 200yr+4' i=0.58 (Toe, thin blanket) i=0.33 (Toe, thick blanket) 200yr+1' i=0.50 (Toe, thin blanket) i=0.28 (Toe, thick blanket) Sensitivity Analysis With the anisotropic ratio (kh/kv=10) for Layer 7 (SP-SM) results in the same as the primary analysis. Anisotropic ratio change does not affect the results.	2.40		-	
					200yr + 1'	69.9	0.20 [Toe, thin blanket =11.3'] 0.45 [Toe, thick blanket =28.6']	0.0		2.42		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 10													
Station 721+00	Cutoff Wall Tip Elev. -10'	76.6	49	27.6	200yr + 4'	73.3	0.25 [Toe, blanket =24.3']	8.2	Exit gradient is calculated at toe. Blanket includes Layer 3 (CL) and Layer 4 (CL). Sensitivity analysis with cutoff wall tip elev.+25' results in; 200yr+4' :i=0.87 [Toe] 200yr+1' : i=0.77 [Toe] Responding to DWR Review Comments ID #70, through seepage flux Q on landside slope are calculated as follows; 200yr+4'; Q=0.00131 gpm/ft of levee 200yr+1'; Q=0.00098 gpm/ft of levee Due to the presence of the clay core in the embankment and very small amount of seepage flux estimated, through seepage is considered as "meeting criteria".	1.85		-	
					200yr + 1'	70.3	0.22 [Toe, blanket =24.3']	8.2		1.87		-	
Station 733+84	Cutoff Wall Tip Elev. -5'	77.7	51.7	26.0	200yr + 4'	73.6	<0.10 [Toe, thin blanket =13.5'] 0.34 [Toe, thick blanket =64.2'] 0.15 [21' from toe, thin blanket =11.7'] 0.37 [21' from toe, thick blanket =63.5']	1.2	Exit gradients are calculated at the toe and 21 feet from toe. Thin blanket includes Layer 3 (CL); thick blanket includes Layer 3 (CL), Layer 4 (SP-SM), and Layer 5 (CL). Responding to DWR Review Comments ID #71, through seepage flux Q on landside slope are calculated as follows; 200ry+4'; Q=0.00047 gpm/ft of levee 200yr+1'; Q=0.00041 gpm/ft of levee Due to the presence of the clay core in the embankment and very small amount of seepage flux estimated, through seepage is considered as "meeting criteria".	1.92		-	
					200yr + 1'	70.6	<0.10 [Toe, thin blanket =13.5'] 0.29 [Toe, thick blanket =64.2'] 0.13 [21' from toe, thin blanket =11.7'] 0.32 [21' from toe, thick blanket =63.5']	0.8		1.95		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 11													
Station 808+85	Cutoff Wall Tip Elev. -5'	78.7	56.5	22.2	200yr + 4'	75.6	<0.10 [Toe, blanket=8.8']	0.0	Exit gradients are calculated at the toe. Blanket includes Layer2 (ML) and Layer 3 (CL).	1.61		-	
					200yr + 1'	72.6	<0.10 [Toe, blanket=8.8']	0.0		1.61		-	
Station 810+00	Cutoff Wall Tip Elev. -5'	78.7	55.6	23.1	200yr + 1'	72.6	Sensitivity Analysis Only Responding to USACE review comment ID # 3, sensitivity analysis was performed with refined aquiclude layer 5 (ML) underlain by gravel layer. The cutoff wall embedded into the aquiclude with both kh=1.0E-4 cm/s and kh=1.0E-5 cm/s produced the following results: <u>Sensitivity analysis</u> with thin aquiclude (7' thick) i = 0.42 for 200yr+1 (kh=1.0E-5 cm/s) i = 0.91 for 200yr+1 (kh=1.0E-4 cm/s) It is considered overly conservative to assume a thin aquiclude layer in combination with higher hydraulic conductivity of 1.0E-04 cm/s, As such, URS conclude from the sensitivity analysis that the proposed rehabilitation option is acceptable.			RDD Analysis Only		1.69	Waterside Levee Slope: Flatter than 3.0H:1V Levee Slope Height = 23 feet Bench Width to Bank Slope = 0 feet Bank Slope: 2.7H to flatter than 3.0H:1V Bank Slope Height = 35 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
Reach 13													
Station 861+33	Cutoff Wall Tip Elev. -30.5' Rehabilitation of existing relief wells (if needed)	81.2	55	26.2	200yr + 4'	78.2	0.42 [Toe, blanket=12.2'] 0.72 [Ditch 12' from toe, blanket=9.4']	0.0 Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 0.78 gpm	Existing relief wells are not modeled. Exit gradient is calculated at toe. Design Tech Memo for Reach 13 (July 2012) presents alternatives 1, 2, 3A, and 3B. Alternative 3A is selected for the primary analysis and alternative 2 and 3B results are presented as sensitivity analyses in GDRR. Responding DWR Review Comments #77, #78, and #83, the following sensitivity analyses were performed. <u>Sensitivity analysis</u> (alternative 2) with fully penetrating cutoff wall with full or partial levee degraded results in;	1.56		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
					200yr + 1'	75.2	0.33 [Toe, blanket=12.2'] 0.60 [Ditch 12' from toe, blanket=9.4']	0.0 Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 0.65 gpm	<u>200yr+4'</u> i<0.10 [Toe, blanket=12.2'] Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 0.03 gpm FS =1.87 (Slope stability) <u>200yr+1'</u> i<0.10 [Toe, blanket=12.2'] Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 0.0 gpm FS =1.87 (Slope stability) <u>Sensitivity analysis</u> (alternative 3B) with full levee degrade and approximately 22 feet gap between cutoff wall tip and aquiclude results in; <u>200yr+4'</u> i=0.61 [Toe, blanket=12.2'] i=0.96 [Ditch 12' from Levee Toe, blanket= 9.4'] Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 1.01 gpm FS =1.38 (Slope stability) <u>200yr+1'</u> i=0.50 [Toe, blanket=12.2'] i=0.81 [Ditch 12' from Levee Toe, blanket=9.4'] Seepage flux Q within 100 feet from toe across 100 feet distance of levee (existing relief well spacing) = 0.85 gpm FS =1.49 (Slope stability)	1.63		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 15													
Station 966+00	No proposed rehabilitation measure as cutoff wall is present	81.5	57.3	24.2	200yr + 1'	77.8	RDD Analysis Only			RDD Analysis Only Responding to DWR Review Comments ID #86, sensitivity analysis was performed with shear strength OCR<2 of CL for Layer 4. The result does not change. No effect on strength change for relatively deep material.		1.52	Representative of LD1 reported that an existing cutoff wall exists at this location, however as-built documents not available. For RDD analyses, dimensions and properties of this existing cutoff wall were assumed. As-builts are needed to confirm. Waterside Levee Slope: 1.2H to 1.4H:1V Levee Slope Height = 24 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA Sensitivity Analysis with shear strength OCR<2 for Layer 4 results in (DWR review comment ID #86) FS=1.51 (same) This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 16													
Station 993+80	None	82.8	61.3	21.5	200yr + 1'	78.7	RDD Analysis Only			RDD Analysis Only Responding to DWR Review Comments ID # 88, sensitivity analysis was performed with using SP-SM by thickening Layer 4 to elevation 30-56 feet on landside. This had no effect on the results due to same phreatic surfaces conditions as in primary analysis .		0.85	Waterside Levee Slope: 1.2H to 1.4H:1V Levee Slope Height = 24 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Critical slip surface occurs in steep levee slope. Possible extent of remediation may be between station 992+00 and 1001+00. Rapid drawdown (RDD) analysis show that slip surfaces with a factor of safety less than or equal to 1 occur within the steep channel slopes and that some of these encroach into the theoretical levee prism. However, slip surfaces that encroach into the existing levee profile and that could impact the global stability of the levee have a factor of safety greater than 1. Therefore, the levee is considered to have an adequate FOS to meet RDD criteria. Wide levee crown in this area. Slope maintenance to address sloughing of steep channel bank slopes may be required in the future.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 997+00	Monitoring for waterside slope distress during and after high water events	82.9	61.3	21.6	200yr + 1'	78.7	RDD Analysis Only			RDD Analysis Only		0.94	Waterside Levee Slope: 2.7H to flatter than 3.0H:1V Levee Slope Height = 21 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 1.4H:1V Bank Slope Height = 33 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Rapid drawdown (RDD) analysis show that slip surfaces with a factor of safety less than or equal to 1 occur within the steep channel slopes and that some of these encroach into the theoretical levee prism. Slip surfaces that encroach into the existing levee profile and that could impact the global stability of the levee have a factor of safety greater than 1. Therefore, the levee is considered to have an adequate FOS to meet RDD criteria. However, the theoretical levee prism daylights out of the lower channel slope and monitoring for signs of slope distress should be performed. Slope maintenance to address sloughing of steep channel bank slopes may be required in the future

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1005+00	None.	83.2	61.9	21.3	200yr + 1'	78.7	RDD Analysis Only			RDD Analysis Only		1.16	Waterside Levee Slope: 1.7H to flatter than 3.0H:1V Levee Slope Height = 21 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.7H:1V Bank Slope Height = 30 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
Station 1006+20	Closure of gap in cutoff wall at 5th Street Bridge crossing: cutoff wall tip elev. +40 '	84.8	70.1	14.7	200yr + 4'	81.7	<0.10 [Toe, thin blanket =13.7'] 0.15 [Toe, thick blanket =50.2'] 0.29 [50 feet from toe, thin blanket =8.6'] 0.26 [50 feet from toe, thick blanket =46.7']	0.0	Exit gradients are calculated at levee toe and 50 feet from toe. Thin blanket includes Layer 3 (ML); Thick blanket includes Layer 3 (ML), Layer 4 (SC), Layer 5 (SP-SM), and Layer 6 (ML). Responding to DWR Review Comments ID # 93, <u>sensitivity analysis</u> was performed with SM (30-49% fines, anisotropic ratio kh/kv=10) instead of ML for Layer 6 results in: <u>200yr+4'</u> i<0.10 [Toe, thin blanket=13.7'] i=0.13 [Toe, thick blanket=50.2'] i=0.50 [50 feet from toe, thin blanket=8.6'] i=0.23 [50 feet from toe, thick blanket=46.8'] FS=1.44 (Slope stability) <u>200yr+1'</u> i<0.10 [Toe, thin blanket=13.7'] i<0.10 [Toe, thick blanket=50.2'] i=0.40 [50 feet from toe, thin blanket=8.6'] i=0.19 [50 feet from toe, thick blanket=46.8'] FS=1.44 (Slope stability)	1.44		-	
					200yr + 1'	78.7	<0.10 [Toe, thin blanket =13.7'] 0.10 [Toe, thick blanket =50.2'] 0.22 [50 feet from toe, thin blanket =8.6'] 0.21 [50 feet from toe, thick blanket =46.7']	0.0		1.44		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1026+50	Closure of 10th Street Bridge crossing: Existing Conditions	91.8	60	31.8	200yr + 3'	81.3	0.42 [Existing landside toe, blanket =39.8']	10.4	Exit gradients are calculated at the toe of the existing levee. Blanket includes Layer 3 (ML), Layer 4 (ML), and Layer 5 (CL). Sensitivity analysis was performed with SM 13 to 29 % fines for Layer 2 in embankment. The results are as below; 200yr+3' i=0.42 [Levee toe], breakout=3.3 feet FS=1.17 (Slope Stability) 200yr i=0.36 [Levee toe], breakout=3.3 feet FS=1.22 (Slope Stability)	1.09			
					200yr	78.3	0.36 [Existing landside toe, blanket=39.8']	6.9		1.18			
	Closure of 10th Street Bridge crossing: Stability Berm 23 feet wide and approximately 7 feet thick at the levee toe.	91.8	60	31.8	200yr + 4'	82.3	0.26 [Levee Toe, blanket =45.7'] 0.49 [Toe of Stability Berm blanket =38.4']	1.0 above toe of berm	Proposed berm modeled assuming 3 to 7 percent of fines (Kh=4.0E-03 cm/s). Exit gradients are calculated at levee toe and at toe of stability berm. Blanket includes Layer 3 (ML), Layer 4 (ML) and Layer 5 (CL). Responding to DWR comment ID EXT/Reach16-001, the filter compatibility check has been performed (see appendix C). Sensitivity analysis was performed with proposed berm with 0 to 2 percent of fines (kh=1.5E-2 cm/s). The results are as below; 200yr+4' i=0.26 [Levee toe] i=0.49 [Toe of stability berm] breakout =0.0' above toe of berm FS=1.55 (Slope stability) 200yr+1' i=0.20 [Levee toe] i=0.42 [Toe of stability berm] breakout=0.0' above toe of berm FS=1.55 (Slope Stability)	1.46 (1.89) localized shallow failure surface at top of the stability berm)		-	
					200yr + 1'	79.3	0.20 [Levee Toe, blanket =45.7'] 0.42 [Toe of Stability Berm blanket =38.4']	0.8 above toe of berm		1.55 (1.94) localized shallow failure surface at top of the stability berm)		-	
Station 1031+00	None	84.6	62.2	22.4	200yr + 1'	79.3	RDD Analysis Only			RDD Analysis Only		1.25	Waterside Levee Slope: 2.1H to flatter than 3.0H:1V Levee Slope Height = 25 feet Bench Width to Bank Slope = 15 feet to 20 feet Bank Slope: 1.7H:1V Bank Slope Height = 17 feet

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 17													
Station 1108+86	Cutoff Wall Tip Elev. +35' with landside depression backfilled.	86.4	64.5	21.9	200yr + 4'	83.1	0.53 [Toe, blanket =7.0'] 0.64 [65' from toe, blanket=4.9']	3.0	Exit gradients are calculated at toe and approximately 65 feet from toe. Blanket includes Layer 2 (CL). Through seepage flux Q on landside slope across 100 feet of Levee <u>200yr+4'</u>; Q=0.00027 gpm/ft of levee <u>200yr+1'</u>; Q=0.00022 gpm/ft of levee	1.36		-	
					200yr + 1'	80.1	0.46 [Toe, blanket =7.0'] 0.58 [65' from toe, blanket=4.9']	1.8		1.42		-	
Station 1116+00	Cutoff Wall Tip Elev. +35' with landside depression backfilled.	86.0	64.8	21.2	200yr + 1'	80.3	RDD Analysis Only			RDD Analysis Only		1.06	Waterside Levee Slope: 2.1H to flatter than 3.0H:1V Levee Slope Height = 22 feet Bench Width to Bank Slope = 0 feet Bank Slope: Flatter than 3.0H:1V Bank Slope Height = 10 feet Existing embankment has been modeled as a normally consolidated silt with c'=0 and a ϕ'=30°. <u>Sensitivity analysis</u> assuming c'=50 psf and a ϕ'=31° generates a min FS=1.27.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1125+00	Cutoff Wall Tip Elev. +35' with landside depression backfilled.	86.0	65.6	20.4	200yr + 4'	83.5	0.19 [Toe, thin blanket =8.3'] 0.19 [Toe, thick blanket=37'] 0.56 [55' from toe, blanket =5.6']	0.0	Exit gradients are calculated at toe and 55 feet from the toe. Thin blanket includes Layer 2 (CL-ML) thick blanket includes Layers 2 through 6. This location is a transition from shallow to deeper cutoff wall. Analysis represents shallow cutoff wall.	1.71		-	
					200yr + 1'	80.5	0.13 [Toe, thin blanket =8.3'] 0.15 [Toe, thick blanket=37'] 0.50 [55' from toe, blanket =5.6']	0.0	Responding IPE review comments ID # 55, sensitivity analysis was performed with CL instead of ML for Layer 6 to evaluate effects of this layer on seepage conditions. Results are as below; <u>200yr+4'</u> i<0.10 [Toe, thin blanket=8.4'] i=0.26 [55' from toe, thin blanket=5.6'] i=0.32 [55' from toe, thick blanket=33.1'] Breakout=0' FS=1.86 (Slope Stability) <u>200yr+1'</u> i<0.10 [Toe, thin blanket=8.4'] i=0.23 [55' from toe, thin blanket=5.6'] i=0.25 [55' from toe, thick blanket=33.1'] Breakout=0' FS=1.92 (Slope Stability)	1.80		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 18													
Station 1138+86	Cutoff Wall Tip Elev. +0'	86.8	66.6	20.2	200yr + 4'	83.6	<0.10 [Toe, blanket =8.8']	0.0	Blanket includes Layer 2 (CL).	1.82		-	
					200yr + 1'	80.6	<0.10 [Toe, blanket =8.8']	0.0		1.84		-	
Station 1163+75	Cutoff Wall Tip Elev. +25'	86.8	66.7	20.1	200yr + 4'	84.1	0.39 [Toe, blanket =5.7'] 0.23 [Toe, blanket =9.8'] 0.20 [Toe, blanket =42.2']	1.2	The first blanket includes Layer 2 (CL). The second blanket includes Layer 2 (CL) and Layer 3 (SM). The third blanket includes Layer 2 (CL) through Layer 5 (ML). Due to the presence of the clay core in the embankment and the cutoff wall, through seepage is considered as "meeting criteria" Responding to DWR review comment #100, exit gradient was calculated through blanket including Layer 2 through Layer 5 (=42.2').	1.60		-	
					200yr + 1'	81.1	0.30 [Toe, blanket =5.7'] 0.18 [Toe, blanket =9.8'] 0.17 [Toe, blanket =42.2']	0.9		1.69		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 19													
Station 1224+00	Cutoff Wall Tip Elev. +5'	87.2	68.1	19.1	200yr + 4'	84.9	0.43 [Toe, blanket =4.9']	1.4	Blanket includes Layer 2(CL).	1.61		-	
					200yr + 1'	81.9	0.33 [Toe, blanket =4.9']	0.7	Sensitivity analysis with cutoff wall tip elev. +25' results in; 200yr+4' :i=0.73 [Toe] 200yr+1' :i=0.59 [Toe]	1.65		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1244+00	Cutoff Wall Tip Elev. -42'	88.3	68.9	19.4	200yr + 4'	85.0	<0.10 [Toe, blanket =5.5']	0.0	Blanket includes Layer 2 (CL).	1.69		-	
					200yr + 1'	82.0	<0.10 [Toe, blanket =5.5']	0.0	Sensitivity Analysis with the cutoff wall tip elev. -35' results in: 200yr+4': i=0.18 [Toe] 200yr+1': i=0.15 [Toe] Responding to DWR Review Comment ID # 103, the following sensitivity analyses were performed. Sensitivity analysis with truncated landside CL aquiclude and added aquifer below results in; 200yr+4' i=0.31 [Toe, blanket=5.5'] Breakout above toe = 1.7' FS = 1.58 (Slope stability) 200yr+1' i=0.24 [Toe, blanket=5.5'] Breakout above toe =1.3' FS = 1.61 (Slope stability) Sensitivity analysis with truncated landside aquicludes (CL and SM) and add aquifer below results in; 200yr+4' i=0.33 [Toe, blanket=5.5'] Breakout above toe = 1.8' FS = 1.57 (Slope stability) 200yr+1' i=0.25 [Toe, blanket=5.5'] Breakout above toe =1.4' FS = 1.61 (Slope stability)	1.69		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1293+00	Cutoff Wall Tip Elev. +35'	89.0	70.5	18.5	200yr + 4'	85.4	<0.10 [Toe, thin blanket =8.7'] 0.24 [Toe, thick blanket=40.0']	0.0	Exit gradient is calculated at toe. Thin blanket includes Layer 2 (CL). Thick blanket includes Layers 2, 3, 4, and 5.	1.79		-	
					200yr + 1'	82.4	<0.10 [Toe, thin blanket =8.7'] 0.19 [Toe, thick blanket=40.0']	0.0		1.79		-	
Reach 20													
Station 1338+83	Cutoff Wall Tip Elev. +50'	89.1	73.8	15.3	200yr + 4'	86.0	<0.10 [Toe, thin blanket =10'] 0.20 [Toe, thick blanket =37']	0.0	Exit gradient is calculated at the toe. Thin blanket includes Layer 2 (SC-SM). Thick blanket includes Layer 2 (SC-SM), Layer 3 (SP-SM), and Layer 4 (CL).	1.59		-	Waterside Levee Slope: 2.7H to flatter than 3.0H:1V Levee Slope Height = 16 feet Bench Width to Bank Slope = 15 feet to 20 feet Bank Slope: 2.1H to 2.7H:1V Bank Slope Height = 13 feet
					200yr + 1'	83.0	<0.10 [Toe, thin blanket =10'] 0.15 [Toe, thick blanket =37']	0.0		1.59		1.95	
Reach 21													
Station 1378+83	Cutoff Wall Tip Elev. +32'	89.3	78.4	10.9	200yr + 4'	87.1	<0.10 [Toe, blanket =8.8'] 0.58 [bottom of ditch 40' from toe, blanket=2.4']	0.0	Exit gradients are calculated at the toe and at the bottom of ditch 40 feet from the toe). Blanket at the bottom of ditch includes Layer 2 (CL). <u>Sensitivity analysis</u> with Layer 5 (ML) replaced with SP results in: <u>200yr+4'</u> i<0.10 [Toe] i=0.58 [bottom of ditch] <u>200yr+1'</u> i<0.10 [Toe] i=0.58 [bottom of ditch]	1.89		-	
					200yr + 1'	84.1	<0.10 [Toe, blanket = 8.8'] 0.58 [bottom of ditch 40' from toe, blanket=2.4']	0.0		1.89		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1400+00	Cutoff Wall Tip Elev. +55	89.9	78.7	11.2	200yr + 4'	87.4	Sensitivity Analysis Only			2.37			
					200yr + 1'	84.4	Responding to IPE Review Comment ID #45, analysis was performed at station 1400+00. The results are as below; <u>200yr+4'</u> i<0.10 [toe, thin blanket =12.5'] i=0.24 [toe, thick blanket =22.3'] i=0.49 [bottom of ditch 50' from toe, thin blanket =6.5'] i=0.51 [bottom of ditch 50' from toe, thick blanket =16.3'] <u>200yr+1'</u> i<0.10 [toe, thin blanket =12.5'] i=0.19 [toe, thick blanket =22.3'] i=0.44 [bottom of ditch 50' from toe, thin blanket =6.5'] i=0.42 [bottom of ditch 50' from toe, thick blanket =16.3']			2.37			
Station 1427+00	Cutoff Wall Tip Elev. +40'	91.4	80.3	11.1	200yr + 4'	87.6	<0.10 [Toe, blanket =10.2'] 0.37 [bottom of ditch 52' from levee toe, blanket=5.6']	0.0	Blanket at the bottom of ditch includes Layer 3 (SM).	1.69		-	
					200yr + 1'	84.6	<0.10 [Toe, blanket =10.2'] 0.29 [bottom of ditch 52' from toe, blanket=5.6']	0.0		1.70		-	
Station 1430+00	Cutoff Wall Tip Elev. +40'	91.0	74.6	16.4	200yr + 4'	87.6	0.34 [Toe at the bottom of canal, blanket=5.0']	0.0	Exit gradient is calculated at the bottom of canal adjacent to levee. Blanket includes Layer 3 (SM). Total head boundary condition as an elevation head at far field is used for the upper pervious Layer 3 (SP-SM) on right side of model. It prevents unrealistic seepage flow back from landside.	1.43		-	
					200yr + 1'	84.6	0.24 [Toe at the bottom of canal, blanket=5.0']	0.0		1.48		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 22													
Station 1458+00	Cutoff Wall Tip Elev. +50' Full levee degrade and reconstruction due to severe animal burrowing (not modeled see note in figure C-R22-1A)	90.7	77	13.7	200yr + 4'	88.2	<0.10 [Toe, blanket =5.8'] <0.10 [Ditch 170' from toe, blanket =2.0']	0.0	Exit gradients are calculated at toe and at bottom of ditch 170 feet from toe. Breakout is beyond landside toe	1.90		-	
					200yr + 1'	85.2	<0.10 [Toe, blanket =5.8'] <0.10 [Ditch 170' from toe, blanket =2.0']	0.0		1.90		-	
Station 1468+83	Cutoff Wall Tip Elev. +55'	91.4	79.1	12.3	200yr + 4'	88.4	<0.10 [Toe, blanket =3.0'] <0.10 [at the bottom of ditch 85' from toe, blanket =5.0']	0.0	Exit gradients are calculated at toe and at the bottom of ditch 85 feet from toe. Breakout is beyond landside toe <u>Sensitivity Analysis</u> For Layer 3 (SC-SM) replaced with SP-SM results in; <u>200yr+4'</u> i<0.10 [Toe] i<0.10 [bottom of ditch] <u>200yr+1'</u> i<0.10 [Toe] i<0.10 [bottom of ditch] <u>Sensitivity Analysis</u> For the Layer 2 (CL) with kv=1.0E-5cm/s results in; <u>200yr+4'</u> i<0.10 [Toe] i<0.10 [bottom of ditch] <u>200yr+1'</u> i<0.10 [Toe] i<0.10 [bottom of ditch]	2.03		-	
					200yr + 1'	85.4	<0.10 [Toe, blanket =3.0'] <0.10 [at the bottom of ditch 85' from toe, blanket=5.0']	0.0		2.03		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1470+00	Cutoff Wall Tip Elev. +55'	91.8	81.5	10.3	200yr + 1'	85.4	RDD Analysis Only			RDD Analysis Only		1.51	Waterside Levee Slope: 1.4H to 2.7H:1V Waterside Levee Slope Height = 10 feet Bench Width to Bank Slope = 0 feet Bank Slope: 2.7H:1V Bank Slope Height = 8 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1499+00	Cutoff Wall Tip Elev. +55'	90.6	79.5	11.1	200yr + 4'	89.2	<0.10 [Toe, thin blanket=9.2'] 0.17 [Toe, thick blanket=26.3'] <0.10 [170' from toe, thin blanket=4.8'] 0.38 [170' from toe, thick blanket=21.9']	0.0	Exit gradients are calculated at the toe and at bottom of ditch 170 feet from the toe. Breakout is beyond toe. Thin blanket includes Layer 2 (CL). Thick blanket includes Layers 2, 3, and 4. Total head boundary condition as an elevation head at far field is used for the upper pervious Layer 3 (SP) on right side of model. It prevents unrealistic seepage flow back (limitation of 2D model) through Layer 3 SP from landside. Responding to DWR Review Comment ID # 111, <u>sensitivity analysis</u> was performed with truncated landside aquiclude with no flow boundary condition at landside to validate the design. The results are as below; <u>200yr+4'</u> i=0.35 [Toe, blanket=9.2'] i=1.54 [170' from toe, blanket=4.8'] Breakout 1.1' FS=1.19 (Slope stability) <u>200yr+1'</u> i=0.26 [Toe, blanket=9.2'] i=1.38 [170' from toe, blanket= 4.8'] Breakout 0.7' FS=1.27 (Slope stability) During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area.	1.62		-	
					200yr + 1'	86.2	<0.10 [Toe, thin blanket=9.2'] <0.10 [Toe, thick blanket=26.3'] <0.10 [170' from toe, thin blanket=4.8'] 0.29 [170' from toe, thick blanket=21.9']	0.0		1.62		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 23													
Station 1508+33	Cutoff Wall Tip Elev. +55'	91.3	81	10.3	200yr + 4'	89.5	0.0 [Toe, phreatic surface below blanket]	0.0	Breakout is beyond toe. Landside ditch is connected to Layer 4 (SP) at 250 feet from toe. Blanket includes Layer 3 (CL-ML). Responding to DWR Review Comments ID #112, the following sensitivity analyses were performed to validate the design. <u>Sensitivity analysis</u> with truncated landside aquiclude and added aquifer at elev. +50' to +60' results in; <u>200ry+4'</u> i<0.10 [Toe, blanket=5.0'] No breakout above toe FS = 1.84 (Slope stability) <u>200ry+1'</u> i<0.10 [Toe, blanket=5.0'] No breakout above toe FS = 1.84 (Slope stability) <u>Sensitivity analysis</u> with truncated landside aquiclude, added aquifer at elev. +50'-+60' and filled ditch with blanket material results in; <u>200ry+4'</u> i=0.34 [Toe, blanket=5.0'] Breakout above toe = 1.2' FS = 1.72 (Slope stability) <u>200ry+1'</u> i=0.22 [Toe, blanket=5.0'] Breakout above toe = 0.6' FS = 1.77 (Slope stability) <u>Sensitivity analysis</u> with truncated landside aquiclude, at elev. +50'-+60', and filled ditch with water results in; <u>200ry+4'</u> i=0.20 [Toe, blanket=5.0'] Breakout above toe = 0.6' <u>FS = 1.77 (Slope stability)</u> <u>200ry+1'</u> i=0.14 [Toe, blanket=5.0'] Breakout above toe = 0' FS = 1.80 (Slope stability) During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area.	1.84		-	
					200yr + 1'	86.5	0.0 [Toe, phreatic surface below blanket]	0.0		1.84		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1536+00	Cutoff Wall Tip Elev. +55'	92.1	85.1	7.0	200yr + 1'	87.4	RDD Analysis Only			RDD Analysis Only		1.32	Waterside Levee Slope: 2.7H:1V Levee Slope Height = 8 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.4H to 2.1H:1V Bank Slope Height = 10 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
Reach 24													
Station 1610+00	Cutoff Wall Tip Elev.+28'	94.4	81.3	13.1	200yr + 1'	89.3	RDD Analysis Only			RDD Analysis Only		- 1.38	Waterside Levee Slope: 1.7H:1V to flatter than 3.0H:1V Levee Slope Height = 8 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Responding to DWR Review Comments ID #115, sensitivity analysis was performed with OCR<2 strength for Layer 1 CL-ML. FS=1.22

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1615+62	Cutoff Wall Tip Elev.+28'	94.4	81.3	13.1	200yr + 4'	92.3	0.50 [Bottom of ditch, blanket=3.4']	0.0	Exit gradient is calculated at the bottom of ditch. Blanket includes Layer 2 (CL-ML). <u>Sensitivity analysis</u> with cutoff wall tip elev. +60' results in; <u>200yr+4'</u> i=2.02 [Bottom of the ditch] <u>200yr+1'</u> i=1.57 [Bottom of the ditch] Responding to DWR Review Comment ID #117, <u>sensitivity analysis</u> was performed with truncated landside aquiclude and added aquifer below. The results are as below; <u>200yr+4'</u> i=1.43 [Bottom of ditch 22' from toe, blanket=3.5'] Breakout above toe = 2.8' FS = 1.41 (Slope stability) <u>200yr+1'</u> i=1.29 [Bottom of ditch 22' from toe, blanket=3.5'] Breakout above toe = 2.6' FS = 1.49 (Slope stability) During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area.	2.03	Responding to the DWR Review Comment ID # 118, <u>sensitivity analysis</u> was performed with OCR<2 strength for layer 1 (c'=0 phi'=30). <u>200+4'</u> FS=1.97 <u>200+1'</u> FS=2.02	-	
					200yr + 1'	89.3	0.38 [Bottom of ditch, blanket=3.4']	0.0		2.09		-	
Reach 27													
Station 1710+00	Cutoff Wall Tip Elev. +65' and Landside slope reconstruction	96.9	85.4	11.5	200yr + 1'	91.0	RDD Analysis Only			RDD Analysis Only		1.81	Waterside Levee Slope: 1.7H:1V to flatter than 3.0H:1V Levee Slope Height = 8 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 28													
Station 1764+00	Cutoff Wall Tip Elev. +45' and Landside slope reconstruction	99.4	86.7	12.7	200yr + 1'	93.1	RDD Analysis Only			RDD Analysis Only		2.00	Waterside Levee Slope: 1.0H:1V to 2.7H:1V Levee Slope Height = 12 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions.
Reach 30													
Station 1820+80	Cutoff Wall Tip Elev.+30'	102.2	88.7	13.5	200yr + 4'	98.4	<0.10 [Toe, blanket=3.6']	0.0	Exit gradient is calculated at the toe. Blanket includes Layer 2 (ML) Responding to DWR Review Comment ID #123, <u>sensitivity analysis</u> was performed with the following adjustment. Layer 5 (CL) is bottom of model so added 10' thick aquifer (SP) below Layer 5 to make it possible to calculate a gradient across Layer 5. Results are as below; <u>200yr+4'</u> i=0.17 [Toe, thin blanket=3.6' (Layer 2)] i=0.18 [Toe, thick blanket=49.3' (Layers 2-5)] <u>200yr+1'</u> i=0.11 [Toe, thin blanket=3.6' (Layer 2)] i=0.15 [Toe, thick blanket=49.3' (Layers 2-5)]	1.80		-	
					200yr + 1'	95.4	<0.10 [Toe, blanket=3.6']	0.0		1.80		-	
Station 1826+94	Cutoff Wall Tip Elev.+30'	102.2	88.7	13.5	200yr + 4'	98.5	0.57 [Toe, blanket=2.3']	0.6	Exit gradient is calculated at the toe. Blanket includes Layer 1 (ML)	1.56		-	
					200yr + 1'	95.5	0.35 [Toe, blanket=2.3']	0.6		1.66		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1892+00	Cutoff Wall Tip Elev. +27'	102.2	88.7	13.5	200yr + 4'	103.3	<0.10 [Toe, blanket=11.3'] <0.10 [180' from the toe, blanket=10.5']	0.0	Exit gradients are calculated at the toe and adjacent landward levee toe. Breakout is beyond toe. Sensitivity Analysis With cutoff wall elev. +75' results in: 200yr+4' i=0.69 [Toe] i=0.87 [180 feet from the levee toe] 200yr+1' i=0.45 [Toe] i=0.65 [180 feet from the levee toe] Sensitivity Analysis With cutoff wall elev. +75 and SM material filled landside ground surface results in: 200yr + 4' i=0.39 [Toe] i=0.87 [180 feet from the levee toe] 200yr+1' i=0.20 [Toe] i=0.65 [180 feet from the levee toe]	2.07		-	
					200yr + 1'	100.3	<0.10 [Toe, blanket=11.3'] <0.10 [180' from the toe, blanket=10.5']	0.0		2.07		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1902+00	Cutoff Wall Tip Elev. +27' *Monitoring for waterside slope distress during and after high water events*	111.1	98	13.1	200yr + 1'	100.9	RDD Analysis Only			RDD Analysis Only		1.01	Waterside Levee Slope: 2.7H:1V Levee Slope Height = 15 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 2.1H:1V Bank Slope Height = 30 feet Critical slip surface occurs at transition between steep bank slopes and flatter levee slope. Section is considered critical due to steep bank slopes and lack of bench. Possible extent of remediation may extend between station 1899+00 and 1911+00. This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. *Rapid drawdown (RDD) analysis show that slip surfaces with a factor of safety less than or equal to 1 occur within the steep channel slopes. However, slip surfaces that encroach into the existing levee profile and that could impact the global stability of the levee have a factor of safety greater than 1. Therefore, the levee is considered to have an adequate FOS to meet RDD criteria.* *Wide levee crown and/or waterside bench in this area.* *Slope maintenance to address sloughing of steep channel bank slopes may be required in the future.* Responding to DWR Review Comments ID#129, sensitivity analysis with 16 feet drop was performed. FS=1.00

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 31													
Station 1907+91	Cutoff Wall Tip Elev. +44' *Slope flattening or other appropriate measures.*	107.7	102.4	5.3	200yr + 4'	104.3	0.41 [bottom of ditch, blanket =3.2']	0.0	Exit gradient is calculated at bottom of ditch adjacent to levee. Blanket includes Layer 1 (CL). Responding to DWR Review Comment ID#130, sensitivity analysis was performed with truncated landside aquiclude and added aquifer below. The results are as below; 200yr+4' i=2.53 [Bottom of ditch approx. 55' from toe, blanket=3.2'] Breakout above bottom of ditch = 5.2' FS = 1.30 (Slope stability) 200yr+1' i=2.09 [Bottom of ditch approx.. 55' from toe, blanket=3.2'] Breakout above bottom of ditch = 4.4' FS = 1.49 (Slope stability) During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area.	2.59		-	Waterside Levee Slope: Flatter than 3.0H:1V Levee Slope Height = 7 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 1.4H:1V Bank Slope Height = 30 feet Critical slip surface occurs at transition between steep bank slopes and flatter levee slope. Section is considered critical due to steep bank slopes and lack of bench. Possible extent of remediation may extend between station 1899+00 and 1911+00. *Rapid drawdown (RDD) analysis show that slip surfaces with a marginal factor of safety occur within the steep channel slopes and that these encroach into the theoretical levee prism and could compromise the integrity of the levee.* *The theoretical levee prism daylights out of the lower channel slope.* *Slope flattening or other appropriate measures should be undertaken in this area.*
					200yr + 1'	101.3	0.31 [bottom of ditch, blanket =3.2']	0.0		2.64		1.02	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1909+00	Cutoff Wall Tip Elev. +44' *Slope flattening or other appropriate measures.*	107.7	98.1	9.9	200yr+1'	101.5	RDD Analysis Only			RDD Analysis Only NOTE: i). sensitivity analysis performed with higher friction angle for gravel layers results in FS<1.0 *ii). analysis results are controlled by oversteepened channel slope, where the theoretical levee prism daylights out of the slope.*		0.93	Waterside Levee Slope: Flatter than 3.0H:1V Levee Slope Height = 10 feet Bench Width to Bank Slope = 0 feet Bank Slope: 1.2H to 1.4H:1V Bank Slope Height = 45 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Critical slip surface occurs at transition between steep bank slopes and flatter levee slope. Section is considered critical due to steep bank slopes and lack of bench. Possible extent of remediation may extend between station 1899+00 and 1911+00. *Rapid drawdown (RDD) analysis show that slip surfaces with a factor of safety less than or equal to 1 occur within the steep channel slopes and that these encroach into the theoretical levee prism and could compromise the integrity of the levee.* *The theoretical levee prism daylights out of the lower channel slope.* *Slope flattening or other appropriate measures should be undertaken in this area.*

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1924+00	Cutoff Wall Tip Elev. +75'	108.5	101.7	6.8	200yr + 1'	103.0	Sensitivity Analysis Only Responding to DWR Review Comment ID #132, seepage and stability analyses were performed. The results are as below; <u>200yr+4'</u> i=0.42 [Bottom of ditch, blanket=12.1'] Breakout above toe = 0' FS = 2.09 (Slope stability) <u>200yr+1'</u> i=0.34 [Bottom of ditch, blanket=12.1'] Breakout above toe = 0' FS = 2.26 (Slope stability)			RDD Analysis Only		1.22	Waterside Levee Slope: 2.7H:1V to flatter than 3.0H:1V Levee Slope Height = 8 feet Bench Width to Bank Slope = 15 feet Bank Slope: 1.4H to 2.1H:1V Bank Slope Height = 28 feet This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Responding to DWR Review Comments ID # 133, sensitivity analysis with drained strength c'=0 psf and phi'=28 degree and undrained strength c=100 psf and phi=15 results in FS=1.10

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1933+00	Cutoff Wall Tip Elev. +50'	109.1	102.8	6.3	200yr + 4'	106.2	<0.10 [Toe, blanket=10.2'] 0.50 [bottom of ditch 70' from toe, blanket =0.6']	0.0	Total head boundary condition as an elevation head at far field is used for the upper pervious Layer 3 (SP-SM) on right side of model. It prevents unrealistic seepage flow back from landside. Exit gradients are calculated at the toe and bottom of ditch 70 feet from the toe. Blanket includes Layer 2 (CL). <u>Sensitivity analysis</u> with no flow boundary condition at landside far field and tip elev. +75 results in; <u>200yr+1'</u> i > 2.0 [bottom of ditch, blanket=0.6'] <u>Sensitivity analysis</u> with no flow boundary condition at landside far field and tip elev. +40 results in; <u>200yr+1'</u> i=0.67 [bottom of ditch, blanket=0.6'] Responding to DWR Review Comment ID #134, <u>sensitivity analysis</u> with no flow boundary condition at landside far field and tip elev. +80 results in; <u>200yr+4'</u> i >2.0 [bottom of ditch, blanket=0.6'] FS=3.83 (Slope stability) <u>200yr+1'</u> i >2.0 [bottom of ditch, blanket=0.6'] FS=3.00 (Slope stability) Responding to DWR Review Comments ID #135, exit gradient for thick blanket are calculated as below; <u>200yr+4'</u> i<0.10 [Toe, thick blanket=18.6' through Layers 2 to 5] <u>200yr+1'</u> i<0.10 [Toe, thick blanket=18.6' through Layers 2 to 5]	3.14		-	
					200yr + 1'	103.2	<0.10 [Toe, blanket=10.2'] 0.33 [bottom of ditch 70' from toe, blanket =0.6']	0.0		3.17		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 32													
Station 1965+80	Cutoff Wall Tip Elev. +40'	110.6	89	21.6	200yr + 4'	106.6	0.30 [Toe, blanket=6.3']	0.0	Blanket includes Layer 1 (SC-SM) and Layer 2 (CL).	1.55	Sensitivity Analysis with OCR<2 for layer 2 (CL) results in; 200yr+4 FS=1.38 200yr+1 FS=1.39	-	
					200yr + 1'	103.6	0.25 [Toe, blanket=6.3']	0.0		1.56		-	
Station 1980+00	Cutoff Wall Tip Elev. +48'	110.4	98	12.4	200yr + 4'	106.8	RDD Analysis Only			RDD Analysis Only		-	Waterside Levee Slope: 1.4H to 2.7H:1V Waterside Levee Slope Height = 13 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA This section was selected for RDD analysis considering waterside slope, bank slope, and embankment, and blanket materials. The embankment and blanket materials are fine-grained plastic soils which are prone to RDD conditions. Critical slip failure surface is located within the steeper portion of the levee slope where the existing silt embankment has been modeled as normally consolidated silt with c'=0 and a ϕ'=30°. Sensitivity analysis assuming c'=50 psf and a ϕ'=31° generates a min FS=1.21.
					200yr + 1'	103.8						0.93	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 1981+50	Cutoff Wall Tip Elev. +48'	111.2	99.1	12.1	200yr + 4'	106.9	<0.10 [Toe, blanket =12.7']	0.0	Breakout is beyond toe.	1.67		-	
					200yr + 1'	103.9	<0.10 [Toe, blanket =12.7']	0.0	Responding to DWR Review Comment ID #139, sensitivity analysis was performed with shallow cutoff tip elev. +86.2'. The results are as below; 200yr+4' i<0.10 [Toe, blanket=12.7'] i=0.88 [Depression 270' from toe, blanket=4.3'] Breakout above toe = 0.7' FS = 1.56 (Slope stability) 200yr+1' i<0.10 [Toe, blanket=12.7'] i=0.78 [Depression 270' from toe, blanket=4.3'] Breakout above toe = 0' FS = 1.67 (Slope stability)	1.67		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 33													
Station 2008+00	Cutoff Wall Tip Elev. +90'	113.0	101.7	11.3	200yr + 4'	108.3	<0.10 [Toe, blanket=7.8']	0.0	Sensitivity Analysis Only	1.49		-	
					200yr + 1'	108.3	<0.10 [Toe, blanket=7.8']	0.0	Responding to IPE Review Comment ID #11, analysis was performed at station 2008+00 to verify the design. 200yr+4' i=0.15 [toe, thin blanket =18.4'] i=0.10 [toe, thick blanket=25.8'] breakout=0.7'above toe 200yr+1' i<0.10 [toe, thin blanket =18.4'] i<0.10 [toe, thick blanket=25.8'] breakout at toe	1.61		-	
Station 2076+90	Cutoff Wall Tip Elev. +33'	118.4	103.8	14.6	200yr + 4'	113.6	<0.10 [Toe, blanket=7.8']	0.0	Blanket includes Layer 2 (ML).	1.56		-	
					200yr + 1'	110.6	<0.10 [Toe, blanket=7.8']	0.0	Responding to DWR Review Comment ID#140, sensitivity analysis was performed with cutoff wall tip elev. +60'. The results are as below; 200yr+4' i=0.81 [Toe, blanket=7.8'] Breakout above toe = 3.6' FS = 1.20 (Slope stability) 200yr+1' i=0.56 [Toe, blanket=7.8'] Breakout above toe = 2.4' FS = 1.24 (Slope stability)	1.56			
Station 2114+00	Cutoff Wall Tip Elev. +90'	121.8	107.0	14.8	200yr + 4'	115.5	=0.57 [Toe, blanket=11.3']	3.1	Exit gradient is calculated at the toe. Blanket includes Layer 2 (ML).	1.42		-	
					200yr + 1'	112.5	=0.36 [Toe, blanket=11.3']	1.5	During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area.	1.56		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 34													
Station 2141+00	Cutoff Wall Tip Elev. +20'	123.5	109.9	13.6	200yr + 4'	117.3	<0.1 [Toe, blanket=9.6']	0.0	Exit gradient is calculated at levee toe. Blanket includes Layer 2 (ML). Sensitivity Analysis With cutoff wall tip elev.+90' results in; 200yr+4': i=0.71 (Toe) 200yr+1': i=0.43 (Toe) Responding to DWR Review Comment ID#145, sensitivity analysis was performed with a thicker blanket, connecting the crown and landside boring, and cutoff wall toe elev. +90'. The results are as below; 200yr+4' i=0.39 [Levee toe, blanket=17.8'] i=0.69 [70 feet from toe, blanket=9.6'] Breakout=1.7' FS=1.47 (Slope Stability) 200yr+1' i=0.23 [Levee toe, blanket=17.8'] i=0.41 [70 feet from toe, blanket=9.6'] Breakout=0.5' FS=1.60 (Slope Stability) During meeting with DWR on Aug. 14 2012, URS agrees to an additional exploration in this area (responding to DWR comment #145).	1.65	Responding to DWR Review Comment ID#144, sensitivity analysis was performed with shear strength of c'=0 psf and p'=30 deg. (OCR<2) for Layer 2. The results are as below; FS=1.44 for 200yr+4 FS=1.45 for 200yr+1	-	
					200yr + 1'	114.3	<0.1 [Toe, blanket=9.6']	0.0		1.65		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 2171+00	Cutoff Wall Tip Elev. +50'	123.9	110.3	13.6	200yr + 4'	119.4	<0.10 [Toe, thin blanket=5.5'] <0.10 [Toe, thick blanket=14.4']	0.0	Exit gradient is calculated at the toe. Thin blanket includes Layer 2 (SM). Thick blanket includes Layer 2 (SM), Layer 3 (SP) and Layer 4 (SM). Responding to DWR Review Comment ID#146, <u>sensitivity analysis</u> was performed with tip elevation +97.2'. The results are as below; <u>200yr+4'</u> i = 0.25 [Toe, thin blanket=5.5'] i = 0.39 [Toe, thick blanket=14.4'] Breakout above LS Toe = 1.2 feet FS = 1.56 (Slope stability) <u>200yr+1'</u> i = 0.18 [Toe, thin blanket=5.5'] i = 0.26 [Toe, thick blanket=14.4'] Breakout above LS Toe = 0.7 feet FS = 1.66 (Slope stability) Responding to DWR Review Comment ID#146, <u>sensitivity analysis</u> was performed with tip elevation +97.2' and with assumed ML for Layer 2 (SM) and Layer (SP). The results are as below; <u>200yr+4'</u> i = 0.85 [Toe, blanket=8.4'] Breakout above LS Toe = 4.4 feet FS = 1.32 (Slope stability) <u>200yr+1'</u> i = 0.57 [Toe, blanket=8.4'] Breakout above LS Toe = 3.0 feet FS = 1.59 (Slope stability)	1.78		-	
					200yr + 1'	116.4	<0.10 [Toe, thin blanket=5.5'] <0.10 [Toe, thick blanket=14.4']	0.0		1.80		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 2177+00	Cutoff Wall Tip Elev. +50'	123.9	110.3	13.6	200yr + 4'	119.7	0.25 [Toe, blanket=13.1']	0.5	Exit gradient is calculated at the toe. Blanket includes Layer 2 (ML). Sensitivity Analysis With the cutoff wall tip elev.+90' results in; 200yr+4': i=0.60 [Toe] 200yr+1': i=0.41 [Toe] NOTE: 2D analysis result did not meet the criteria for 200yr+4. Location is at 90 degree bend of levee; 3D effect expected to increase gradient to higher than allowable.	1.44	Sensitivity Analysis With the cutoff wall tip elev.+90' results in; 200yr+4' FS=1.04 200yr+1' FS=1.27	-	
					200yr + 1'	116.7	0.17 [Toe, blanket=13.1']	0.5		1.53			

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 35													
Station 2211+30	Cutoff Wall Tip Elev. +45'	126.2	110.3	15.9	200yr + 4'	124.7	<0.10 [Toe, blanket=15.1'] <0.10 [60 ' from toe blanket=13.3']	0.0	Exit gradients are calculated at the toe and 60 feet from the toe. Blanket includes Layer 2 (ML). Sensitivity Analysis With the thin blanket (=7.0 feet) determined based on WM0007_007B results in; 200yr+4': i<0.10 [Toe] 200yr+1': i<0.10 [Toe]	1.81		-	Waterside Levee Slope: 2.7H:1V to flatter than 3.0H:1V Levee Slope Height = 17 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA
					200yr + 1'	121.7	<0.10 [Toe, blanket=15.1'] <0.10 [60' from toe, blanket=13.3']	0.0		1.81		1.40	
Reach 36													
Station 2250+78	Cutoff Wall Tip Elev. +70'	130	113.2	16.8	200yr + 4'	126.6	<0.10 [Toe, blanket=6.5'] =0.16 [Toe, blanket=38.5']	0.0	Exit gradient is calculated at levee toe. Blanket includes Layer 2 (SC). A set of sensitivity analyses were performed by raising the cutoff wall tip elevation to +70 feet and incorporating a 3 feet thick aquiclude from elevation 75 to 78 feet. Both GC and CL materials were considered for the aquiclude. Responding to DWR Review Comment ID#152, sensitivity analysis was performed with aquiclude modeled as GC (kh=4.0E-04 cm/s) material. The results are as below; 200yr+4' i=0.65 [Levee toe (6.5 feet blanket – SC(2))] i=0.13 [Levee toe (38.5 feet blanket-SC, GW, and GC)] Breakout above LS Toe = 3.0 feet FS = 1.55 (Slope stability) 200yr+1' i=0.49 [Levee toe (6.5 feet blanket – SC(2))] i=0.10 [Levee toe (38.5 feet blanket-SC, GW, and GC)] Breakout above LS Toe = 2.3 feet FS = 1.68 (Slope stability)	2.05		-	
					200yr + 1'	123.6	<0.10 [Toe, blanket=6.5'] =0.12 [Toe, blanket = 38.5']	0.0		2.06		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 37													
Station 2276+76	Cutoff Wall Tip Elev. +42'	132.4	117.2	15.2	200yr + 4'	127.8	0.28 [10' from Toe, blanket=9.7']	0.0	Exit gradient is calculated at the toe. Blanket includes Layer 2 (ML).	1.75		-	
					200yr + 1'	124.8	0.22 [10' from Toe, blanket=9.7']	0.0	Responding to DWR Review Comment ID#153 a set of sensitivity analyses were performed with revised Layer 3 (SP) layering, a cutoff wall tip elevation of +70 feet, and hydraulic conductivity of Kh = 1.2E-03 cm/s for the aquiclude (Layer 5), based on WM007_067S. The results are as below: 200yr+4' i = 0.93 [10' from toe (9.7' blanket –ML(2)] Breakout above LS Toe = 3.5 feet FS = 1.14 (Slope stability) 200yr+1' i = 0.69 [10' from toe (9.7' blanket –ML(2)] Breakout above LS Toe = 2.2 feet FS = 1.43 (Slope stability) Responding to DWR review comment #25, sensitivity analysis was performed with Layer 4 Kh increased by factor 10 to 5.0E-01 cm/s. The results are as below; 200yr+4' i=0.24 [10' from toe (x=54.5), blanket=9.7'] Breakout above toe = 0' FS = 1.80 (Slope stability) 200yr+1' i=0.19 [10' from toe (x=54.5), blanket=9.7'] Breakout above toe = 0' FS = 1.82 (Slope stability) Above gradients are slightly lower than the original analysis. Results for analysis with Layer 4 Kh increased by factor of 100 are the same as those above.	1.81		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 2280+00	Cutoff Wall Tip Elev. +45'	132.1	116.3	15.8	200yr + 1'	124.9	RDD Analysis Only			RDD Analysis Only		1.53 (slip surface on WS bank) 1.58 (deep slip surface)	Waterside Levee Slope: 2.7H:1V to flatter than 3.0H:1V Levee Slope Height = 15 feet Bench Width to Bank Slope = 15 feet Bank Slope: 1.2H to 2.7H:1V Bank Slope Height = 12 feet
Reach 38													
Station 2291+75 (WITHOUT ML BLANKET LAYER)	Seepage berm 11' high extending horizontally at elev. 200yr + 4' for a distance of 50' from the landside levee slope before tapering to a height of 3' at a distance of 170' from levee centerline	132.9	117.6	15.3	200yr + 4'	128.5	No blanket condition	No breakout above seepage berm	Considering the presence of the pond north of this reach, total head boundary condition equal to ground surface elevation was used. The ground surface head boundary condition was assigned along the vertical face of landside in model at 1,000 feet away from the levee. Poorly graded Gravel (GP-GM, 5% fines) was assumed for seepage berm. The cutoff wall from Reach 37 extends about 200 feet into Reach 38 (from station 2290+00 to station 2292+00). This cutoff wall was not modeled in the analysis.	4.09		-	
					200yr + 1'	125.5	No blanket condition	No breakout above seepage berm		4.44		-	
Station 2291+75 (WITH ML BLANKET LAYER)	Seepage berm 11' high extending horizontally at elev. 200yr + 4' for a distance of 50' from the landside levee slope before tapering to a height of 3' at a distance of 170' from levee centerline	132.9	117.6	15.3	200yr + 4'	128.5	0.77 [toe of berm, blanket=10.6']	1.5	Considering the presence of the pond north of this reach, total head boundary condition equal to ground surface elevation was used. The ground surface head boundary condition was assigned along the vertical face of landside in model at 1,000 feet away from the levee. Poorly graded Gravel (GP-GM, 5% fines) was assumed for seepage berm. Responding to DWR comment ID EXT/Reach38-002 and IPE comment ID EXT/Reach38-001, the filter compatibility check has been performed (see appendix C).	2.68		-	
					200yr + 1'	125.5	0.57 [toe of berm, blanket=10.6']	1.5		3.25		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 40													
Station 2328+58	No rehabilitation measure	137.0	119.0	18.0	200yr + 4'	133.0	No blanket condition	Elev. 119.7' (Pond WSE 119.0') Elev. 108.0' (Pond WSE 106.5')	The pond was evaluated with a lower water level equal to that recorded during the LiDAR survey (Elevation 106.5 feet) and a higher water level equal to the landside ground surface elevation (Elevation 119.0 feet). A no flow boundary condition at a distance of 2,000 feet from the levee centerline was assumed for the landside vertical face of the model. A continuous bathymetry survey is not available for these landside ponds. However, the HDR design team has performed a line survey at Station 2328+00. The cross section for Station 2328+58 was developed using the DWR ULE LiDAR data and extended in the pond using the FRWL survey data. Responding to DWR review comment ID EXT/Reach40-002 for Design Tech Memo, the rate of seepage flow into the landside pond is calculated as below: <u>Pond WSE 119.0'</u> 200yr+4': Total Q = 0.42 gpm/ft Q above El. 119.0' = 0.08 gpm/ft 200yr+1': Total Q = 0.30 gpm/ft Q above El. 119.0' = 0.06 gpm/ft <u>Pond WSE 106.5'</u> 200yr+4': Total Q = 0.63 gpm/ft Q above El. 106.5' = 0.26 gpm/ft 200yr+1': Total Q = 0.52 gpm/ft Q above El. 106.5' = 0.19 gpm/ft	1.47 (Pond WSE 119.0') 1.57 (Pond WSE 106.5')			
					200yr + 1'	130.0	No blanket condition	Elev. 119.0' (Pond WSE 119.0') Elev. 108.0' (Pond WSE 106.5')		1.52 (Pond WSE 119.0') 1.65 (Pond WSE 106.5')			

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Station 2332+91	Seepage berm 120' feet wide, 9' thick at levee toe and 3' thick at berm toe	137.6	120.2 (Top of seepage berm: 129.2)	17.4 (Top of levee to top of seepag e berm: 8.4)	200yr + 4'	134.1	<0.10 [Toe of levee, blanket=14.6'] <0.10 [Toe of seepage berm, blanket=5.2']	No breakout above seepage berm	Exit gradients are calculated at the toe of levee and toe of seepage berm. Blanket at levee toe includes Layer 11 (GP-GM, seepage berm) and Layer 3 (CL). Blanket at seepage berm toe includes Layer 3 (CL). Responding to DWR comment ID EXT/Reach40-003, and IPE comment IDs EXT/Reach40-002 and EXT/Reach40-003, the filter compatibility check has been performed (see appendix C). Responding to DWR review comment ID EXT/Reach40-002 for Design Tech Memo, the rate of seepage flow from the berm toe extending 125 ft landside is calculated as below: 200yr+4': Total Q = 0.33 gpm/ft 200yr+1': Total Q = 0.29 gpm/ft	2.25		-	
					200yr + 1'	131.1	<0.10 [Toe of levee, blanket=14.6'] <0.10 [Toe of seepage berm, blanket=5.2']	No breakout above seepage berm		2.25		-	
Station 2349+00	Seepage berm 100' feet wide, 9' thick at levee toe and 3' thick at berm toe	139.0	120.0	19.0	200yr + 4'	135.4	<0.10 [Toe of levee, blanket=22.3'] 0.23 [Toe of seepage berm, blanket=13.4']	No breakout above seepage berm	Considering the presence of the Sutter Butte Canal in this area, a total head boundary condition used at the landside vertical face of the model at 1,800 feet away from the levee. Exit gradients are calculated at the toe of levee and toe of seepage berm. Blanket at levee toe includes Layer 8 (GP-GM, seepage berm), Layer 1 (GP-GM), and Layer 2 (SC). Blanket at seepage berm toe includes Layer 1 (GP-GM) and Layer 2 (SC). Responding to DWR comment ID EXT/Reach40-003, and IPE comment IDs EXT/Reach40-002 and EXT/Reach40-003, the filter compatibility check has been performed (see appendix C).	2.27		-	
					200yr + 1'	132.4	<0.10 [Toe of levee, blanket=22.3'] 0.13 [Toe of seepage berm, blanket=13.4']	No breakout above seepage berm		2.27		-	

Table 8-1. Seepage, Stability and Rapid Drawdown Analysis Results.

Reach and Station for Analysis Section	Rehabilitation Measure at Analysis Section	Top of Levee Elevation (NAVD88, approx. feet)	Landside Toe Elevation (NAVD88, approx. feet)	Levee Height[1] (feet)	Flood Level Analyzed	Water Surface Elevation (NAVD88, feet)	Steady State Seepage Analysis Results			Landside Slope Stability Analysis Results		Rapid Drawdown Analysis Results	
							Average Vertical Exit Gradient, i [Location, Blanket Thickness]	Breakout Above Landside Toe (feet)	Comments	Minimum Factor of Safety	Comments	Minimum Factor of Safety[2]	Comments
Reach 41													
Station 2362+70	Seepage berm 100' wide, 5' thick at levee toe (including 1' thick filter layer) and 3' thick at berm toe	139.2	123.2	16.0	200yr + 4'	135.8	0.35 [Toe of levee, blanket=16.1'] 0.92 [Toe of berm, blanket=10.6']	No breakout above seepage berm	Considering the presence of Thermalito Afterbay on north of this reach and a lateral canal on west of this reach, total head boundary condition equal to the ground surface elevation was used for analysis. The ground surface head boundary condition was assigned along the landside vertical face in the model at 1000 feet away from the levee. Gravel (GP-GM, 3-7% fines) was used for seepage berm material. ASTM C33 fine aggregate was used as filter drain. Geotextile separator was not modeled in the analysis. Exit gradients are calculated through the clay blanket at the toe of levee and toe of berm. Responding to DWR comment ID EXT/Reach41-002, and USACE comment ID #14, the filter compatibility check has been performed (see appendix C).	1.77		-	Waterside Levee Slope: 2.1H to 2.7H:1V Levee Slope Height = 16 feet Bench Width to Bank Slope = > 35 feet Bank Slope: NA Bank Slope Height = NA
					200yr + 1'	132.8	0.19 [Toe of levee, blanket=16.1'] 0.70 [Toe of berm, blanket=10.6']	No breakout above seepage berm		1.77		1.65	

[1]Measured from the landside toe to the top of the levee.
[2]Assumed a 26-foot drop for Reaches 2 to 17 and a 15-foot drop for Reaches 18 to 41.

ENCLOSURE L

MEMORANDUM ON ULE AND SBFCA SEISMIC EVALUATIONS

REVIEW
of the ULE – Sutter Task Area: Feather River West (Right Bank) Levee
Seismic Vulnerability Issues

Reviewed Documents

The reviewed documents were:

- (1) Phase 1 Preliminary Geotechnical Evaluation Report (P1GER) – Sutter Study Area, prepared by URS for DWR, dated March 2008 (Chapter 5.4 and Appendix C-3)
- (2) Geotechnical Design Recommendations Report – Feather River West Levee Project Segment 7, prepared by Blackburn Consulting for SBFCA, dated October 2012 (Sections 2.6.5 and 8.4, Appendix A, Appendix B(-1), Appendix B-2, Appendix D
- (3) Geotechnical Design Recommendations Report – Feather River West Levee Project Segments 1 through 6, prepared by URS for SBFCA, dated October 2012 (Sections 6.5 and 8.3, Appendix A, Appendix B(-1), Appendix B-2, Appendix D

Additional data were obtained from the following documents:

- Phase 1 Geotechnical Data Report (P1GDR) – Sutter Study Area, prepared by URS for DWR, dated November 2008

Seismicity Assessment

Ground accelerations for 100-year, 200-year and 500-year return period earthquake events were estimated using information from the Guidance Document for Geotechnical Analyses version 5 (URS, 2007). The evaluations in document (1) considered all 3 return periods; documents (2) and (3) used the 200-year return period only. The computed peak ground acceleration values in document (1) are compared below with data from USGS Interactive Deaggregations (2008) recommended by USACE, for slightly different return periods:

Table 1 – Seismicity Assessment in Document (1)

Levee Segment	Return Period (years) URS / USGS*	Peak Ground Acceleration	
		Per URS	Per USGS
Sutter Bypass and Wadsworth Canal	100 / 108	0.11g	0.11g
	200 / 224	0.14g	0.15g
	500 / 475	0.18g	0.19g to 0.20g
Feather River South	100 / 108	0.10g to 0.11g	0.10g to 0.11g
	200 / 224	0.13g to 0.14g	0.14g to 0.15g
	500 / 475	0.16g to 0.18g	0.18g to 0.19g
Feather River North	100 / 108	0.10g	0.10g
	200 / 224	0.13g	0.13g to 0.14g
	500 / 475	0.16g	0.18g

Note: * USGS date are available for 50% probability of exceedance in 75 years, or 108-year average return period; 20% probability of exceedance in 50 years, or 224-year average return period; and 10% probability of exceedance in 50 years, or 475-year average return period.

It is evident that the URS evaluation and our independent one are in good agreement. Differences of up to 0.02g for the 500-year return period event are not significant.

It is noted that of major importance in PGA evaluation is the assumed amplification of the seismic motion, which depends mainly on the shear wave velocity of the upper 30 meters near the ground surface, Vs30. URS assumed in all cases Vs30 = 800 fps = 240 m/s. For verification of this assumption shear wave velocity measurements were not available, but 20 borings with SPT's to a depth of at least 100 feet were used to evaluate Vs30 through correlations with N_{60} available in literature, as shown in Figure 1 [Figures 23 for large data base of all types of soils and Figure 24 for granular soils, from USACE WES (1987)]. The 20 borings considered were along the Feather River South levee system (MA3), between Stations 2204+77 and 3211+79.

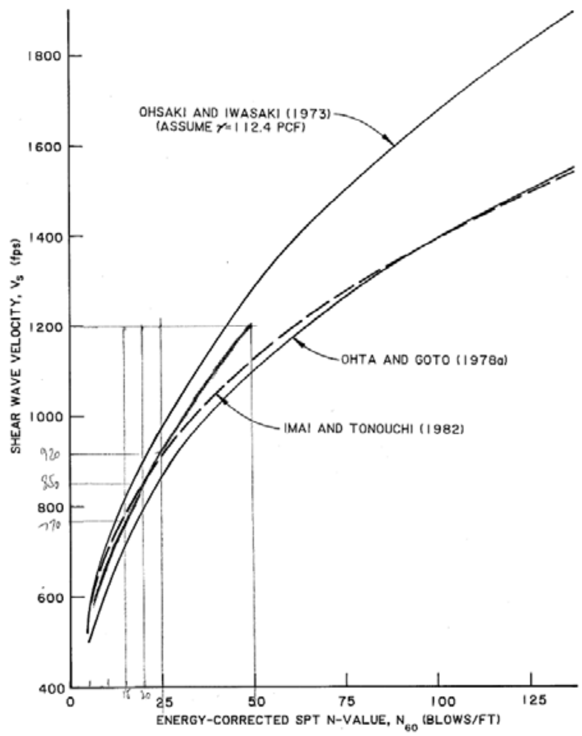

Figure 23. Comparison of results for N versus V_s correlations (proposed by select studies)

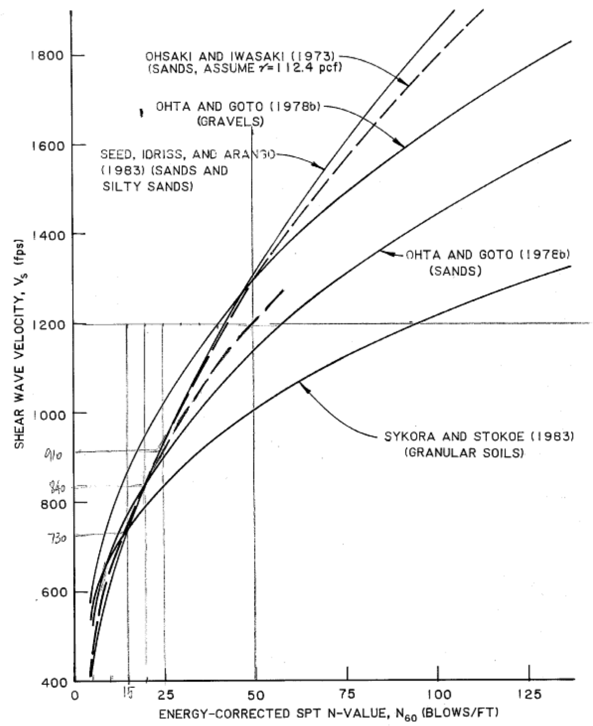

Figure 24. Comparison of results for N versus V_s correlations in granular soils (proposed by select studies)

Figure 1. Excerpt of USACE WES (1987): Average curves were considered that pass through the point represented by N_{60} = 50 and Vs = 1200 fps, which is the boundary between stiff soil and soft rock in USGS classification.

The harmonic mean (which is the average value definition appropriate for velocity and, therefore, for N_{60} when it is used as a proxy for Vs) for each boring varied between 12.3 and 31.0, with an arithmetic mean of them of 19.6; the corresponding Vs value on the average curves in Figure 1 is 844 fps and 834 fps, respectively. The selected value of 800 fps is considered appropriate, slightly conservative with respect to the average curves but close to the middle of the range corresponding to the various recommended criteria. It is mentioned that the same value of 800 fps (240 m/s) was used in conjunction with the evaluation of PGA per USGS, as presented in the last column of Table 1.

The expected earthquake magnitudes (Mw) associated with the three return period events listed in Table 1 are:
- Mw = 6.5 for 100-year return period event,
- Mw = 7.0 for 200-year return period event, and
- Mw=8.0 for 500-year return period event, per URS study.

The USGS reference confirmed these estimates, except for the 500-year return period event where Mw can be as high as 9.0 for Feather River North, probably due to relative proximity to the Cascadia Subduction zone.

In summary, the reviewer found the seismicity assessment in document (1) correct.

Documents (2) and (3) used for the seismicity assessment the ULE Hazard Map for a 200-year Return Period developed for Marysville, Sacramento, and Stockton regions. This map provides PGA assuming stiff soil condition (V_{s30} = 335 m/s); this is different from V_{s30} = 240 m/s used in document (1) and we found acceptable and corresponds to stiff soil outcropping motion. Consequently, there is a significant difference between URS estimates and the values recommended by USGS for a similar return period but a softer soil condition. See the comparison in the following table:

Table 2 – Seismicity Assessment in Documents (2) and (3)

Levee Segment	Return Period (years) URS / USGS*	Peak Ground Acceleration	
		Per URS	Per USGS
Feather River South Reaches 1 – 6	200 / 224	0.125g	0.14g
Feather River South Reaches 7 – 22	200 / 224	0.125g	0.13g to 0.14g
Feather River North Reaches 23 – 41	200 / 224	0.11g	0.13g

Based on the graph in Figure 1 we estimate V_{s30} = 335 m/s to correspond to N_{60} = 42.5. Our spot checks found, as shown before, an average N_{60} of about 20, with the range of 12 to 31 for the evaluated locations (20 borings along the Feather River South levee). It is emphasized that the representative average for an individual boring is the harmonic mean, as N_{60} is a proxy for shear wave velocity; consequently the low values occurring generally near the surface have a much higher weight than the high values at greater depth on the mean value.

The non-conservative assumption with regard to PGA selection may be compensated by a conservative triggering criterion, as shown below.

Liquefaction Triggering Analyses

As specified in Chapter 5.4 “Seismic Vulnerability Analyses” of the report (1), liquefaction analyses have been done for the following levee units:

In Northern Sutter Bypass (Tisdale Bypass to Wadsworth Canal) and Wadsworth Canal (ENGEO analysis sections), liquefaction triggering analyses for 500-year return period earthquake events were performed using the results of SPT borings and 2 year flood elevations. No seismic deformation analyses were performed in these sections. The liquefaction triggering analyses were not available and, therefore, not reviewed. However, it is believed the same methodology as for the unit listed below was used, so the comments below should apply to this unit also.

In Southern Sutter Bypass (Tisdale Bypass to Feather River Confluence) and Feather River South sections (URS analysis sections), liquefaction triggering analyses for 500-year, 200-year, and 100-year return period earthquake events were performed using the results of SPT borings and typical winter water levels (February Mean). The liquefaction analyses were performed to the bottom of the exploration depths. Based on the results of the liquefaction analyses, seismic deformation analyses were performed in eight sections using the results of liquefaction analyses for 500-year return period earthquake event. If the results of the analyses indicated “probably uncompromised” seismic vulnerability class, no further analyses were performed. If the results of the analyses indicated “Possibly Compromised,” “Likely Compromised,” or “Compromised” seismic vulnerability classes, further analyses were performed using liquefaction analyses results for 200-year and 100-year return period earthquake events. The evaluations by URS will be discussed in detail in what follows.

In Feather River North sections (GEI analysis sections), liquefaction triggering analyses for 500-year, 200-year, and 100-year return period earthquake events were performed using the results of SPT borings and typical winter (February Mean) or summer (August Mean) water levels, which one was higher. The liquefaction analyses were performed to a depth of 50 feet below the bottom of the levee base. Based on the results of the liquefaction analyses, seismic deformation analyses were performed in nine sections using the results of liquefaction analyses for 500-year return period earthquake event. Neither the results of the liquefaction triggering analysis nor those of seismic deformation analyses were available, but it is assumed the same procedures as URS did for Feather River South sections were used.

URS spreadsheet for liquefaction triggering analysis is based on the procedure recommended by Cetin et al. (2004), that is slightly more conservative than the procedure by Youd et al. (2001) recommended in USACE Guidelines (draft ETL 1110-2-580); however, both procedures are considered state-of-the-practice and can alternatively be used.

The analyses were performed on 20 borings with SPT drilled along the Feather River South, practically one representative boring for each reach, M through Z4 (Stations 2166+00 to 3295+59). The results for the first two analyzed borings are plotted in Figure 2.

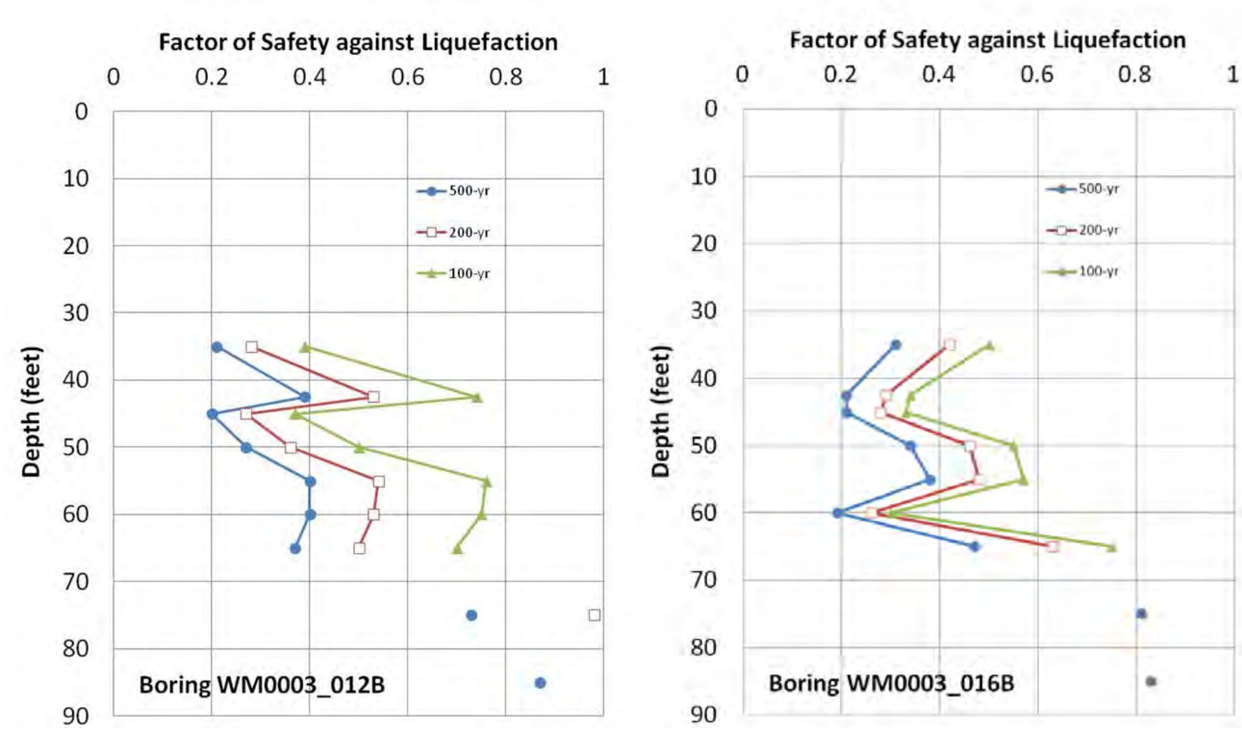

Figure 2. Variation with depth of the factor of safety against liquefaction for two locations in the southern part of the Feather River South levee system (Stations 2204+77 and 2282+82).

The plots in Figure 2 are representative for many other locations along the Feather River South. With the exception of some deep, isolated layers (at 75 feet and 85 feet), the easiest liquefiable layer (in these cases between depths of 35 feet and 65 feet) liquefies under the action of the 500-year return period event, but also under the 200-year and 100-year. It makes little difference if FS_{liq} is of the order of 0.2-0.4 or 0.3-0.5 or 0.4-0.7; in all cases the 30-foot layer is expected to fully liquefy. In all three cases the residual undrained strength is expected to mobilize, which theoretically depends on N (SPT) only and is independent on the FS_{liq}.

It is noted that in the cases presented in Figure 2 the liquefiability index ($N_{1,60\text{-}cs}$) was relatively low, with the 33-percentile average of the 30-foot liquefiable layer of 8.9 and 5.0, respectively. There is, however, little difference in other locations. Figure 3 presents the results obtained in two locations at the northern limit of the Feather River South levee system, where the liquefiability index ($N_{1,60\text{-}cs}$) was relatively higher, with the 33-percentile average of the 20-foot or 30-foot liquefiable layer of 11.6 and 11.0, respectively. It is evident that in this case also the liquefaction occurs with all three different return period considered.

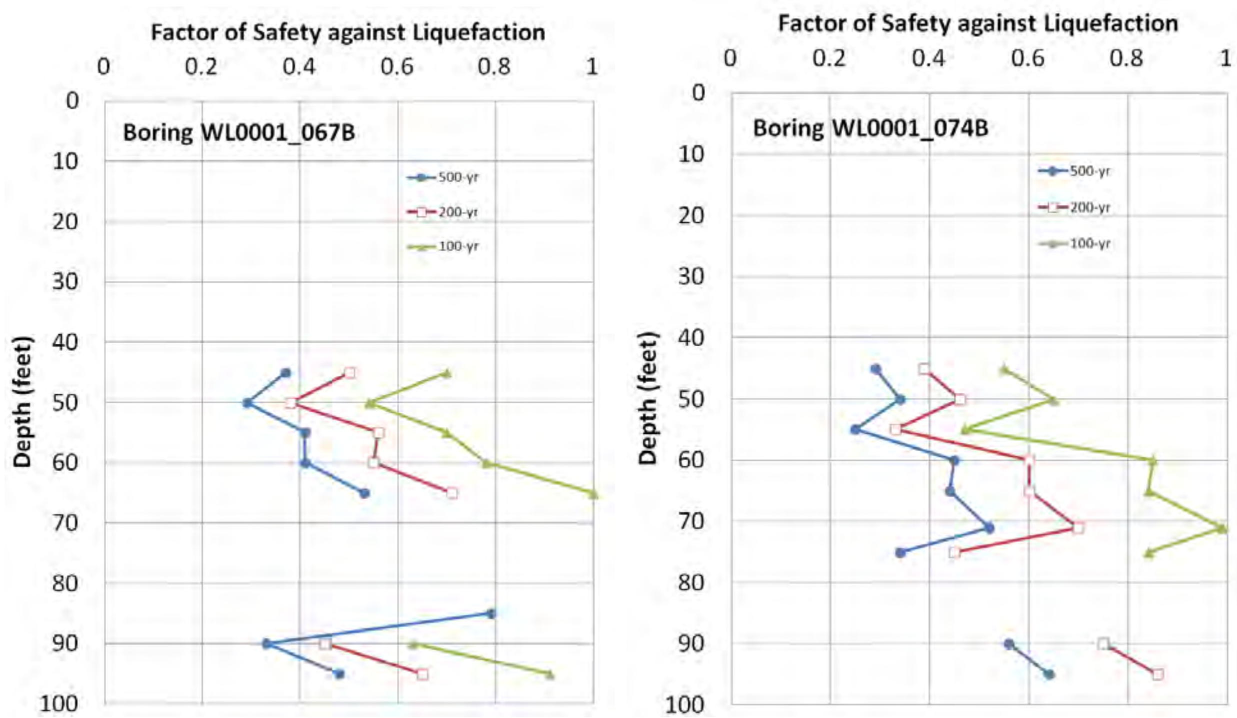

Figure 3. Variation with depth of the factor of safety against liquefaction for two locations in the northern part of the Feather River South levee system (Stations 3027+34 and 3073+29).

Documents (2) and (3) used similar procedure for triggering analysis, but for 200-year return period only. Not all details were available, but the DWR Guidance Document, Revision 10, developed by URS was used in both cases.

In general, the evaluation of the liquefaction susceptibility is satisfactory with all three documents.

<u>Levee Stability Evaluation</u>

Once significant liquefaction of the foundation soil was found probable, stability analysis is performed in steps:

- First, post-earthquake slope stability analyses were performed using limit equilibrium software. The computer program used is not specified in documents (2) and (3); with document (1) Utexas4 was apparently used. In accordance with USACE requirements, the DWR-ULE Guidance requires the Spencer approach be the basis for all stability analyses. We believe this requirement was met.

An important parameter in post-earthquake analyses is the residual strength of liquefiable materials. As with USACE (2013) draft guidelines the correlation proposed by Seed and Harder (1990) is recommended. Correctly, for estimating the undrained residual strength from the Seed & Harder plot, the lower 1/3 curve of the upper and lower range limits was used.

USACE (2013) recommends to alternatively use Seed and Harder (1990) and Olson and Stark (2002) procedures and to consider both results as possible occurrences. Although we would prefer both procedures to be considered in evaluation, the URS recommended approach is state-of-the-practice and acceptable.

Also in accordance with USACE concepts, if the calculated post-earthquake FS, for any calculated potential failure surface, is less or equal to 1.0, then the study-section should be considered unstable (i.e. "compromised") and no further evaluations are needed. In addition to this criterion, USACE (2013) Guidelines state that for intermittent-hydraulically loaded levees (like Feather River levees) further evaluation is not needed if FS > 1.2 and the levee should be considered stable and continuing to function after the design earthquake occurrence.

- Next step was to perform a pseudo-static stability analysis with a horizontal seismic coefficient not well defined; the suggested value (0.5 K_{max}, where K_{max} is the maximum seismic coefficient for a potential sliding mass) is not justified. It is noted that this evaluation is not mentioned in the Guidance Document (URS, 2012) or the USACE (2013) Guidelines. However, it was included in a Technical Memorandum prepared by URS in October 2012 for SBFCA and apparently accepted by technical reviewers (Les Harder, Jr., Lelio Mejia, John Hess, and Francke Walberg).

If the minimum calculated FS for this pseudo-acceleration level ($K = 0.5\ K_{max}$) was greater than 1.0, no additional evaluation was considered necessary and the study section was classified as probably uncompromised and further refinement for the vulnerability evaluation was considered unnecessary.

It is noted that the criterion FS > 1.0 for the pseudo-static stability analysis is in lieu of the USACE supported (USACE, 2013) criterion of FS > 1.2 for the post-earthquake stability analysis. More evaluations are necessary for deciding which criterion is the most appropriate.

Deformation Analysis

Deformation analysis should be done if the study section could not be classified through previous analyses as "Compromised" or "Probably Uncompromised". The goal of the seismic deformation analyses is to evaluate post-seismic vertical displacements at the levee crest, classification into the other two categories ("Likely Compromised" and "Possibly Compromised") be made possible. USACE (2013) Guidelines offer several alternate methods for estimation of displacements, including that based on the Newmark approach which is the only one recommended by URS Technical Memorandum and used in documents (2) and (3).

The recommended approach is based on the Makdisi and Seed, 1978 method, but replaces the original range of results with an average one. Figure 4 compares the range in the original Makdisi & Seed study (interpolated for M7 between the original ranges provided for M6.5 and M7.5) with the average curve recommended in URS Technical Memorandum. The obtained Newmark displacement should be multiplied by 0.7 to obtain an estimate of the freeboard loss (vertical displacement).

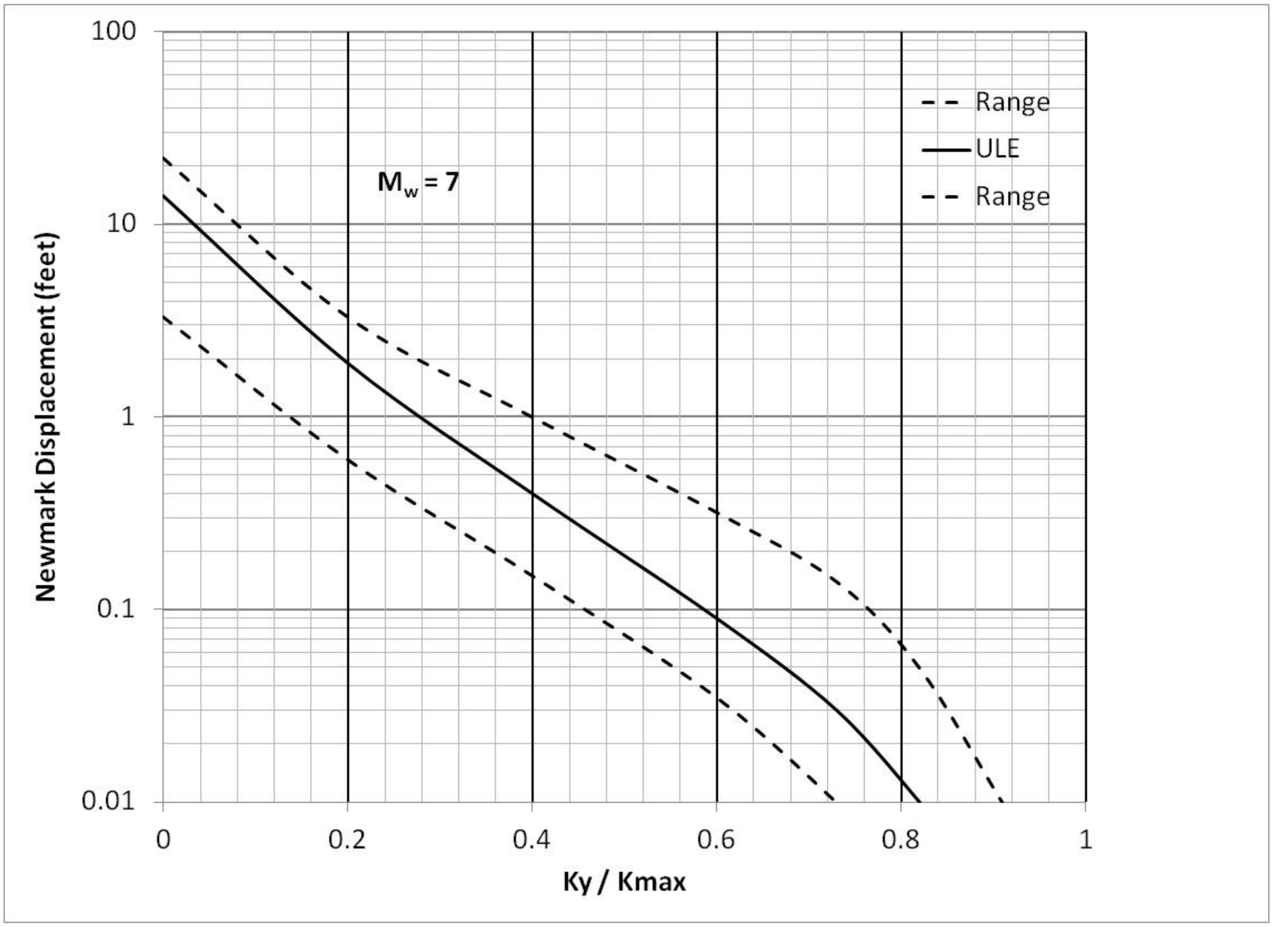

Figure 4. Comparison between the URS (2012) recommended correlation for ULE (solid line) and the range indicated by Makdisi and Seed (1978) (dashed lines).

It is evident that the actual scatter of the results, which is not mentioned in URS (2012) Guidelines, for a credible classification based on more or less than 1 foot remaining freeboard after the seismic deformation. For example, if the original freeboard was 3 feet, for a remaining 1 foot of freeboard the estimated Newmark deformation should be 2 feet/0.7 = 3 feet. From Figure 4, an estimate of 3 feet would correspond to K_y / K_{max} = 0.15, but the range corresponding to this ratio is 0.9 feet to 5 feet. Based on the average curve (solid line) the seismic vulnerability classification would be on the boundary between "Possibly Compromised" and "Likely Compromised" (remaining freeboard = 1 foot), but based on the possible range the study section can be classified either "Likely Compromised" (no remaining freeboard) or "Probably Uncompromised" (0.9 x 0.7 = 0.6 feet vertical deformation, which leaves a freeboard of 2.4 feet and represents less than 5% of landside levee height). It is considered that the procedure is not sensitive enough for a credible seismic vulnerability classification.

It is also noted that in the main text of the document (3), section 6.5, the main parameter used in seismic vulnerability classification is "the amount of deformation", but in the referenced Appendix B-2 "Technical Memorandum" the corresponding parameter is "the amount of vertical deformation". There is significant difference between the two parameters, because the amount of deformation can be the Newmark deformation or the displacement in any direction, including horizontal.

Conclusions

All reviewed documents are satisfactory with reference to liquefaction assessment, evaluation of levee stability, and seismic vulnerability classification in class “compromised”. There are not currently available trustable procedures for classification in the other classes. USACE (2013) Guidelines recommend using 2-3 procedures of several presented in appendices, including the procedure recommended and used by URS, but there is no indication if one is better than the others. More experience with using the proposed methods is necessary for an acceptable methodology of seismic vulnerability classification.

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USACE WES (1987). Examination of Existing Shear Wave Velocity and Shear Modulus Correlations in Soils, by David Sykora, *Miscellaneous Paper GL-87-22*, 101 pages.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B. şi Stokoe, K. H. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. *Journal* *of Geotechnical and Geoenvironmental Engineering*, *ASCE*, Vol. 127, No.10, October, pp 817-833.

ENCLOSURE M

REPORTS PREPARED BY OUTSIDE AGENCIES (AVAILABLE AS SEPARATE FILES UPON REQUEST)

DWR ULE P1GER
DWR ULE P1GDR
DWR ULE SGDR
DWR NULE GAR
DWR NULE GAR Appendix C
DWR NULE GAR Appendix E
SBFCA Geotechnical Design Recommendations for the PFR
SBFCA Final GDR
SBFCA Final GDRR
Modified SBFCA 65% Plans

WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

Project No. 105638

SPK – Sutter Basin Project

Two Alternatives for the Sutter Basin Project, as presented by Sacramento District, have 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 scopes, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the 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 October 10, 2013, the Cost MCX certifies the estimated total project cost of the two alternatives:

ALTERNATIVE SB-7
FY 2014 Price Level: $391,840,000
Fully Funded Amount: $440,530,000

ALTERNATIVE SB-8
FY 2014 Price Level: $688,930,000
Fully Funded Amount: $791,970,000

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.

Kim C. Callan, PE, CCE, PM1
Chief, Cost Engineering MCX
Walla Walla District

US Army Corps of Engineers®

****TOTAL PROJECT COST SUMMARY****

10/10/2013

THIS ESTIMATE IS BASED ON THE SCOPE CONTAINED IN THE DRAFT FEASIBILITY REPORT, ALT. SB-7

PROJECT: Sutter Basin TPCS

LOCATION: CALIFORNIA

U. S. ARMY CORPS OF ENGINEER, SACRAMENTO DISTRICT

P.O.C.: JEREMIAH A. FROST, CHIEF, COST ENGINEERING SECTION

Current MCACES Estimate Prepared: 25-Jul-2013

Effective Price Level (EPL): 1-Oct-2013

		ESTIMATED COST				PROGRAM YEAR(BUDGET EC) 2014 EFF. PRICE LEVEL DATE:1-Oct-2013 PROJECT FIRST COST				TOTAL PROJECT COST (FULLY FUNDED) SPENT THRU: 1-Oct-2013				
WB NO.	Civil Works FEATURE DESCRIPTION	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC. (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	COST ($K)	ESC. MIDPT(%)	COST ($K)	CNTG ($K)	FULLY FUNDED ($K)
Index Codes: 0 - no esc. applied; A - Administration; C - Combined indexes; All other codes used coincides with the Code of Accounts.														
		Contingency Applied To Remaining Cost Only												
	FEDERAL COSTS													
6	FISH & WILDLIFE FACILITIES	5,032	1,006	20	6,038	0.00	5,032	1,006	6,038	0	12	5,611	1,122	6,733
11	LEVEES & FLOODWALLS	176,205	63,717	36	239,922	0.00	176,205	63,717	239,922	0	11	196,085	70,906	266,991
18	CULT. RESRC. PRESERV. (1	1,655	598		2,253		1,655	598	2,253	0		1,841	665	2,506
	Data Recovery	1,200	433		1,633	0.00	1,200	433	1,633	0		1,334	482	1,816
	Inventory/Evaluation/Mitigation Costs	455	165	36	620	0.00	455	165	620	0	11	507	183	690
	SUBTOTAL FEDERAL & NON-FEDERAL CONSTRUCTION COSTS	182,892	65,321		248,213		182,892	65,321	248,213	0		203,537	72,693	276,230
1	LANDS & DAMAGES, Admin (2	6,952	348	5	7,300	0.00	6,952	348	7,300	0	17	8,168	408	8,576
30	PLAN/ENGINEERING/DESIGN	32,622	11,797	36	44,419	0.00	32,622	11,797	44,419	0	18	38,534	13,934	52,468
31	CONSTRUCTION MANAGE'MT	15,406	5,570	36	20,976	0.00	15,406	5,570	20,976	0	23	18,943	6,849	25,792
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION	237,872	83,036		320,908		237,872	83,036	320,908	0		269,182	93,884	363,066
	NON-FEDERAL CONTRIBUTION (-)	-48,533	-17,105		-65,638		-48,533	-17,105	-65,638	0		-56,289	-19,847	-76,136
	TOTAL FEDERAL COSTS	**$189,339**	**$65,931**		**$255,270**		**$189,339**	**$65,931**	**$255,270**	**$0**		**$212,893**	**$74,037**	**$286,930**
	NON-FEDERAL COSTS													
1	LANDS AND DAMAGES	31,811	10,579	33	42,390	0.00	31,811	10,579	42,390	0	8.5	34,523	11,481	46,004
2	RELOCATIONS	20,962	7,580		28,542		20,962	7,580	28,542	0		23,105	8,355	31,460
	Relocations Construction Cost	16,376	5,922	36	22,298	0.00	16,376	5,922	22,298	0	10	18,074	6,536	24,610
	Plan/Engineering/Design	2,948	1,066	36	4,014	0.00	2,948	1,066	4,014	0	8.8	3,209	1,160	4,369
	Construction Mangement	1,638	592	36	2,230	0.00	1,638	592	2,230	0	11	1,822	659	2,481
	SUBTOTAL NON-FEDERAL	52,773	18,159		70,932		52,773	18,159	70,932	0		57,628	19,836	77,464
	NON-FEDERAL CONTRIBUTION (+)	48,533	17,105		65,638		48,533	17,105	65,638	0		56,289	19,847	76,136
	TOTAL NON-FEDERAL COSTS	**$101,306**	**$35,264**		**$136,570**		**$101,306**	**$35,264**	**$136,570**	**$0**		**$113,917**	**$39,683**	**$153,600**
	TOTAL FEDERAL AND NON-FEDERAL COSTS	**$290,645**	**$101,195**		**$391,840**		**$290,645**	**$101,195**	**$391,840**	**$0**		**$326,810**	**$113,720**	**$440,530**

GENERAL NOTES

(1 Cultural Resources Preservation costs was provided by Cultural Resources Archaeologist.
(2 Federal administrative costs for non-Federal land acquisition.
(3 The Fully Funded cost estimate was prepared in compliance with Indexes used from CWCCIS reflecting OMB future rates Mar. 31, 2013
(4 01 Account for Land and Damages cost are from Real Estates.
(5 06 Account Fish and Wildlife Cost was provided by SPK Environmental Planning.
(6 30 Account Planning, Engineering and Design and 31 Account Construction Management cost was provided by its respective organizations.

CONTINGENCY RATIONALE

(A CONTINGENCIES USED WAS DERIVED BY THE COST RISK ANALYSIS PROCESS AND IS BASED ON A 80% CONFIDENCE LEVEL

TOTAL FEDERAL COSTS	**$286,930**
TOTAL NON-FEDERAL COSTS	**$153,600**
THE MAXIMUM PROJECT COSTS	**$440,530**

______________________________ **CHIEF, COST ENGINEERING**

______________________________ **PROJECT MANAGER**

______________________________ **CHIEF, REAL ESTATE**

****TOTAL PROJECT COST SUMMARY****

10/10/2013

THIS ESTIMATE IS BASED ON THE SCOPE CONTAINED IN THE DRAFT FEASIBILITY REPORT, ALT. SB-8

PROJECT: Sutter Basin TPCS

LOCATION: CALIFORNIA

U. S. ARMY CORPS OF ENGINEER, SACRAMENTO DISTRICT

P.O.C.: JEREMIAH A. FROST, CHIEF, COST ENGINEERING SECTION

Current MCACES Estimate Prepared: 25-Jul-2013

Effective Price Level (EPL): 1-Oct-2013

PROGRAM YEAR(BUDGET EC) 2014

EFF. PRICE LEVEL DATE:1-Oct-2013

TOTAL PROJECT COST (FULLY FUNDED)

			ESTIMATED COST					PROJECT FIRST COST			SPENT THRU: 1-Oct-2013				
WB NO.	Civil Works FEATURE DESCRIPTION		COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC. (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	COST ($K)	ESC. MIDPT(%)	COST ($K)	CNTG ($K)	FULLY FUNDED ($K)
	Index Codes: 0 - no esc. applied; A - Administration; C - Combined indexes; All other codes used coincides with the Code of Accounts.														
			Contingency Applied To Remaining Cost Only												
	FEDERAL COSTS														
6	FISH & WILDLIFE FACILITIES		6,330	1,265	20	7,595	0.00	6,330	1,265	7,595	0	14	7,226	1,445	8,671
11	LEVEES & FLOODWALLS		306,367	106,488	35	412,855	0.00	306,367	106,488	412,855	0	13	347,604	120,821	468,425
18	CULT. RESRC. PRESERV. (1		3,030	1,076		4,106		3,030	1,076	4,106			3,399	1,207	4,606
	Federal Obligations from NED Cost.		1,655	598		2,253		1,655	598	2,253			1,841	665	2,506
	Data Recovery		1,200	433		1,633		1,200	433	1,633	0		1,334	482	1,816
	Inventory/Evaluation/Mitigation Costs		455	165		620		455	165	620	0		507	183	690
	Cost Beyond NED Cost.		1,375	478		1,853		1,375	478	1,853	0		1,558	542	2,100
	Data Recovery	18	1,000	348	35	1,348	0.00	1,000	348	1,348	0	13	1,134	394	1,528
	Inventory/Evaluation/Mitigation Costs	18	375	130	35	505	0.00	375	130	505	0	13	424	148	572
	SUBTOTAL FEDERAL & NON-FEDERAL CONSTRUCTION COSTS		315,727	108,829		424,556		315,727	108,829	424,556	0		358,229	123,473	481,702
1	LANDS & DAMAGES, Admin (2		11,143	557	5	11,700	0.00	11,143	557	11,700	0	22	13,549	677	14,226
30	PLAN/ENGINEERING/DESIGN		56,285	19,565	35	75,850	0.00	56,285	19,565	75,850	0	22	68,804	23,916	92,720
31	CONSTRUCTION MANAGE'MT		26,580	9,239	35	35,819	0.00	26,580	9,239	35,819	0	27	33,791	11,746	45,537
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION		409,735	138,190		547,925		409,735	138,190	547,925	0		474,373	159,812	634,185
	NON-FEDERAL CONTRIBUTION(-)		-220,396	-72,259		-292,655		-220,396	-72,259	-292,655	0		-261,480	-85,775	-347,255
	TOTAL FEDERAL NED COSTS		**$189,339**	**$65,931**		**$255,270**		**$189,339**	**$65,931**	**$255,270**	**$0**		**$212,893**	**$74,037**	**$286,930**
	NON-FEDERAL COSTS														
1	LANDS AND DAMAGES		41,795	11,751	28	53,546	0.00	41,795	11,751	53,546	0	11	46,222	12,995	59,217
2	RELOCATIONS		64,900	22,559		87,459		64,900	22,559	87,459	0		73,143	25,425	98,568
	Relocations Construction Cost		50,703	17,624	35	68,327	0.00	50,703	17,624	68,327	0	13	57,271	19,907	77,178
	Plan/Engineering/Design		9,127	3,172	35	12,299	0.00	9,127	3,172	12,299	0	11	10,123	3,519	13,642
	Construction Management		5,070	1,763	35	6,833	0.00	5,070	1,763	6,833	0	13	5,749	1,999	7,748
	SUBTOTAL NON-FEDERAL		106,695	34,310		141,005		106,695	34,310	141,005	0		119,365	38,420	157,785
	NON-FEDERAL CONTRIBUTION (+)		220,396	72,259		292,655		220,396	72,259	292,655	0		261,480	85,775	347,255
	Non-Federal Contribution - NED		48,533	17,105		65,638		48,533	17,105	65,638	0		56,289	19,847	76,136
	Additional Cost Above NED		171,863	55,154		227,017		171,863	55,154	227,017	0		205,191	65,928	271,119
	TOTAL NON-FEDERAL COSTS		**$327,091**	**$106,569**		**$433,660**		**$327,091**	**$106,569**	**$433,660**	**$0**		**$380,845**	**$124,195**	**$505,040**
	TOTAL FEDERAL AND NON-FEDERAL COSTS		**$516,430**	**$172,500**		**$688,930**		**$516,430**	**$172,500**	**$688,930**	**$0**		**$593,738**	**$198,232**	**$791,970**

GENERAL NOTES

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(2 Federal administrative costs for non-Federal land acquisition.
(3 The Fully Funded cost estimate was prepared in compliance with Indexes used from CWCCIS reflecting OMB future rates Mar. 31, 2013
(4 01 Account for Land and Damages cost are from Real Estates.
(5 06 Account Fish and Wildlife Cost was provided by SPK Environmental Planning.
(6 30 Account Planning, Engineering and Design and 31 Account Construction Management cost was provided by its respective organizations.

(A CONTINGENCIES USED WAS DERIVED BY THE COST RISK ANALYSIS PROCESS AND IS BASED ON A 80% CONFIDENCE LEVEL

	DOLLAR(K)
TOTAL FEDERAL COSTS	**$286,930**
TOTAL NON-FEDERAL COSTS	**$505,040**
THE MAXIMUM PROJECT COSTS	**$791,970**

____________________ **CHIEF, COST ENGINEERING**

____________________ **PROJECT MANAGER**

____________________ **CHIEF, REAL ESTATE**

US Army Corps of Engineers.

Sacramento District
Engineering Division

Sutter Basin Pilot Feasibility Report - Environmental Impact Report / Supplemental Environmental Impacts Statement

Butte and Sutter Counties, California

COST ENGINEERING APPENDIX

Oct 2013

TABLE OF CONTENTS PAGE

1. BASIS OF ESTIMATE

COST ESTIMATE FOR DRAFT ALTERNATIVE ARRAY

Cost estimates were developed to compare the draft array of alternatives presented in the Feasibility Study Report. These cost estimates were utilized to select the final array of alternatives and were based on a level 4 per requirement of ER 110-2-1302. In developing the reconnaissance level cost estimates of the various measures and alternatives (combined measures) for the Sutter Basin project, the Cost Engineering team utilized a methodology wherein costs for levee improvements or new levees (sans relocations) were developed using a parametric spreadsheet based on typical cross sections for differing types of levee improvements. Costs for relocations and construction other than that directly related to the levee were compiled based on either 1) historical costs - past levee projects in the vicinity of Sacramento, 2) estimating software MII (MCACES, 2nd Generation) or PACES, or 3) based on a percentage of construction costs. In lieu of the time constraints of the 24-month fast-track pilot study schedule, these methods were used for preparing costs for the purpose of selecting the final array of alternatives. Refer to Attachment A for further detail on the background and approach to developing cost estimates for the draft array of alternatives.

FEASIBILITY COST ESTIMATES FOR FINAL ALTERNATIVE ARRAY

The baseline feasibility cost estimates for the final array of alternatives (SB-1, SB-7 & SB-8) were developed using the design drawings provided by Civil Design. The quantities take-off calculations were provided by the Sacramento District's Civil Design section to produce the feasibility estimates. There are 41 Reaches spanning approximately 40 miles of levee. The breakdown of the alternatives by reach is further described in paragraph 2. Most of the geotechnical levee repair for the alternatives is to be accomplished with a soil-bentonite slurry wall constructed in the centerline of the levee. There are locations where jet grout, seepage berm or relief wells are also utilized but they are small in magnitude relative to the SB slurry wall. Of the 41 Reaches, there are several Reaches where no levee work is proposed (Reach 14, 15, 29, and 39).

Due to the large scope, the project is broken into construction contracts. To facilitate comparison to the local sponsor Early Implementation Project (EIP), similar contract reaches were utilized. These contracts have no impact on the total project cost. Based on the anticipated yearly funding availability, the reaches were combined in more manageable contracts, totaling approximately $40 million per contract per year for the feasibility study. Refer to tables in paragraph 5 for breakdown of contracts by reach.

2. PROJECT SCOPE/DESCRIPTION

There are three final Alternatives (SB-1, SB-7 & SB-8) to be evaluated for selection of the Recommended Plan. Alternative SB-1 is the No Action plan, which is to do nothing; hence a cost estimate was not created. It is assumed that Alternative SB-1 has no federal cost.

Alternative SB-7 is to Fix-in-Place the Feather River West Levee from the Sunset Weir to Laurel Avenue. The Alternative SB-7 project footprint extends from FRWL Reach 2 through 21. Alternative SB-8 is to Fix-in-Place the Feather River West Levee from Thermalito Afterbay to Laurel Avenue. The SB-8 project footprint extends from FRWL Reach 2 through 41. Alternative SB-8 is an incremental addition to Alternative SB-7 and all elements in Alternative SB-7 exist in Alternative SB-8.

Alternative SB-8 is almost equivalent to the Feather River West Levee Project (FRWLP) with the exception of Reach 6. At Reach 6, the Sponsor has constructed the Star Bend Setback Levee. However, during plan formulation the PDT proposed to have Reach 6 as a Fix-In-Place levee in lieu of Setback Levee because it is more cost effective. The Sponsor is seeking credit for work at this location. An estimate for the Star Bend Setback levee was created for cost comparison.

The designs for Alternatives SB-7 and SB-8 are similar in terms of levee remedial methods needed to reduce flood risk to the Sutter Basin. The vast majority of levee remediation is to reduce seepage by constructing a soil-bentonite slurry cutoff wall through the centerline of the levee and rebuild the levee to pre-project geometry. At some locations, seepage berms, relief wells, deep-soil-mixing, jet grout cutoff walls, canal relocations, and slight levee relocations to provide O&M access roads are included but they are minor relative to the soil-bentonite cutoff wall construction. Detail of the design remedial methods can be found in the Civil Design Appendix.

Along the FRWL, there are abandoned utilities that need to be removed. Active utilities such as pressure pipes, irrigation pipes, drainage pipes, electrical, sewer, gas, cable and water lines are to be removed and replaced in order to construct the soil-bentonite cutoff wall. Temporary utilities service is to be provided during the service outages. Roads on the levee crowns that must be removed in order to demolish or relocate utilities will be replaced.

3. MII COST ESTIMATE – NOTES & ASSUMPTIONS

The MII estimate used the QTO's provided by Civil Design. An estimate on the construction contracts and years for Alternatives SB-7 and SB-8 is presented below in paragraph 5.

MCACES PROGRAM & LIBRARIES

The program and libraries used for the MCACES cost estimate are as follows:

a. MII version 4.1 Build 4
b. 2010 Cost Book
c. 2011 EP1110-1-8 Equipment Library for Region VII.

OVERTIME

Overtime is included in the estimate. Assumption is 10 hour workdays, 6 days per week.

ACQUISITION PLAN

Construction contracts are assumed to be Invitation For Bid (IFB), Competitive, Unrestricted Full and Open Competition and all businesses may respond.

CONTRACTING PLAN

The prime contractor is expected to be an earthwork contractor responsible for site work, borrow site excavation, levee degradation, slurry wall construction, and levee embankment reconstruction. The utilities penetration relocation is expected to be done by a specialty subcontractor. Material hauling, hydroseeding, jet grouting, asphalt pavement, and other miscellaneous work are expected to be performed by subcontractors.

SITE ACCESS

The project footprint follows the existing levee along the west bank of the Feather River, northernmost from Thermalito Afterbay and extending southernmost to near the Sutter Bypass and Feather River confluence. The levee is assumed to be maintained by local Reclamation Districts (RD) and it is expected that the levee is accessible from the landside. Staging areas or stockpile areas are constructed every 2,500 lineal feet along or near the levee landside /waterside toes. Stripped topsoil material, aggregate base, and levee degrade material can be readily stockpiled in the staging areas. Haul routes for import/export material is expected to be on existing roads and highways (no barge transport). No new roadway for site access is expected to be constructed.

BORROW/DISPOSAL AREAS

Borrow sources identified by the sponsor are incorporated into the estimate. A material balance calculation was performed by SPK's Civil Design and Cost Engineering sections using sponsor QTO's for levee fill materials (Types 1, 2 & Random) available at each borrow site. It was concluded that there is enough material to satisfy the fill demand for Alternatives SB7 and SB8. The suitability of the borrow source/material has been evaluated by the SPK Geotech Section (please refer to Geotechnical Engineering Appendix for detail). Non-hazardous unsuitable fill material is assumed to be used to backfill the borrow pits. Other construction waste is assumed to be disposed of off-site in accordance with local, state, and federal regulations. HTRW waste is assumed to be absent from the project. Construction waste can be safely disposed of within a 30 mile radius of the site.

CONSTRUCTION METHODOLGY

The construction methodologies for the soil-bentonite slurry wall excavation and placement are considered to be standard, except for deep walls (greater than 85 feet). Below this depth a conventional long reach hydraulic excavator cannot be used. The method provided in the cost estimate opts for the contractor to utilize a deep-soil-mixing (DSM) method for a design depth of cutoff wall greater than 85 feet.

CONSTRUCTION WORK WINDOWS

Due to environmental and wildlife concerns (wildlife habitat, migratory season, mating season etc.) it is assumed that a normal construction season would typically span from the month of May through October. Typically, USACE and local flood agencies want the levee to be reconstructed by October due to the beginning of the storm season. This is a flood safety measure. Depending on local jurisdiction and permitting weather, construction tasks such as hydroseeding, asphalt pavement repair of levee crown, and associated work that does not undermine the structural integrity of the levee during a storm event may be permissible beyond October. The irrigation canal that runs parallel to the levee landside toe is operational from April through February. The construction window for work in the canal is limited from February through April. One approach for working around this limitation is to obtain an encroachment permit for a variance to work outside the normal construction season prior to working in the canal. Another approach is to install sheet pile cutoff walls to insure that the work within the levee does not lead to excessive seepage or possible failure of the canal bank. This second approach does not require a variance. For the purposes of the feasibility report, the estimate assumes installation of a sheet pile cutoff wall. Depending on the scope of work and pipe crossing type, each approach is site specific and will be more closely dealt with on a case by case basis in the PED phase.

UNIQUE TECHNIQUES OF CONSTRUCTION
In close proximity to existing bridge abutments, underground utilities, or railroad tracks, a jet grout cutoff wall is to be constructed in lieu of the slurry cutoff wall.

EQUIPMENT AND LABOR AVAILABILITY AND DISTANCE TRAVELED
The project is in Yuba City, an urban city environment and equipment & labor is readily available within a 100 mile radius of the site. No labor shortage is anticipated.

ENVIRONMENTAL CONCERNS
Environmental protection requires consideration of air, water, and land, and involves noise management, solid-waste management and management of other pollutants. In order to prevent or provide for abatement and control of any environmental pollution arising from construction, the Prime Contractor and Subcontractors in the performance of the contract shall comply with all applicable Federal, State, and local laws, as well as regulations concerning environmental pollution control and abatement. The Contractor shall use best management practices at all times to minimize the potential for environmental impacts.

LABOR RATES
This estimate meets the Davis Bacon wage rates for Davis Bacon Wage Determination for the State of California, General Decision Number: CA130009 04/05/2013 CA9 .

EQUIPMENT RATES
Equipment rates were obtained from quotes or verbal/telephone conversations and the MII 2011 EP1110-1-8 Equipment Library for Region VII.

MATERIAL COST
Material prices are obtained from vendor quotes, supply catalogs, previous estimates and the MII Cost Book.

SALES TAX
California State Sales tax is applied at 8.00%.

OMRR&R
The proposed project reaches of Alternatives SB-7 and SB-8 are currently maintained as part of the Federal Sacramento River Flood Control Project. The OMRR&R for the proposed project would be similar as the existing project. Therefore, no OMRR&R cost are included in the estimate. A qualitative analysis of the OMRR&R costs was performed to validate this assumption. Both alternatives are comprised almost entirely of installation of a soil-bentonite cutoff wall within the structural section of the levee. The levee will be reconstructed to existing pre-project geometry and meet USACE standards. The slurry wall will reduce the short term maintenance cost due to a reduction in seepage. The reconstruction of the upper half of the levee (side slopes, vegetation removal, grass re-establishment, and crown road replacement) will also reduce the short term maintenance cost. With the installation of the slurry wall, many of the existing relief wells can be decommissioned or converted to other functions and this would reduce short term maintenance costs. The Levee Safety requirements for typical levee cross-sections (side slopes, crown and O&M road widths, etc.) will somewhat increase the current maintenance costs due to a larger footprint of vegetation management. The replacement of utility and drainage pipe crossings would reduce maintenance costs in the short term. Overall, the short term OMRR&R will decrease. However, in long term the OMRR&R cost is about the same because the commitments remain unchanged.

LIFE CYCLE COST
A life cycle cost estimate was not performed for the study.

4. CONTRACTOR MARKUPS

Prime Contractor's Markups – Below is the breakdown of the Prime CTR markups.

Prime Contractor	Own Work	Sub Work
JOOH	10.00%	10.00%
HOOH	10.00%	10.00%
Profit	9.00%	5.00%
Bond	1.50%	1.50%

Subcontractors' Markups – Below is the breakdown of the general subcontractors' markups.

Piping/Relocation	Own Work
JOOH	8.00%
HOOH	10.00%
Profit	8.00%

Jet Grout	Own Work
JOOH	10.00%
HOOH	11.00%
Profit	8.00%

Paving	Own Work
HOOH	8.00%
Profit	8.00%

The contractor markups presented in the tables above are representative of past civil works estimates performed in the Sacramento region. Depending on the bidding environment and availability of work in the region, the contractor markups can be higher or lower but the markups are expected be near those shown above. It is assumed that the subcontractors will perform all of their own work and will not subcontract any portion of it.

In addition to the contractor markups, a direct cost markup for Small Tools is estimated at 1.50% of Labor costs.

5. CONSTRUCTION SCHEDULE (SEE ATTACHED)

Alternative SB-7 is expected to consist of five (5) construction contracts. Alternative SB-8 would consist of seven (7) construction contracts. With the exception of the Star Bend FIP contract, each contract is assumed to be completed in two construction season. Star Bend FIP is a relatively small contract and it is assumed it can be constructed concurrently in the same year with another contract. If funding permits, multiple contracts can be awarded in the same year. An approximation on the construction contracts and year(s) of construction is presented below. The schedule assumes the project gets authorized and appropriated through the construction window. This projection assumes that there is no funding shortage to implement the contract(s) in a given year. Other considerations in drafting the construction schedule includes public safety, availability of qualified contractors and special construction equipment, construction windows, funding constraints and acquisition of real estate.

SB-7

CONTRACT		FRWLP Reaches	Year for Construction
A		2–5	2020-2021
STAR BEND FIP		6	2019-2020
B		7–12	2019-2020
C	C1	13-18	2017-2018
	C2	19-21	2018-2019

SB-8

CONTRACT		FRWLP Reaches	Year for Construction
A		2–5	2022-2023
STAR BEND FIP		6	2021-2022
B		7–12	2021-2022
C	C1	13-18	2017-2018
	C2	19-25	2018-2019
D	D1	26-33	2019-2020
	D2	34-41	2020-2021

6. COST AND SCHEDULE RISK ANALYSIS (SEE ATTACHED)

The scope of the risk analysis was to calculate and present the cost and schedule 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 contingency derived from the CSRA for Alternatives SB-7 and SB-8 is approximately 35% and 36% respectively.

7. TOTAL PROJECT COSTS (SEE ATTACHED)

REAL ESTATE (01 and 02 Accounts)
The Real Estate cost estimate (01 Account Lands & Damages and Administrative costs) is performed by the SPK Real Estate Division and provided to the Cost Engineering section. The 01 Account Lands and Damages, Relocation Assistance Payment, and New Utility Easements cost estimates were appraised to include 50% incremental costs (please refer to the Real Estate Appendix for more detail). These technical Real Estate increments estimated by the appraiser are independent of the contingency derived though the Cost and Schedule Risk Analysis (CSRA). The contingency for the Federal and Non-Federal Real Estate Administrative costs is estimated at 5% was provided by the Real Estate Division. The CSRA identified no additional contingencies for the 01 Account. The overall contingency for the 01 Account is 33% and 28% for Alternative SB-7 and Alternative SB-8 respectively. For the 02 Account Relocations, the Real Estate Division assessed no contingencies. The CSRA evaluated the relocations and have applied contingencies of 35% and 36% for SB-7 and SB-8 respectively.

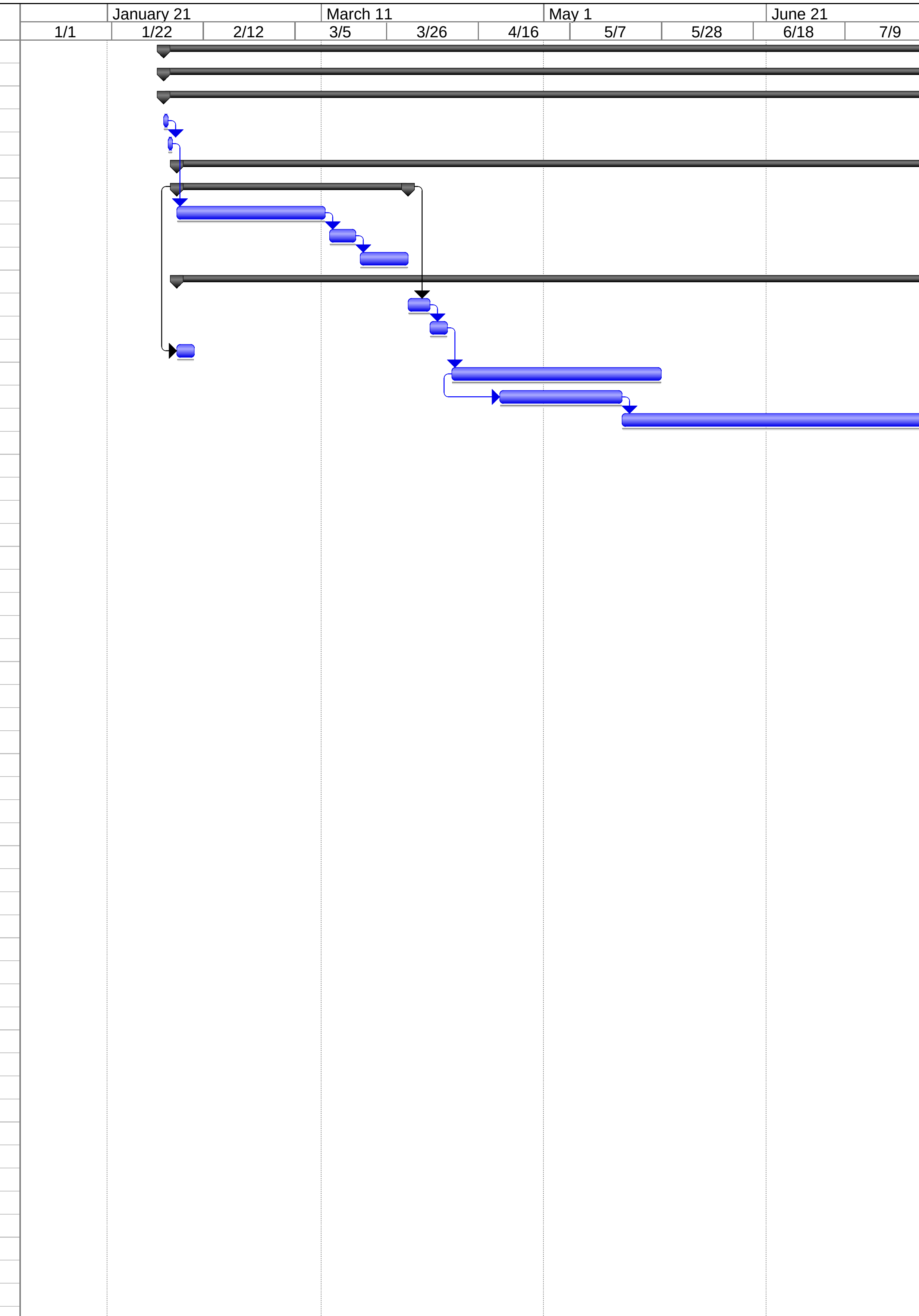
ENVIRONMENTAL MITIGATION (06 Account)
The Environmental Mitigation cost estimate is performed by SPK Environmental Planning and provided to Cost Engineering. It is understood that Environmental Planning included its own contingencies (20%) in the Environmental Mitigation estimate due to their experience and field of expertise. Environmental Mitigation includes costs for Riparian Forest, Oak Woodlands, Elderberry, Giant Garter Snake, Wetlands, Air Quality, and ETL Compliance (please refer to the Environmental Planning Appendix for more detail). Environmental Planning also provided costs for tree removal. Since this is a construction cost the contingency applied to this task will be that derived from the Cost and Schedule Risk Analysis.

CULTURAL RESOURCES PRESERVATION (18 Account)
The Cultural Resources Preservation costs estimate was developed by SPK Archeologist and provided to Cost Engineering. The contingency applied to this account will be that derived from the Cost and Schedule Risk Analysis.

PLANNING, ENGINEERING AND DESIGN (30 Account)
The cost for Planning, Engineering and Design (PED) was provided by the project manager.

CONSTRUCTION MANAGEMENT (31 Account)
The cost for Construction Management (CM) was provided by construction.

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
1		**Sutter Basin SB-7**	**1445 days?**	**Fri 2/3/17**	**Wed 9/15/21**		
2		**Contract C (13-21)**	**802 days?**	**Fri 2/3/17**	**Tue 8/27/19**		
3		**Contract C1 (13-18)**	**539 days?**	**Fri 2/3/17**	**Wed 10/24/18**		
4		Contract Award	1 day	Fri 2/3/17	Fri 2/3/17		
5		NTP	1 day	Sat 2/4/17	Sat 2/4/17	4	
6		**Construction Year 1**	**279 days?**	**Mon 2/6/17**	**Wed 12/27/17**		
7		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/6/17**	**Thu 3/30/17**		
8		Submittals	30 days	Mon 2/6/17	Sat 3/11/17	5	
9		Mobilization	6 days	Mon 3/13/17	Sat 3/18/17	8	
10		Staging Areas Setup	10 days	Mon 3/20/17	Thu 3/30/17	9	
11		**Levees & Floodwalls Reach 13 - 18:**	**279 days?**	**Mon 2/6/17**	**Wed 12/27/17**		
12		Top Soil Stripping	4 days	Fri 3/31/17	Tue 4/4/17	7	
13		Clearing & Grubbing	4 days	Wed 4/5/17	Sat 4/8/17	12	
14		Remove AB Surfacing	4 days	Mon 2/6/17	Thu 2/9/17	7SS	
15		Degrade Exisiting Levees	42 days	Mon 4/10/17	Sat 5/27/17	13	
16		SB Cutoff Wall Conventional	24 days	Fri 4/21/17	Thu 5/18/17	15SS+10 days	
17		SB Cutoff Wall DSM	87 days	Fri 5/19/17	Mon 8/28/17	16	
18		Jet Grouting	18 days	Tue 8/29/17	Mon 9/18/17	17	
19		Levee Embankment Fill	76 days	Sat 9/30/17	Wed 12/27/17	18SS+28 days	
20		Top Soil Replacment	4 days	Sat 12/23/17	Wed 12/27/17	19FF	
21		AB Surfacing Levee Crown	1 day	Wed 12/27/17	Wed 12/27/17	19FF	
22		Relief Well Conveyance Ditch	1 day?	Wed 12/27/17	Wed 12/27/17	19FF	
23		**Construction Year 2**	**191 days**	**Fri 3/16/18**	**Wed 10/24/18**		
24		**Mob, Demob & Preparatory Work**	**16 days**	**Fri 3/16/18**	**Tue 4/3/18**		
25		Mobilization	6 days	Fri 3/16/18	Thu 3/22/18		
26		Staging Areas Setup	10 days	Fri 3/23/18	Tue 4/3/18	25	
27		**Levees & Floodwalls Reach 13 - 18:**	**191 days**	**Fri 3/16/18**	**Wed 10/24/18**		
28		Top Soil Stripping	4 days	Wed 4/4/18	Sat 4/7/18	24	
29		Clearing & Grubbing	4 days	Mon 4/9/18	Thu 4/12/18	28	
30		Remove AB Surfacing	4 days	Fri 3/16/18	Tue 3/20/18	24SS	
31		Degrade Exisiting Levees	42 days	Fri 4/13/18	Thu 5/31/18	29	
32		SB Cutoff Wall Conventional	24 days	Wed 4/25/18	Tue 5/22/18	31SS+10 days	
33		SB Cutoff Wall DSM	87 days	Wed 5/23/18	Fri 8/31/18	32	
34		Jet Grouting	18 days	Sat 9/1/18	Fri 9/21/18	33	
35		Levee Embankment Fill	76 days	Sat 7/28/18	Wed 10/24/18	34FF+28 days	
36		Top Soil Replacment	4 days	Sat 10/20/18	Wed 10/24/18	35FF	
37		AB Surfacing Levee Crown	1 day	Wed 10/24/18	Wed 10/24/18	36FF	
38		**Contract C2 (19-21)**	**490 days**	**Fri 2/2/18**	**Tue 8/27/19**		
39		Contract Award	1 day	Fri 2/2/18	Fri 2/2/18		
40		NTP	1 day	Sat 2/3/18	Sat 2/3/18	39	
41		**Construction Year 1**	**155 days**	**Mon 2/5/18**	**Fri 8/3/18**		
42		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/5/18**	**Thu 3/29/18**		
43		Submittals	30 days	Mon 2/5/18	Sat 3/10/18	40	
44		Mobilization	6 days	Mon 3/12/18	Sat 3/17/18	43	
45		Staging Areas Setup	10 days	Mon 3/19/18	Thu 3/29/18	44	
46		**Levees & Floodwalls Reach 19-21:**	**155 days**	**Mon 2/5/18**	**Fri 8/3/18**		
47		Top Soil Stripping	3 days	Fri 3/30/18	Mon 4/2/18	42	
48		Clearing & Grubbing	2 days	Tue 4/3/18	Wed 4/4/18	47	
49		Remove AB Surfacing	2 days	Mon 2/5/18	Tue 2/6/18	42SS	
50		Degrade Exisiting Levees	16 days	Thu 4/5/18	Mon 4/23/18	48	
51		Excavate Cutoff Trench	2 days	Tue 4/17/18	Wed 4/18/18	50SS+10 days	
52		SB Cutoff Wall Conventional	37 days	Tue 4/17/18	Tue 5/29/18	50SS+10 days	
53		SB Cutoff Wall DSM	34 days	Wed 5/30/18	Sat 7/7/18	52	
54		Levee Embankment Fill	27 days	Mon 7/2/18	Wed 8/1/18	53SS+28 days	
55		AB Surfacing Levee Crown	2 days	Thu 8/2/18	Fri 8/3/18	54	
56		Top Soil Replacment	6 days	Sat 7/28/18	Fri 8/3/18	55FF	



Project: Sutter Basin Rev 1
Date: Thu 3/14/13

Task		Progress		Summary		External Tasks		Deadline
Split		Milestone		Project Summary		External Milestone		

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
57		**Construction Year 2**	**155 days**	**Thu 2/28/19**	**Tue 8/27/19**		
58		**Mob, Demob & Preparatory Work**	**46 days**	**Thu 2/28/19**	**Mon 4/22/19**		
59		Submittals	30 days	Thu 2/28/19	Wed 4/3/19		
60		Mobilization	6 days	Thu 4/4/19	Wed 4/10/19	59	
61		Staging Areas Setup	10 days	Thu 4/11/19	Mon 4/22/19	60	
62		**Levees & Floodwalls Reach 19-21:**	**109 days**	**Tue 4/23/19**	**Tue 8/27/19**		
63		Top Soil Stripping	3 days	Tue 4/23/19	Thu 4/25/19	58	
64		Clearing & Grubbing	2 days	Fri 4/26/19	Sat 4/27/19	63	
65		Remove AB Surfacing	2 days	Tue 4/23/19	Wed 4/24/19	58	
66		Degrade Exisiting Levees	16 days	Mon 4/29/19	Thu 5/16/19	64	
67		Excavate Cutoff Trench	2 days	Fri 5/10/19	Sat 5/11/19	66SS+10 days	
68		SB Cutoff Wall Conventional	37 days	Fri 5/10/19	Fri 6/21/19	66SS+10 days	
69		SB Cutoff Wall DSM	34 days	Sat 6/22/19	Wed 7/31/19	68	
70		Levee Embankment Fill	27 days	Thu 7/25/19	Sat 8/24/19	69SS+28 days	
71		AB Surfacing Levee Crown	2 days	Mon 8/26/19	Tue 8/27/19	70	
72		Top Soil Replacment	6 days	Wed 8/21/19	Tue 8/27/19	71FF	
73		**Contract B (7-12)**	**504 days?**	**Tue 2/5/19**	**Mon 9/14/20**		
74		**Contract B (7-12)**	**504 days?**	**Tue 2/5/19**	**Mon 9/14/20**		
75		Contract Award	1 day	Tue 2/5/19	Tue 2/5/19		
76		NTP	1 day	Wed 2/6/19	Wed 2/6/19	75	
77		**Construction Year 1**	**187 days?**	**Thu 2/7/19**	**Thu 9/12/19**		
78		**Mob, Demob & Preparatory Work**	**46 days**	**Thu 2/7/19**	**Mon 4/1/19**		
79		Submittals	30 days	Thu 2/7/19	Wed 3/13/19	76	
80		Mobilization	6 days	Thu 3/14/19	Wed 3/20/19	79	
81		Staging Areas Setup	10 days	Thu 3/21/19	Mon 4/1/19	80	
82		**Levees & Floodwalls Reach 7-12**	**141 days?**	**Tue 4/2/19**	**Thu 9/12/19**		
83		Top Soil Stripping	6 days	Tue 4/2/19	Mon 4/8/19	78	
84		Clearing & Grubbing	5 days	Tue 4/9/19	Sat 4/13/19	83	
85		Remove AB Surfacing	3 days	Fri 6/7/19	Mon 6/10/19	78,86FF	
86		Degrade Exisiting Levees	49 days	Mon 4/15/19	Mon 6/10/19	84	
87		Excavate Cutoff Trench	4 days	Fri 4/26/19	Tue 4/30/19	86SS+10 days	
88		Excavate Inspection/Key Trench	1 day?	Wed 5/8/19	Wed 5/8/19	87SS+10 days	
89		SB Cutoff Wall Conventional	83 days	Fri 4/26/19	Wed 7/31/19	86SS+10 days	
90		Levee Embankment Fill	83 days	Wed 5/29/19	Mon 9/2/19	89SS+28 days	
91		AB Surfacing Levee Crown	9 days	Tue 9/3/19	Thu 9/12/19	90	
92		Top Soil Replacment	12 days	Fri 8/30/19	Thu 9/12/19	91FF	
93		**Construction Year 2**	**157 days?**	**Mon 3/16/20**	**Mon 9/14/20**		
94		**Mob, Demob & Preparatory Work**	**16 days**	**Mon 3/16/20**	**Thu 4/2/20**		
95		Mobilization	6 days	Mon 3/16/20	Sat 3/21/20		
96		Staging Areas Setup	10 days	Mon 3/23/20	Thu 4/2/20	95	
97		**Levees & Floodwalls Reach 7-12**	**141 days?**	**Fri 4/3/20**	**Mon 9/14/20**		
98		Top Soil Stripping	6 days	Fri 4/3/20	Thu 4/9/20	94	
99		Clearing & Grubbing	5 days	Fri 4/10/20	Wed 4/15/20	98	
100		Remove AB Surfacing	3 days	Tue 6/9/20	Thu 6/11/20	94,101FF	
101		Degrade Exisiting Levees	49 days	Thu 4/16/20	Thu 6/11/20	99	
102		Excavate Cutoff Trench	4 days	Tue 4/28/20	Fri 5/1/20	101SS+10 days	
103		Excavate Inspection/Key Trench	1 day?	Sat 5/9/20	Sat 5/9/20	102SS+10 days	
104		SB Cutoff Wall Conventional	83 days	Tue 4/28/20	Sat 8/1/20	101SS+10 days	
105		Levee Embankment Fill	83 days	Sat 5/30/20	Thu 9/3/20	104SS+28 days	
106		AB Surfacing Levee Crown	9 days	Fri 9/4/20	Mon 9/14/20	105	
107		Top Soil Replacment	12 days	Tue 9/1/20	Mon 9/14/20	106FF	
108		**Star Bend FIP (6)**	**425 days?**	**Tue 2/5/19**	**Sat 6/13/20**		
109		**Star Bend FIP (6)**	**425 days?**	**Tue 2/5/19**	**Sat 6/13/20**		
110		Contract Award	1 day	Tue 2/5/19	Tue 2/5/19		
111		NTP	1 day	Wed 2/6/19	Wed 2/6/19	110	
112		**Construction Year 1**	**108 days**	**Thu 2/7/19**	**Wed 6/12/19**		

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Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
113		**Mob, Demob & Preparatory Work**	**46 days**	**Thu 2/7/19**	**Mon 4/1/19**		
114		Submittals	30 days	Thu 2/7/19	Wed 3/13/19	111	
115		Mobilization	6 days	Thu 3/14/19	Wed 3/20/19	114	
116		Staging Areas Setup	10 days	Thu 3/21/19	Mon 4/1/19	115	
117		**Levees & Floodwalls Reach 6**	**62 days**	**Tue 4/2/19**	**Wed 6/12/19**		
118		Top Soil Stripping	2 days	Tue 4/2/19	Wed 4/3/19	113	
119		Clearing & Grubbing	2 days	Thu 4/4/19	Fri 4/5/19	118	
120		Degrade Exisiting Levees	10 days	Sat 4/6/19	Wed 4/17/19	119	
121		Excavate Cutoff Trench	1 day	Thu 4/18/19	Thu 4/18/19	120SS+10 days	
122		SB Cutoff Wall Conventional	29 days	Thu 4/18/19	Tue 5/21/19	120SS+10 days	
123		Levee Embankment Fill	18 days	Tue 5/21/19	Mon 6/10/19	122SS+28 days	
124		AB Surfacing Levee Crown	2 days	Tue 6/11/19	Wed 6/12/19	123	
125		Top Soil Replacment	3 days	Mon 6/10/19	Wed 6/12/19	124FF	
126		**Construction Year 2**	**78 days?**	**Mon 3/16/20**	**Sat 6/13/20**		
127		**Mob, Demob & Preparatory Work**	**16 days**	**Mon 3/16/20**	**Thu 4/2/20**		
128		Mobilization	6 days	Mon 3/16/20	Sat 3/21/20		
129		Staging Areas Setup	10 days	Mon 3/23/20	Thu 4/2/20	128	
130		**Levees & Floodwalls Reach 6**	**62 days?**	**Fri 4/3/20**	**Sat 6/13/20**		
131		Top Soil Stripping	2 days	Fri 4/3/20	Sat 4/4/20	127	
132		Clearing & Grubbing	2 days	Mon 4/6/20	Tue 4/7/20	131	
133		Degrade Exisiting Levees	10 days	Wed 4/8/20	Sat 4/18/20	132	
134		Excavate Cutoff Trench	1 day	Mon 4/20/20	Mon 4/20/20	133SS+10 days	
135		Excavate Inspection/Key Trench	1 day?	Fri 5/1/20	Fri 5/1/20	134SS+10 days	
136		SB Cutoff Wall Conventional	29 days	Mon 4/20/20	Fri 5/22/20	133SS+10 days	
137		Levee Embankment Fill	18 days	Fri 5/22/20	Thu 6/11/20	136SS+28 days	
138		AB Surfacing Levee Crown	2 days	Fri 6/12/20	Sat 6/13/20	137	
139		Top Soil Replacment	3 days	Thu 6/11/20	Sat 6/13/20	138FF	
140		**Contract A (2-5)**	**506 days**	**Tue 2/4/20**	**Wed 9/15/21**		
141		**Contract A (2-5)**	**506 days**	**Tue 2/4/20**	**Wed 9/15/21**		
142		Contract Award	1 day	Tue 2/4/20	Tue 2/4/20		
143		NTP	1 day	Wed 2/5/20	Wed 2/5/20	142	
144		**Construction Year 1**	**188 days**	**Thu 2/6/20**	**Fri 9/11/20**		
145		**Mob, Demob & Preparatory Work**	**46 days**	**Thu 2/6/20**	**Mon 3/30/20**		
146		Submittals	30 days	Thu 2/6/20	Wed 3/11/20	143	
147		Mobilization	6 days	Thu 3/12/20	Wed 3/18/20	146	
148		Staging Areas Setup	10 days	Thu 3/19/20	Mon 3/30/20	147	
149		**Levees & Floodwalls Reach 2-5:**	**142 days**	**Tue 3/31/20**	**Fri 9/11/20**		
150		Top Soil Stripping	8 days	Tue 3/31/20	Wed 4/8/20	145	
151		Clearing & Grubbing	6 days	Thu 4/9/20	Wed 4/15/20	150	
152		Remove AB Surfacing	3 days	Tue 3/31/20	Thu 4/2/20	145	
153		Degrade Exisiting Levees	46 days	Thu 4/16/20	Mon 6/8/20	151	
154		Excavate Cutoff Trench	4 days	Tue 4/28/20	Fri 5/1/20	153SS+10 days	
155		Excavate Inspection/Key Trench	4 days	Sat 5/9/20	Wed 5/13/20	154SS+10 days	
156		SB Cutoff Wall Conventional	51 days	Tue 4/28/20	Thu 6/25/20	153SS+10 days	
157		SB Cutoff Wall DSM	67 days	Fri 6/26/20	Fri 9/11/20	156	
158		Levee Embankment Fill	82 days	Sat 5/30/20	Wed 9/2/20	156SS+28 days	
159		AB Surfacing Levee Crown	5 days	Thu 9/3/20	Tue 9/8/20	158	
160		Top Soil Replacment	8 days	Mon 8/31/20	Tue 9/8/20	159FF	
161		**Construction Year 2**	**158 days**	**Tue 3/16/21**	**Wed 9/15/21**		
162		**Mob, Demob & Preparatory Work**	**16 days**	**Tue 3/16/21**	**Fri 4/2/21**		
163		Mobilization	6 days	Tue 3/16/21	Mon 3/22/21		
164		Staging Areas Setup	10 days	Tue 3/23/21	Fri 4/2/21	163	
165		**Levees & Floodwalls Reach 2-5**	**142 days**	**Sat 4/3/21**	**Wed 9/15/21**		
166		Top Soil Stripping	8 days	Sat 4/3/21	Mon 4/12/21	162	
167		Clearing & Grubbing	6 days	Tue 4/13/21	Mon 4/19/21	166	
168		Remove AB Surfacing	3 days	Sat 4/3/21	Tue 4/6/21	162	

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Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
169		Degrade Exisiting Levees	46 days	Tue 4/20/21	Fri 6/11/21	167	
170		Excavate Cutoff Trench	4 days	Sat 5/1/21	Wed 5/5/21	169SS+10 days	
171		Excavate Inspection/Key Trench	4 days	Thu 5/13/21	Mon 5/17/21	170SS+10 days	
172		SB Cutoff Wall Conventional	51 days	Sat 5/1/21	Tue 6/29/21	169SS+10 days	
173		SB Cutoff Wall DSM	67 days	Wed 6/30/21	Wed 9/15/21	172	
174		Levee Embankment Fill	82 days	Thu 6/3/21	Mon 9/6/21	172SS+28 days	
175		AB Surfacing Levee Crown	5 days	Tue 9/7/21	Sat 9/11/21	174	
176		Top Soil Replacment	8 days	Fri 9/3/21	Sat 9/11/21	175FF	

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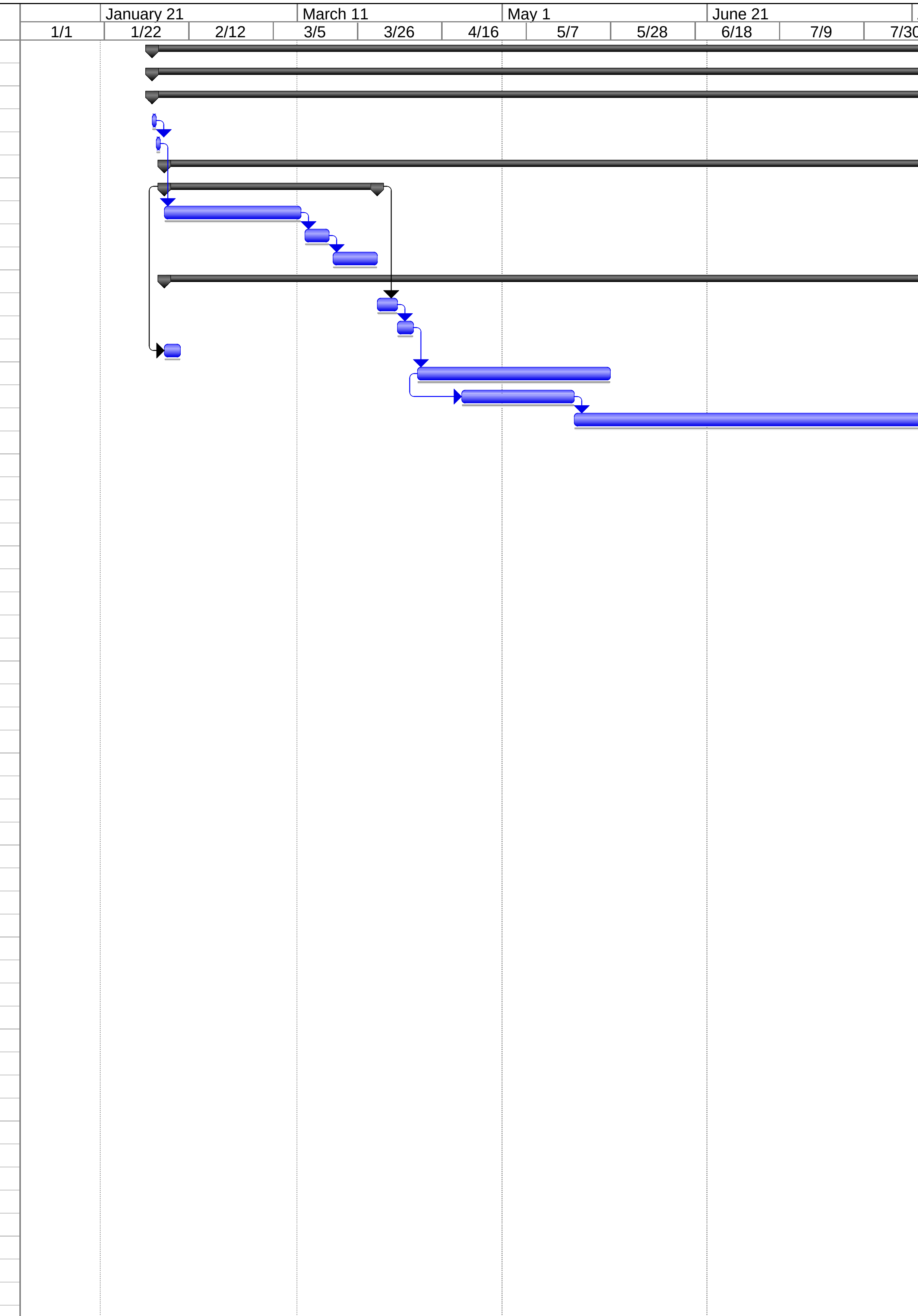
Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
1		**Sutter Basin SB-8**	**2071 days?**	**Fri 2/3/17**	**Fri 9/15/23**		
2		**Contract C (13-25)**	**865 days?**	**Fri 2/3/17**	**Fri 11/8/19**		
3		**Contract C1 (13-18)**	**539 days?**	**Fri 2/3/17**	**Wed 10/24/18**		
4		Contract Award	1 day	Fri 2/3/17	Fri 2/3/17		
5		NTP	1 day	Sat 2/4/17	Sat 2/4/17	4	
6		**Construction Year 1**	**279 days?**	**Mon 2/6/17**	**Wed 12/27/17**		
7		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/6/17**	**Thu 3/30/17**		
8		Submittals	30 days	Mon 2/6/17	Sat 3/11/17	5	
9		Mobilization	6 days	Mon 3/13/17	Sat 3/18/17	8	
10		Staging Areas Setup	10 days	Mon 3/20/17	Thu 3/30/17	9	
11		**Levees & Floodwalls Reach 13 - 18:**	**279 days?**	**Mon 2/6/17**	**Wed 12/27/17**		
12		Top Soil Stripping	4 days	Fri 3/31/17	Tue 4/4/17	7	
13		Clearing & Grubbing	4 days	Wed 4/5/17	Sat 4/8/17	12	
14		Remove AB Surfacing	4 days	Mon 2/6/17	Thu 2/9/17	7SS	
15		Degrade Exisiting Levees	42 days	Mon 4/10/17	Sat 5/27/17	13	
16		SB Cutoff Wall Conventional	24 days	Fri 4/21/17	Thu 5/18/17	15SS+10 days	
17		SB Cutoff Wall DSM	87 days	Fri 5/19/17	Mon 8/28/17	16	
18		Jet Grouting	18 days	Tue 8/29/17	Mon 9/18/17	17	
19		Levee Embankment Fill	76 days	Sat 9/30/17	Wed 12/27/17	18SS+28 days	
20		Top Soil Replacment	4 days	Sat 12/23/17	Wed 12/27/17	19FF	
21		AB Surfacing Levee Crown	1 day	Wed 12/27/17	Wed 12/27/17	19FF	
22		Relief Well Conveyance Ditch	1 day?	Wed 12/27/17	Wed 12/27/17	19FF	
23		**Construction Year 2**	**191 days**	**Fri 3/16/18**	**Wed 10/24/18**		
24		**Mob, Demob & Preparatory Work**	**16 days**	**Fri 3/16/18**	**Tue 4/3/18**		
25		Mobilization	6 days	Fri 3/16/18	Thu 3/22/18		
26		Staging Areas Setup	10 days	Fri 3/23/18	Tue 4/3/18	25	
27		**Levees & Floodwalls Reach 13 - 18:**	**191 days**	**Fri 3/16/18**	**Wed 10/24/18**		
28		Top Soil Stripping	4 days	Wed 4/4/18	Sat 4/7/18	24	
29		Clearing & Grubbing	4 days	Mon 4/9/18	Thu 4/12/18	28	
30		Remove AB Surfacing	4 days	Fri 3/16/18	Tue 3/20/18	24SS	
31		Degrade Exisiting Levees	42 days	Fri 4/13/18	Thu 5/31/18	29	
32		SB Cutoff Wall Conventional	24 days	Wed 4/25/18	Tue 5/22/18	31SS+10 days	
33		SB Cutoff Wall DSM	87 days	Wed 5/23/18	Fri 8/31/18	32	
34		Jet Grouting	18 days	Sat 9/1/18	Fri 9/21/18	33	
35		Levee Embankment Fill	76 days	Sat 7/28/18	Wed 10/24/18	34FF+28 days	
36		Top Soil Replacment	4 days	Sat 10/20/18	Wed 10/24/18	35FF	
37		AB Surfacing Levee Crown	1 day	Wed 10/24/18	Wed 10/24/18	36FF	
38		**Contract C2 (19-25)**	**553 days**	**Fri 2/2/18**	**Fri 11/8/19**		
39		Contract Award	1 day	Fri 2/2/18	Fri 2/2/18		
40		NTP	1 day	Sat 2/3/18	Sat 2/3/18	39	
41		**Construction Year 1**	**218 days**	**Mon 2/5/18**	**Tue 10/16/18**		
42		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/5/18**	**Thu 3/29/18**		
43		Submittals	30 days	Mon 2/5/18	Sat 3/10/18	40	
44		Mobilization	6 days	Mon 3/12/18	Sat 3/17/18	43	
45		Staging Areas Setup	10 days	Mon 3/19/18	Thu 3/29/18	44	
46		**Levees & Floodwalls Reach 19-25:**	**218 days**	**Mon 2/5/18**	**Tue 10/16/18**		
47		Top Soil Stripping	5 days	Fri 3/30/18	Wed 4/4/18	42	
48		Clearing & Grubbing	4 days	Thu 4/5/18	Mon 4/9/18	47	
49		Remove AB Surfacing	4 days	Mon 2/5/18	Thu 2/8/18	42SS	
50		Degrade Exisiting Levees	26 days	Tue 4/10/18	Wed 5/9/18	48	
51		Excavate Cutoff Trench	5 days	Sat 4/21/18	Thu 4/26/18	50SS+10 days	
52		Excavate Inspection/Key Trench	3 days	Thu 5/3/18	Sat 5/5/18	51SS+10 days	
53		SB Cutoff Wall Conventional	67 days	Sat 4/21/18	Sat 7/7/18	50SS+10 days	
54		SB Cutoff Wall DSM	34 days	Mon 7/9/18	Thu 8/16/18	53	
55		Levee Embankment Fill	53 days	Fri 8/10/18	Wed 10/10/18	54SS+28 days	
56		AB Surfacing Levee Crown	5 days	Thu 10/11/18	Tue 10/16/18	55	



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Task		Progress		Summary		External Tasks		Deadline
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ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
57	Top Soil Replacment	10 days	Fri 10/5/18	Tue 10/16/18	56FF	
58	**Construction Year 2**	**218 days**	**Thu 2/28/19**	**Fri 11/8/19**		
59	**Mob, Demob & Preparatory Work**	**46 days**	**Thu 2/28/19**	**Mon 4/22/19**		
60	Submittals	30 days	Thu 2/28/19	Wed 4/3/19		
61	Mobilization	6 days	Thu 4/4/19	Wed 4/10/19	60	
62	Staging Areas Setup	10 days	Thu 4/11/19	Mon 4/22/19	61	
63	**Levees & Floodwalls Reach 19-25:**	**172 days**	**Tue 4/23/19**	**Fri 11/8/19**		
64	Top Soil Stripping	5 days	Tue 4/23/19	Sat 4/27/19	59	
65	Clearing & Grubbing	4 days	Mon 4/29/19	Thu 5/2/19	64	
66	Remove AB Surfacing	4 days	Tue 4/23/19	Fri 4/26/19	59	
67	Degrade Exisiting Levees	26 days	Fri 5/3/19	Sat 6/1/19	65	
68	Excavate Cutoff Trench	5 days	Wed 5/15/19	Mon 5/20/19	67SS+10 days	
69	Excavate Inspection/Key Trench	5 days	Mon 5/27/19	Fri 5/31/19	68SS+10 days	
70	SB Cutoff Wall Conventional	67 days	Wed 5/15/19	Wed 7/31/19	67SS+10 days	
71	SB Cutoff Wall DSM	34 days	Thu 8/1/19	Mon 9/9/19	70	
72	Levee Embankment Fill	53 days	Tue 9/3/19	Sat 11/2/19	71SS+28 days	
73	AB Surfacing Levee Crown	5 days	Mon 11/4/19	Fri 11/8/19	72	
74	Top Soil Replacment	10 days	Tue 10/29/19	Fri 11/8/19	73FF	
75	**Contract D (26-41)**	**793 days**	**Mon 2/4/19**	**Mon 8/16/21**		
76	**Contract D1 (26-33)**	**541 days**	**Mon 2/4/19**	**Mon 10/26/20**		
77	Contract Award	1 day	Mon 2/4/19	Mon 2/4/19		
78	NTP	1 day	Tue 2/5/19	Tue 2/5/19	77	
79	**Construction Year 1**	**228 days**	**Wed 2/6/19**	**Tue 10/29/19**		
80	**Mob, Demob & Preparatory Work**	**46 days**	**Wed 2/6/19**	**Sat 3/30/19**		
81	Submittals	30 days	Wed 2/6/19	Tue 3/12/19	78	
82	Mobilization	6 days	Wed 3/13/19	Tue 3/19/19	81	
83	Staging Areas Setup	10 days	Wed 3/20/19	Sat 3/30/19	82	
84	**Levees & Floodwalls Reach 26-33:**	**228 days**	**Wed 2/6/19**	**Tue 10/29/19**		
85	Top Soil Stripping	4 days	Mon 4/1/19	Thu 4/4/19	80	
86	Clearing & Grubbing	3 days	Fri 4/5/19	Mon 4/8/19	85	
87	Remove AB Surfacing	3 days	Wed 2/6/19	Fri 2/8/19	80SS	
88	Degrade Exisiting Levees	17 days	Tue 4/9/19	Sat 4/27/19	86	
89	Excavate Cutoff Trench	4 days	Sat 4/20/19	Wed 4/24/19	88SS+10 days	
90	Excavate Inspection/Key Trench	5 days	Thu 5/2/19	Tue 5/7/19	89SS+10 days	
91	SB Cutoff Wall Conventional	68 days	Sat 4/20/19	Mon 7/8/19	88SS+10 days	
92	SB Cutoff Wall DSM	60 days	Tue 7/9/19	Mon 9/16/19	91	
93	Jet Grouting	20 days	Sat 8/24/19	Mon 9/16/19	92FF	
94	Levee Embankment Fill	61 days	Fri 8/9/19	Fri 10/18/19	91SS+28 days,93I	
95	AB Surfacing Levee Crown	4 days	Sat 10/19/19	Wed 10/23/19	94	
96	Top Soil Replacment	7 days	Wed 10/16/19	Wed 10/23/19	95FF	
97	**Canal @ STA 1753+00**	**17 days**	**Thu 10/10/19**	**Tue 10/29/19**		
98	Construction New Canal	8 days	Thu 10/10/19	Fri 10/18/19	86,94FF	
99	Backfill Old Canal	9 days	Sat 10/19/19	Tue 10/29/19	98	
100	**Construction Year 2**	**193 days**	**Mon 3/16/20**	**Mon 10/26/20**		
101	**Mob, Demob & Preparatory Work**	**16 days**	**Mon 3/16/20**	**Thu 4/2/20**		
102	Mobilization	6 days	Mon 3/16/20	Sat 3/21/20		
103	Staging Areas Setup	10 days	Mon 3/23/20	Thu 4/2/20	102	
104	**Levees & Floodwalls Reach 26-33**	**193 days**	**Mon 3/16/20**	**Mon 10/26/20**		
105	Top Soil Stripping	4 days	Fri 4/3/20	Tue 4/7/20	101	
106	Clearing & Grubbing	3 days	Wed 4/8/20	Fri 4/10/20	105	
107	Remove AB Surfacing	3 days	Mon 3/16/20	Wed 3/18/20	101SS	
108	Degrade Exisiting Levees	17 days	Sat 4/11/20	Thu 4/30/20	106	
109	Excavate Cutoff Trench	4 days	Thu 4/23/20	Mon 4/27/20	108SS+10 days	
110	Excavate Inspection/Key Trench	5 days	Tue 5/5/20	Sat 5/9/20	109SS+10 days	
111	SB Cutoff Wall Conventional	68 days	Thu 4/23/20	Fri 7/10/20	108SS+10 days	
112	SB Cutoff Wall DSM	60 days	Sat 7/11/20	Fri 9/18/20	111	

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Task | Progress | Summary | External Tasks | Deadline
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ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
113		Jet Grouting	20 days	Thu 8/27/20	Fri 9/18/20	112FF	
114		Levee Embankment Fill	61 days	Wed 8/12/20	Wed 10/21/20	111SS+28 days,1	
115		AB Surfacing Levee Crown	4 days	Thu 10/22/20	Mon 10/26/20	114	
116		Top Soil Replacment	7 days	Mon 10/19/20	Mon 10/26/20	115FF	
117		**Contract D2 (34-41)**	**481 days**	**Mon 2/3/20**	**Mon 8/16/21**		
118		Contract Award	1 day	Mon 2/3/20	Mon 2/3/20		
119		NTP	1 day	Tue 2/4/20	Tue 2/4/20	118	
120		**Construction Year 1**	**163 days**	**Wed 2/5/20**	**Wed 8/12/20**		
121		**Mob, Demob & Preparatory Work**	**46 days**	**Wed 2/5/20**	**Sat 3/28/20**		
122		Submittals	30 days	Wed 2/5/20	Tue 3/10/20	119	
123		Mobilization	6 days	Wed 3/11/20	Tue 3/17/20	122	
124		Staging Areas Setup	10 days	Wed 3/18/20	Sat 3/28/20	123	
125		**Levees & Floodwalls Reach 34-41**	**117 days**	**Mon 3/30/20**	**Wed 8/12/20**		
126		Top Soil Stripping	4 days	Mon 3/30/20	Thu 4/2/20	121	
127		Clearing & Grubbing	5 days	Fri 4/3/20	Wed 4/8/20	126	
128		Remove AB Surfacing	2 days	Sat 5/2/20	Mon 5/4/20	121,129FF	
129		Degrade Exisiting Levees	22 days	Thu 4/9/20	Mon 5/4/20	127	
130		Excavate Cutoff Trench	3 days	Tue 4/21/20	Thu 4/23/20	129SS+10 days	
131		SB Cutoff Wall Conventional	31 days	Tue 4/21/20	Tue 5/26/20	129SS+10 days	
132		SB Cutoff Wall DSM	67 days	Wed 5/27/20	Wed 8/12/20	131	
133		Levee Embankment Fill	23 days	Sat 5/23/20	Thu 6/18/20	131SS+28 days	
134		AB Surfacing Levee Crown	3 days	Fri 6/19/20	Mon 6/22/20	133	
135		Top Soil Replacment	7 days	Mon 6/15/20	Mon 6/22/20	134FF	
136		**Construction Year 2**	**133 days**	**Mon 3/15/21**	**Mon 8/16/21**		
137		**Mob, Demob & Preparatory Work**	**16 days**	**Mon 3/15/21**	**Thu 4/1/21**		
138		Mobilization	6 days	Mon 3/15/21	Sat 3/20/21		
139		Staging Areas Setup	10 days	Mon 3/22/21	Thu 4/1/21	138	
140		**Levees & Floodwalls Reach 34-41**	**117 days**	**Fri 4/2/21**	**Mon 8/16/21**		
141		Top Soil Stripping	4 days	Fri 4/2/21	Tue 4/6/21	137	
142		Clearing & Grubbing	5 days	Wed 4/7/21	Mon 4/12/21	141	
143		Remove AB Surfacing	2 days	Thu 5/6/21	Fri 5/7/21	137,144FF	
144		Degrade Exisiting Levees	22 days	Tue 4/13/21	Fri 5/7/21	142	
145		Excavate Cutoff Trench	3 days	Sat 4/24/21	Tue 4/27/21	144SS+10 days	
146		SB Cutoff Wall Conventional	31 days	Sat 4/24/21	Sat 5/29/21	144SS+10 days	
147		SB Cutoff Wall DSM	67 days	Mon 5/31/21	Mon 8/16/21	146	
148		Levee Embankment Fill	23 days	Thu 5/27/21	Tue 6/22/21	146SS+28 days	
149		AB Surfacing Levee Crown	3 days	Wed 6/23/21	Fri 6/25/21	148	
150		Top Soil Replacment	7 days	Fri 6/18/21	Fri 6/25/21	149FF	
151		**Contract B (7-12)**	**503 days?**	**Fri 2/5/21**	**Wed 9/14/22**		
152		**Contract B (7-12)**	**503 days?**	**Fri 2/5/21**	**Wed 9/14/22**		
153		Contract Award	1 day	Fri 2/5/21	Fri 2/5/21		
154		NTP	1 day	Sat 2/6/21	Sat 2/6/21	153	
155		**Construction Year 1**	**187 days?**	**Mon 2/8/21**	**Mon 9/13/21**		
156		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/8/21**	**Thu 4/1/21**		
157		Submittals	30 days	Mon 2/8/21	Sat 3/13/21	154	
158		Mobilization	6 days	Mon 3/15/21	Sat 3/20/21	157	
159		Staging Areas Setup	10 days	Mon 3/22/21	Thu 4/1/21	158	
160		**Levees & Floodwalls Reach 7-12**	**141 days?**	**Fri 4/2/21**	**Mon 9/13/21**		
161		Top Soil Stripping	6 days	Fri 4/2/21	Thu 4/8/21	156	
162		Clearing & Grubbing	5 days	Fri 4/9/21	Wed 4/14/21	161	
163		Remove AB Surfacing	3 days	Tue 6/8/21	Thu 6/10/21	156,164FF	
164		Degrade Exisiting Levees	49 days	Thu 4/15/21	Thu 6/10/21	162	
165		Excavate Cutoff Trench	4 days	Tue 4/27/21	Fri 4/30/21	164SS+10 days	
166		Excavate Inspection/Key Trench	1 day?	Sat 5/8/21	Sat 5/8/21	165SS+10 days	
167		SB Cutoff Wall Conventional	83 days	Tue 4/27/21	Sat 7/31/21	164SS+10 days	
168		Levee Embankment Fill	83 days	Sat 5/29/21	Thu 9/2/21	167SS+28 days	

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Project: Sutter Basin Rev 1
Date: Thu 3/14/13

Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
169		AB Surfacing Levee Crown	9 days	Fri 9/3/21	Mon 9/13/21	168	
170		Top Soil Replacment	12 days	Tue 8/31/21	Mon 9/13/21	169FF	
171		**Construction Year 2**	**157 days?**	**Wed 3/16/22**	**Wed 9/14/22**		
172		**Mob, Demob & Preparatory Work**	**16 days**	**Wed 3/16/22**	**Sat 4/2/22**		
173		Mobilization	6 days	Wed 3/16/22	Tue 3/22/22		
174		Staging Areas Setup	10 days	Wed 3/23/22	Sat 4/2/22	173	
175		**Levees & Floodwalls Reach 7-12**	**141 days?**	**Mon 4/4/22**	**Wed 9/14/22**		
176		Top Soil Stripping	6 days	Mon 4/4/22	Sat 4/9/22	172	
177		Clearing & Grubbing	5 days	Mon 4/11/22	Fri 4/15/22	176	
178		Remove AB Surfacing	3 days	Thu 6/9/22	Sat 6/11/22	172,179FF	
179		Degrade Exisiting Levees	49 days	Sat 4/16/22	Sat 6/11/22	177	
180		Excavate Cutoff Trench	4 days	Thu 4/28/22	Mon 5/2/22	179SS+10 days	
181		Excavate Inspection/Key Trench	1 day?	Tue 5/10/22	Tue 5/10/22	180SS+10 days	
182		SB Cutoff Wall Conventional	83 days	Thu 4/28/22	Tue 8/2/22	179SS+10 days	
183		Levee Embankment Fill	83 days	Tue 5/31/22	Sat 9/3/22	182SS+28 days	
184		AB Surfacing Levee Crown	9 days	Mon 9/5/22	Wed 9/14/22	183	
185		Top Soil Replacment	12 days	Thu 9/1/22	Wed 9/14/22	184FF	
186		**Star Bend FIP (6)**	**424 days?**	**Fri 2/5/21**	**Tue 6/14/22**		
187		**Star Bend FIP (6)**	**424 days?**	**Fri 2/5/21**	**Tue 6/14/22**		
188		Contract Award	1 day	Fri 2/5/21	Fri 2/5/21		
189		NTP	1 day	Sat 2/6/21	Sat 2/6/21	188	
190		**Construction Year 1**	**108 days**	**Mon 2/8/21**	**Sat 6/12/21**		
191		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/8/21**	**Thu 4/1/21**		
192		Submittals	30 days	Mon 2/8/21	Sat 3/13/21	189	
193		Mobilization	6 days	Mon 3/15/21	Sat 3/20/21	192	
194		Staging Areas Setup	10 days	Mon 3/22/21	Thu 4/1/21	193	
195		**Levees & Floodwalls Reach 6**	**62 days**	**Fri 4/2/21**	**Sat 6/12/21**		
196		Top Soil Stripping	2 days	Fri 4/2/21	Sat 4/3/21	191	
197		Clearing & Grubbing	2 days	Mon 4/5/21	Tue 4/6/21	196	
198		Degrade Exisiting Levees	10 days	Wed 4/7/21	Sat 4/17/21	197	
199		Excavate Cutoff Trench	1 day	Mon 4/19/21	Mon 4/19/21	198SS+10 days	
200		SB Cutoff Wall Conventional	29 days	Mon 4/19/21	Fri 5/21/21	198SS+10 days	
201		Levee Embankment Fill	18 days	Fri 5/21/21	Thu 6/10/21	200SS+28 days	
202		AB Surfacing Levee Crown	2 days	Fri 6/11/21	Sat 6/12/21	201	
203		Top Soil Replacment	3 days	Thu 6/10/21	Sat 6/12/21	202FF	
204		**Construction Year 2**	**78 days?**	**Wed 3/16/22**	**Tue 6/14/22**		
205		**Mob, Demob & Preparatory Work**	**16 days**	**Wed 3/16/22**	**Sat 4/2/22**		
206		Mobilization	6 days	Wed 3/16/22	Tue 3/22/22		
207		Staging Areas Setup	10 days	Wed 3/23/22	Sat 4/2/22	206	
208		**Levees & Floodwalls Reach 6**	**62 days?**	**Mon 4/4/22**	**Tue 6/14/22**		
209		Top Soil Stripping	2 days	Mon 4/4/22	Tue 4/5/22	205	
210		Clearing & Grubbing	2 days	Wed 4/6/22	Thu 4/7/22	209	
211		Degrade Exisiting Levees	10 days	Fri 4/8/22	Tue 4/19/22	210	
212		Excavate Cutoff Trench	1 day	Wed 4/20/22	Wed 4/20/22	211SS+10 days	
213		Excavate Inspection/Key Trench	1 day?	Mon 5/2/22	Mon 5/2/22	212SS+10 days	
214		SB Cutoff Wall Conventional	29 days	Wed 4/20/22	Mon 5/23/22	211SS+10 days	
215		Levee Embankment Fill	18 days	Mon 5/23/22	Sat 6/11/22	214SS+28 days	
216		AB Surfacing Levee Crown	2 days	Mon 6/13/22	Tue 6/14/22	215	
217		Top Soil Replacment	3 days	Sat 6/11/22	Tue 6/14/22	216FF	
218		**Contract A (2-5)**	**505 days**	**Fri 2/4/22**	**Fri 9/15/23**		
219		**Contract A (2-5)**	**505 days**	**Fri 2/4/22**	**Fri 9/15/23**		
220		Contract Award	1 day	Fri 2/4/22	Fri 2/4/22		
221		NTP	1 day	Sat 2/5/22	Sat 2/5/22	220	
222		**Construction Year 1**	**188 days**	**Mon 2/7/22**	**Tue 9/13/22**		
223		**Mob, Demob & Preparatory Work**	**46 days**	**Mon 2/7/22**	**Thu 3/31/22**		
224		Submittals	30 days	Mon 2/7/22	Sat 3/12/22	221	

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Task | Progress | Summary | External Tasks | Deadline

Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
225		Mobilization	6 days	Mon 3/14/22	Sat 3/19/22	224	
226		Staging Areas Setup	10 days	Mon 3/21/22	Thu 3/31/22	225	
227		**Levees & Floodwalls Reach 2-5:**	**142 days**	**Fri 4/1/22**	**Tue 9/13/22**		
228		Top Soil Stripping	8 days	Fri 4/1/22	Sat 4/9/22	223	
229		Clearing & Grubbing	6 days	Mon 4/11/22	Sat 4/16/22	228	
230		Remove AB Surfacing	3 days	Fri 4/1/22	Mon 4/4/22	223	
231		Degrade Exisiting Levees	46 days	Mon 4/18/22	Thu 6/9/22	229	
232		Excavate Cutoff Trench	4 days	Fri 4/29/22	Tue 5/3/22	231SS+10 days	
233		Excavate Inspection/Key Trench	4 days	Wed 5/11/22	Sat 5/14/22	232SS+10 days	
234		SB Cutoff Wall Conventional	51 days	Fri 4/29/22	Mon 6/27/22	231SS+10 days	
235		SB Cutoff Wall DSM	67 days	Tue 6/28/22	Tue 9/13/22	234	
236		Levee Embankment Fill	82 days	Wed 6/1/22	Sat 9/3/22	234SS+28 days	
237		AB Surfacing Levee Crown	5 days	Mon 9/5/22	Fri 9/9/22	236	
238		Top Soil Replacment	8 days	Thu 9/1/22	Fri 9/9/22	237FF	
239		**Construction Year 2**	**158 days**	**Thu 3/16/23**	**Fri 9/15/23**		
240		**Mob, Demob & Preparatory Work**	**16 days**	**Thu 3/16/23**	**Mon 4/3/23**		
241		Mobilization	6 days	Thu 3/16/23	Wed 3/22/23		
242		Staging Areas Setup	10 days	Thu 3/23/23	Mon 4/3/23	241	
243		**Levees & Floodwalls Reach 2-5**	**142 days**	**Tue 4/4/23**	**Fri 9/15/23**		
244		Top Soil Stripping	8 days	Tue 4/4/23	Wed 4/12/23	240	
245		Clearing & Grubbing	6 days	Thu 4/13/23	Wed 4/19/23	244	
246		Remove AB Surfacing	3 days	Tue 4/4/23	Thu 4/6/23	240	
247		Degrade Exisiting Levees	46 days	Thu 4/20/23	Mon 6/12/23	245	
248		Excavate Cutoff Trench	4 days	Tue 5/2/23	Fri 5/5/23	247SS+10 days	
249		Excavate Inspection/Key Trench	4 days	Sat 5/13/23	Wed 5/17/23	248SS+10 days	
250		SB Cutoff Wall Conventional	51 days	Tue 5/2/23	Thu 6/29/23	247SS+10 days	
251		SB Cutoff Wall DSM	67 days	Fri 6/30/23	Fri 9/15/23	250	
252		Levee Embankment Fill	82 days	Sat 6/3/23	Wed 9/6/23	250SS+28 days	
253		AB Surfacing Levee Crown	5 days	Thu 9/7/23	Tue 9/12/23	252	
254		Top Soil Replacment	8 days	Mon 9/4/23	Tue 9/12/23	253FF	

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Project: Sutter Basin Rev 1
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Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

US Army Corps
of Engineers®

Sutter Basin Feasibility Study
Alternative Selection Plans
Project Cost and Schedule Risk Analysis Report

Prepared for:

U.S. Army Corps of Engineers,
Sacramento District

Prepared with:

U.S. Army Corps of Engineers
Cost Engineering Mandatory Center of Expertise, Walla Walla

August 1, 2013

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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 Sutter Basin Feasibility Study for two Alternatives (SB7 and SB8). In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a formal risk analysis study was conducted for the development of contingency on the total project cost. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommend 80% confidence level of successful execution to project completion.

The Sutter Basin Study consists of levee remediations necessary to reduce flood risk to the Sutter Basin. The vast majority of levee remediation consists of seepage prevention by constructing a soil-bentonite slurry cutoff wall through the centerline of the levee and rebuild the levee to pre-project geometry.

The Sutter Basin Feasibility Study considers three (3) Alternatives; Do Nothing; SB7, a Fix-in-Place alternative running for the Feather River West Levee from Sunset Weir to Laurel Avenue; and SB8, a Fix-in-Place alternative for the Feather River West Levee running from Thermalito Afterbay to Laurel Avenue (essentially SB7 plus the additional length from Thermalito Afterbay to Sunset Weir).

Specific to the Sutter Basin project, the base case construction cost for

- SB7 (excluding Accounts 01 Lands and Damages, 02 Fish and Wildlife Facilities, 30 Planning, Engineering and Design and 31 Construction Management) is estimated at approximately $194 Million. Based on the results of the analysis, the Cost Engineering Mandatory Center of Expertise for Civil Works (Walla Walla District) recommends a contingency value of approximately $70.5 Million, or 36%.

- SB8 (excluding Accounts 01 Lands and Damages, 02 Fish and Wildlife Facilities, 30 Planning, Engineering and Design and 31 Construction Management) is estimated at approximately $364 Million. Based on the results of the analysis, the Cost Engineering Mandatory Center of Expertise for Civil Works (Walla Walla District) recommends a contingency value of approximately $126.4 Million, or 35%.

In conjunction with the Sacramento team, the Cost Engineering Mandatory Center of Expertise (MCX) for Civil Works performed risk analysis by applying the *Monte Carlo* technique, producing the aforementioned contingencies and identifying key risk drivers.

The following tables ES-1 and ES-2portray the developed contingencies for both alternatives and resulting approximate project costs. The recommended contingencies are based on an 80% confidence level, as per USACE Civil Works guidance. The following tables are not an exact replica of the final reported Total Project costs due to rounding and late cost adjustments. The calculated contingencies are approximate and reflective of those items and cost studied. The following cost accounts were excluded for the risk study:

- The 01-Lands and Damages and the 06-Fish and Wildlife contingencies were established outside of the risk model.

- The 30-Preconstruction, Engineering and Design and the 31-Construction Management carry the same % of contingency value as construction; the theory being is that as constructions cost are impacted, so are these two respective accounts.

Table ES-1A. Contingency Analysis Table – Alternative SB7

Base Cost Estimate	**$194,048,000**	
Confidence Level	**Value ($$)**	**Contingency (%)**
5%	$33,495,693	17.26%
50%	$56,363,817	29.05%
80%	**$70,533,025**	**36.35%**
95%	$83,658,086	43.11%

Table ES-1B. Contingency Analysis Table – Alternative SB8

Base Cost Estimate	**$363,638,000**	
Confidence Level	**Value ($$)**	**Contingency (%)**
5%	$59,186,930	16.28%
50%	$100,985,958	27.77%
80%	**$126,390,500**	**34.76%**
95%	$149,857,593	41.21%

The risk analysis and resulting contingencies are presented as both a cost in dollars and a per cent of the base costs. The risk analysis was performed on a specific cost at a specific point in time. Subtle changes in the costs used to support the risk analysis do not have a significant bearing on contingency dollars or per cent when risk remains constant. As costs fluctuate to a slight degree and risks remain constant, greater emphasis is placed on the per cent value.

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The key cost risk drivers identified through sensitivity analysis for both Alternatives SB7 and SB8 are CA-3 (Availability of Qualified Contractors) and CA-1 (Multiple Construction Contracts), which together contribute some 69 percent of the statistical cost variance.

- Availability of Qualified Contractors. Captures the risk of limited competition. Multiple other contracts with similar seepage cutoff wall construction could be ongoing at time of contract award, potentially limiting the pool of contractors available to perform the work, impacting the ultimate contract costs.
- Multiple Construction Contracts. Captures the risk funding constraints will require multiple construction contracts, resulting in construction inefficiencies (multiple mob/demobs) and increasing contract oversight and administration costs.

Moderate cost risks, when combined, can also become a cost impact. The greater moderate risks include:

- Availability of suitable Borrow Sources.
- Potential Future Construction Claims and Modifications
- Potential System Wide Improvement Framework (SWIF) Requirements
- Funding Delays

The key schedule risk drivers identified through sensitivity analysis both Alternatives SB7 and SB8 are CA-1 (Multiple Construction Contracts) and PPM-2 (Vertical Team Review and Approval), which together contribute some 72 percent of the statistical schedule variance.

- Multiple Construction Contracts captures the risk funding constraints will require multiple non-concurrent construction contracts, extending the time required to complete the total project.
- Vertical Team Review and Approval captures the risk high time demands on vertical teams have created a backlog of projects and resulting in the potential for delays in the approval process and subsequent schedule slips.
- Funding Delays captures the possible delays in availability in Federal funds and the resulting issues that a protracted construction schedule can place the project at greater risks related to more stringent environmental restrictions, scope changes, political changes, escalation exceeding OMB projections, greater potential for extreme commodity availability and inflation.

Moderate schedule risks, when combined, can also become a time and resulting cost impact. The greater moderate risks include:

- Construction Windows for Endangered Species
- Potential Unknown HTRW Sites
- Potential Cultural Discoveries
- Project Competing with Other Priorities (Staffing)
- Potential Future Construction Claims and Modifications

Recommendations, as detailed within the main report, include the implementation of cost and schedule contingencies, further iterative study of risks throughout the project life-cycle, potential mitigation throughout the PED phase, and proactive monitoring and control of risk identified in this study.

MAIN REPORT

1.0 PURPOSE

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 Sutter Basin Feasibility Study for two Alternatives (SB7 and SB8).

2.0 BACKGROUND

The Sutter Basin Study consists of levee remediation necessary to reduce flood risk to the Sutter Basin. The vast majority of levee remediation consists of seepage prevention by constructing a soil-bentonite slurry cutoff wall through the centerline of the levee and rebuild the levee to pre-project geometry. At some locations, seepage berm, relief wells, deep-soil-mixing, jet grout cutoff wall, canal relocation, and slight levee relocation to provide O&M access roads are included but they are minor relative to the soil-bentonite cutoff wall construction.

The Sutter Basin Feasibility Study considers three (3) Alternatives; Do Nothing; SB7, a Fix-in-Place alternative running for the Feather River West Levee from Sunset Weir to Laurel Avenue; and SB8, a Fix-in-Place alternative for the Feather River West Levee running from Thermalito Afterbay to Laurel Avenue (essentially SB7 plus the additional length from Thermalito Afterbay to Sunset Weir).

The primary project sponsors are the Sutter Butte Flood Control Agency (SBFCA). and the California Department of Water Resources (DWR).The work will likely be complete in 5-7 phases due to funding increment limitations. It is likely that the contracts will be acquired using a RFP procurement. The current construction schedule is approximately 24 months in duration. Construction of the first phase (Star Bend) has been started by the Sponsor with additional phases to begin construction in late FY 3013.

As a part of study effort, Sacramento District has requested that the USACE Cost Engineering Mandatory Center of Expertise for Civil Works (Cost Engineering MCX) provide a risk analysis study to establish the resulting contingencies.

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. This report is intended to serve as part of the risk management plan. The CSRA applies the principles 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 study and presentation 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 base case Micro Computer Aided Cost Estimating System (MCACES) cost estimate, 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 problems, 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 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.

The Walla Walla Cost Engineering MCX performed the Cost and Schedule Risk Analysis, relying on local Sacramento District staff to provide information gathering. The Walla Walla Cost Engineering MCX facilitated an on-site risk identification meeting on January 24, 2013 with the Sacramento District PDT to produce a risk register that served as the framework for the risk analysis. Participants in risk identification meeting included the following:

Name	Organization	Title
Peter Blodgett	USACE - SPK	Hydraulic Engineer
William Bolte	USACE - NWW	Cost Engineer (Risk Facilitator)
Jane Bolton	USACE - SPK	Geotechnical Engineer
Matt Davis	USACE - SPK	Environmental Engineer
Tri Duong	USACE - SPK	Cost Engineer
Mark Ellis	USACE - SPK	Project Manager
Miki Fujitsubo	USACE - SPK	Planner
Erik Gomez	USACE - SPK	Economist
S. Joe Griffin	USACE - SPK	Cultural Resources
Richard Kristof	USACE - SPK	Civil Engineer
Tung Le	USACE - SPK	Structural Engineer
Michael Musto	DWR	Sponsor Representative
Laurie Parker	USACE – SPK	Real Estate
David Peterson	PBI	Sponsor Representative

Representatives from Construction and Contracting were contacted after the on-site risk identification meeting and given the initial Risk Registry for their review. Their subsequent input has been incorporated into the final Risk Registry.

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. Subsequent meetings focused primarily on risk factor assessment and quantification.

Additionally, numerous 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 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 quantifying the costs associated with the Sutter Basin project.

a. The Sacramento District provided MII MCACES (Micro-Computer Aided Cost Estimating Software) files and a summary Excel spreadsheet detailing all project costs by contract and serves as the basis for the final cost and schedule risk analyses.

b. The cost comparisons and risk analyses performed and reflected within this report are based on design scope and estimates that are at the feasibility level.

c. The CSRA excludes

- The 01-Lands and Damages and the 06-Fish and Wildlife contingencies were established outside of the risk model.

- The 30-Preconstruction, Engineering and Design and the 31-Construction Management carry the same % of contingency value as construction; the theory being is that as constructions cost are impacted, so are these two respective accounts.

d. Schedules are analyzed for impact to the project cost in terms of both uncaptured escalation (variance from OMB factors and the local market) and unavoidable fixed contract costs and/or languishing federal administration costs incurred throughout delay.

Specific to the Sutter Basin project, the schedule was analyzed only for impacts due to residual fixed costs.

e. The risk analyses accounted for escalation over and above the projected Office of Management and Budget (OMB). Based on a detailed calculations for the Isabella Lake Project, Sacramento District has calculated California is 1.92% higher than the OMB rates.

f. Per the data in the estimate, the Overhead percentage for the Prime Contractor is 10%, and 10% for the Subcontractors. Thus, the assumed residual fixed cost rate for this project is 10%. For the P80 schedule, this comprises approximately 22% of the total contingency and 8% of the base cost estimate (9.2% for SB7 and 7.7% for SB8). This is due to the accrual of residual fixed costs associated with delay associated with the implementation schedule.

g. The Cost 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.

h. 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 construction cost contingencies for the P50 and P95 confidence levels are also provided for illustrative purposes only.

Table 1A. Construction Cost Contingency Summary – SB7

Risk Analysis Forecast	Total Construction Cost	Total Contingency[1] ($)	Total Contingency (%)
50% Confidence Level			
Construction Cost	$250,411,817	$56,363,817	29.05%
80% Confidence Level			
Construction Cost	$264,581,025	$70,533,025	36.35%
95% Confidence Level			
Construction Cost	$277,706,086	$83,658,086	43.11%

Notes:
1) These figures combine uncertainty in the baseline cost estimates and schedule.

Table 1B. Construction Cost Contingency Summary – SB8

Risk Analysis Forecast	Total Construction Cost	Total Contingency[1] ($)	Total Contingency (%)
50% Confidence Level			
Construction Cost	$464,623,958	$100,985,958	27.77%
80% Confidence Level			
Construction Cost	$490,028,500	$126,390,500	34.76%
95% Confidence Level			
Construction Cost	$513,495,593	$149,857,593	41.21%

Notes:
1) These figures combine uncertainty in the baseline cost estimates and schedule.

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 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 1A. Cost Sensitivity Analysis – SB7

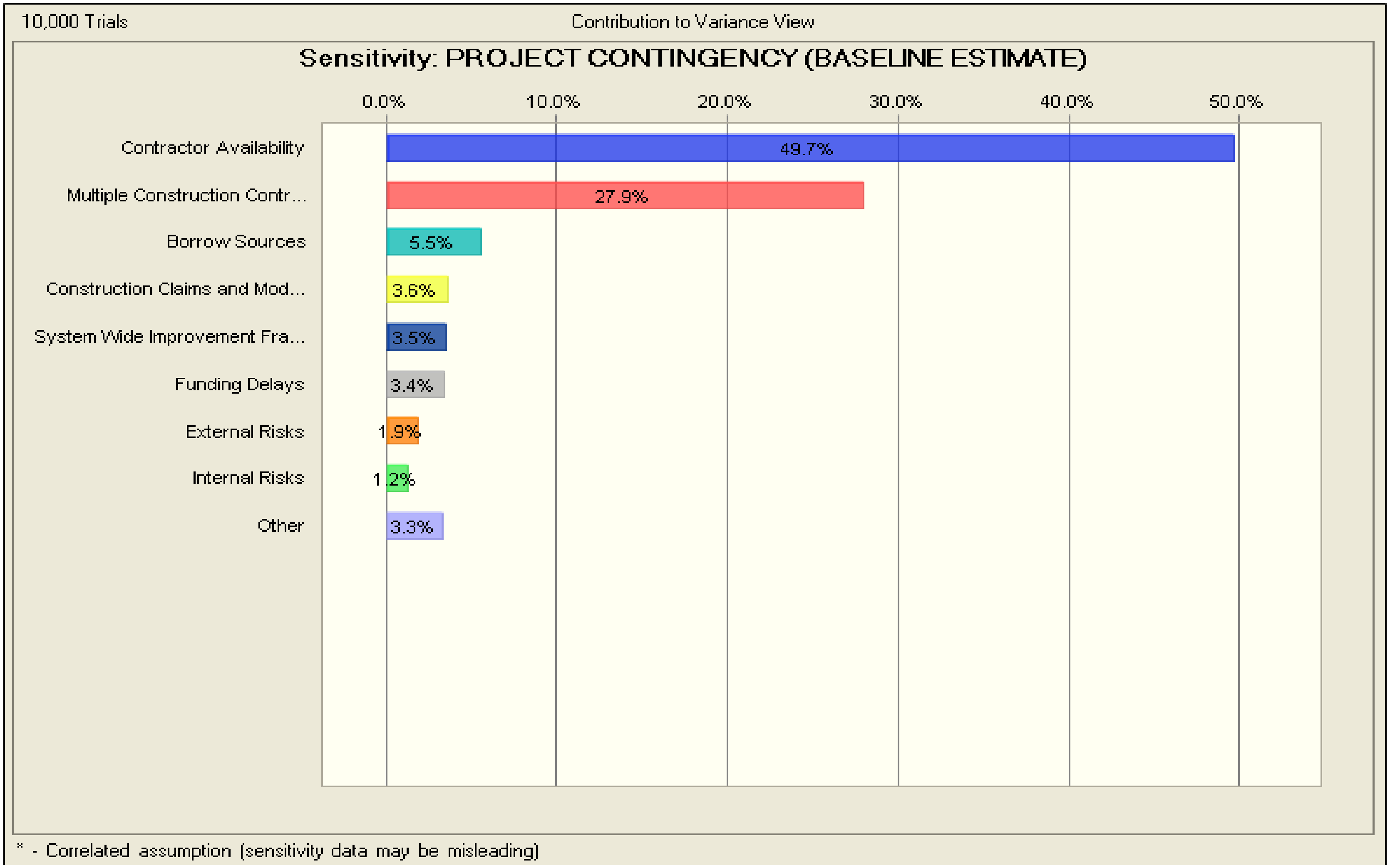
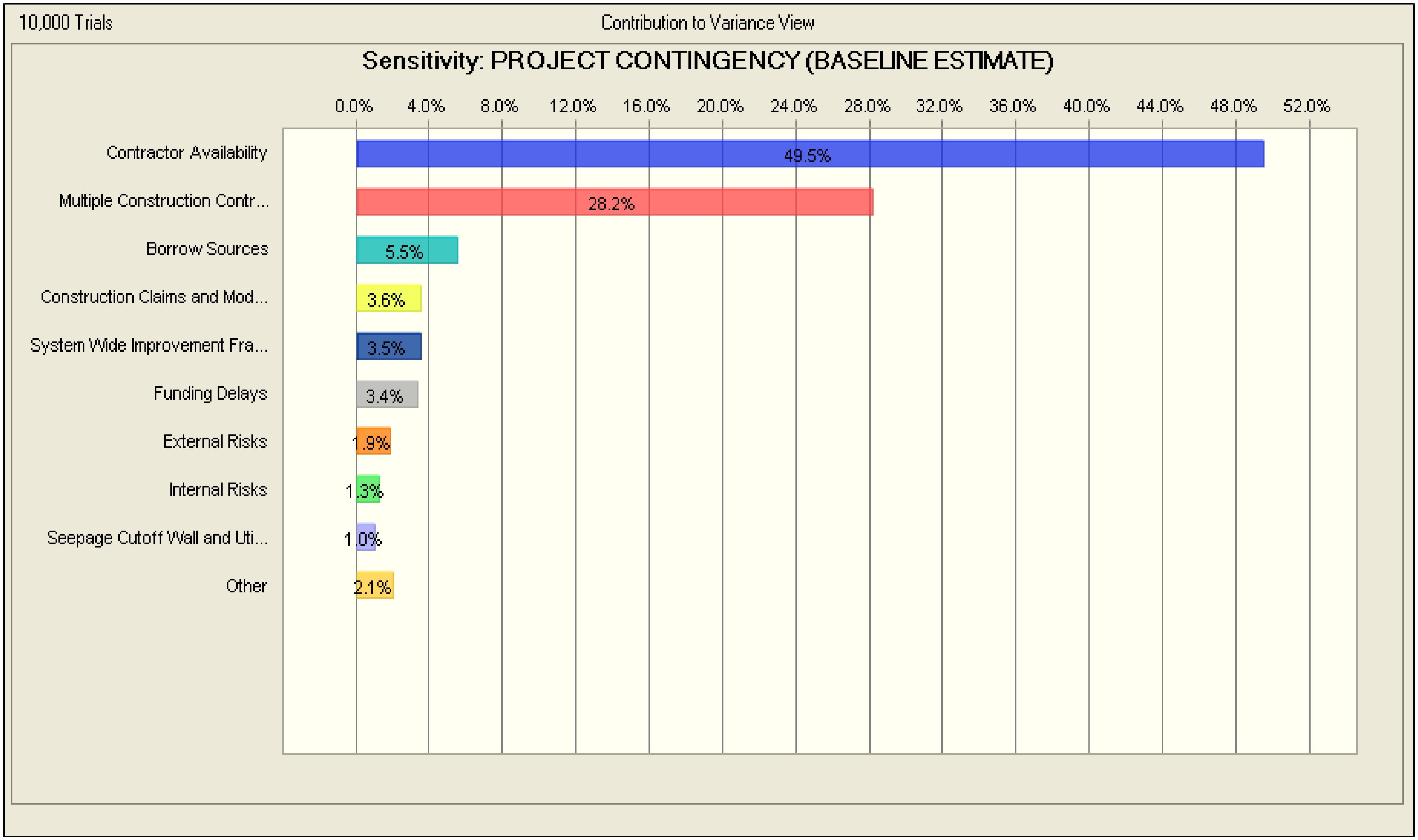


Figure 1B. Cost Sensitivity Analysis – SB8



6.3 Schedule Contingency Risk Analysis

Table 2 provides the schedule duration contingencies calculated for the P80 confidence level. The schedule duration contingencies for the P50 and P95 confidence levels are also provided for illustrative purposes.

Schedule duration contingency was quantified as 37 months for SB7 and 41 months for SB8 based on the P80 level of confidence. These contingencies were used to calculate the projected residual fixed cost impact of project delays that are included in the Table 1 presentation of total cost contingency. The schedule contingencies were calculated by applying the high level schedule risks identified in the risk register for each option to the durations of critical path and near critical path tasks.

The schedule was not resource loaded and contained open-ended tasks and non-zero lags (gaps in the logic between tasks) that limit the overall utility of the schedule risk analysis. These issues should be considered as limitations in the utility of the schedule contingency data presented. Schedule contingency impacts presented in this analysis are based solely on projected residual fixed costs.

Table 2A. Schedule Duration Contingency Summary – SB7

Risk Analysis Forecast	Baseline Schedule Duration (months)	Contingency[1] (months)
50% Confidence Level		
Project Duration	60	28
80% Confidence Level		
Project Duration	60	37
95% Confidence Level		
Project Duration	60	45

Notes:
1) The schedule was not resource loaded and contained open-ended tasks and non-zero lags (gaps in the logic between tasks) that limit the overall utility of the schedule risk analysis. These issues should be considered as limitations in the utility of the schedule contingency data presented in Table 2.

Table 2A. Schedule Duration Contingency Summary – SB8

Risk Analysis Forecast	Baseline Schedule Duration (months)	Contingency[1] (months)
50% Confidence Level		
Project Duration	84	31
80% Confidence Level		
Project Duration	84	41
95% Confidence Level		
Project Duration	84	50

Notes:
1) The schedule was not resource loaded and contained open-ended tasks and non-zero lags (gaps in the logic between tasks) that limit the overall utility of the schedule risk analysis. These issues should be considered as limitations in the utility of the schedule contingency data presented in Table 2.

Figure 2A. Schedule Sensitivity Analysis – SB7

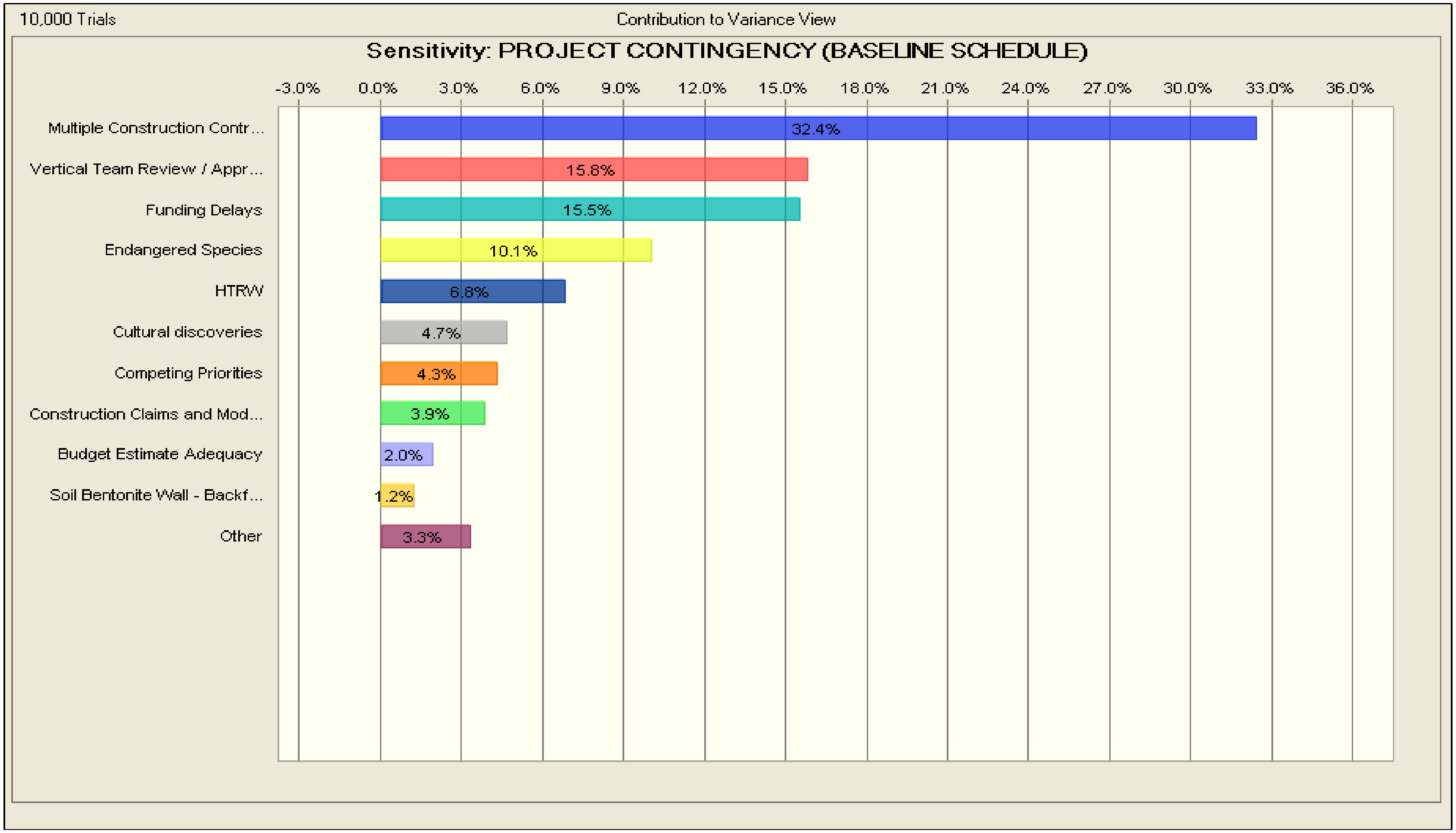
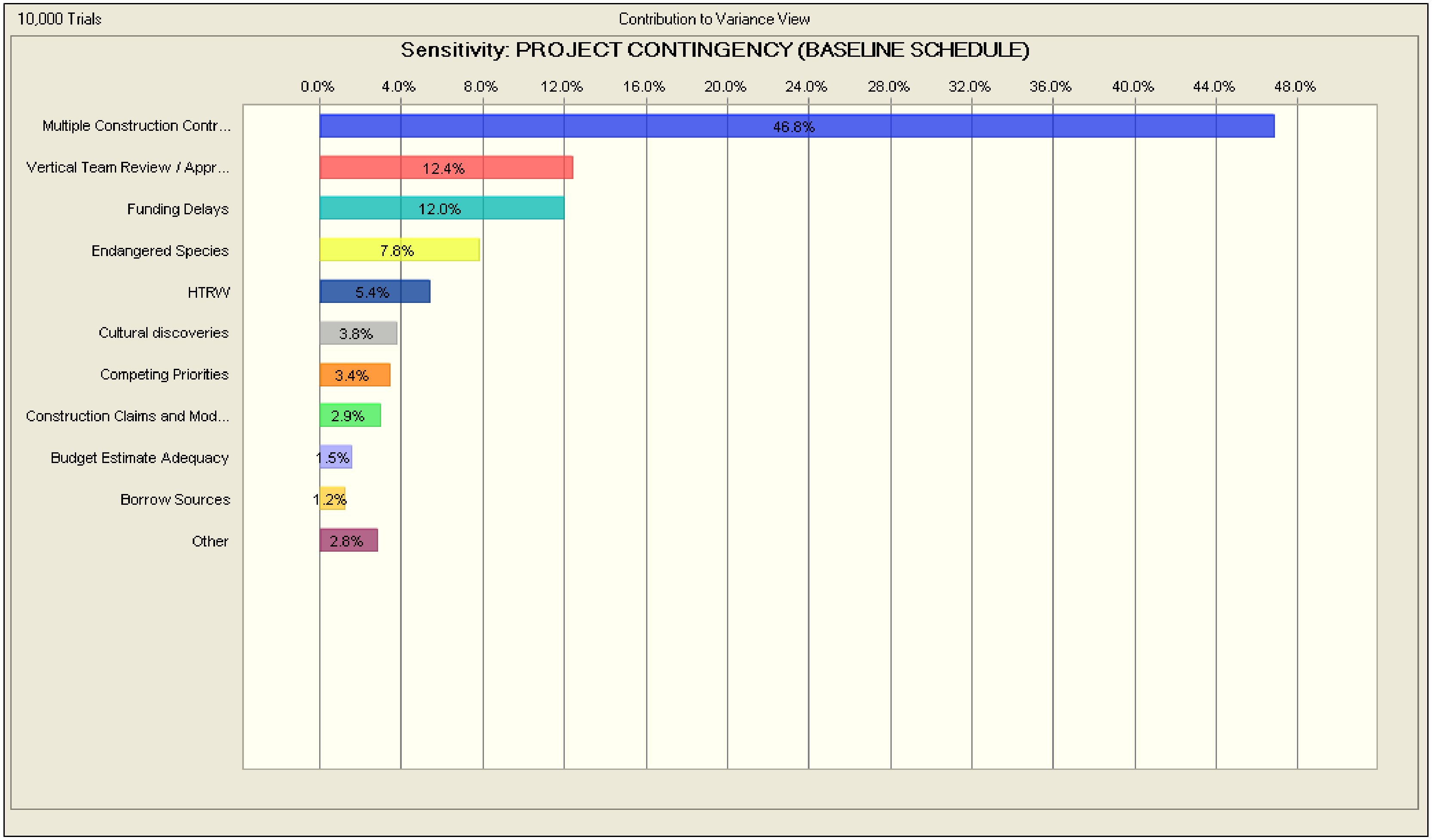


Figure 2B. Schedule Sensitivity Analysis – SB8



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 comparison summaries are provided in Table 3 and Figure 3. Additional major findings and observations of the risk analysis are listed below.

1. The key cost risk drivers identified through sensitivity analysis for both Alternatives SB7 and SB8 are CA-3 (Availability of Qualified Contractors) and CA-1 (Multiple Construction Contracts), which together contribute 75 percent of the statistical cost variance.

2. The key schedule risk drivers identified through sensitivity analysis for both Alternatives SB7 and SB8 are CA-1 (Multiple Construction Contracts), PPM-2 (Vertical Team Review and Approval) and FL-1 (Funding Delays), which together contribute some 70 percent of the statistical schedule variance.

3. Operation and maintenance activities were not included in the cost estimate or schedules. Therefore, a full life cycle risk analysis could not be performed. Risk analysis results or conclusions could be significantly different if the necessary operation and maintenance activities were included.

Table 3A. SB7 - Construction Cost Comparison Summary (Uncertainty Analysis)

Confidence Level	Project Cost ($)	Contingency ($)	Contingency (%)
0%	**$209,800,350**	**$15,752,350**	**8.12%**
5%	**$227,543,693**	**$33,495,693**	**17.26%**
10%	**$231,972,265**	**$37,924,265**	**19.54%**
15%	**$235,247,242**	**$41,199,242**	**21.23%**
20%	**$237,696,139**	**$43,648,139**	**22.49%**
25%	**$240,020,408**	**$45,972,408**	**23.69%**
30%	**$242,188,027**	**$48,140,027**	**24.81%**
35%	**$244,349,283**	**$50,301,283**	**25.92%**
40%	**$246,369,322**	**$52,321,322**	**26.96%**
45%	**$248,421,570**	**$54,373,570**	**28.02%**
50%	**$250,411,817**	**$56,363,817**	**29.05%**
55%	**$252,541,259**	**$58,493,259**	**30.14%**
60%	**$254,643,854**	**$60,595,854**	**31.23%**
65%	**$256,823,021**	**$62,775,021**	**32.35%**
70%	**$259,168,844**	**$65,120,844**	**33.56%**
75%	**$261,716,448**	**$67,668,448**	**34.87%**
80%	**$264,581,025**	**$70,533,025**	**36.35%**
85%	**$267,992,159**	**$73,944,159**	**38.11%**
90%	**$271,948,428**	**$77,900,428**	**40.14%**
95%	**$277,706,086**	**$83,658,086**	**43.11%**
100%	**$307,215,136**	**$113,167,136**	**58.32%**

Table 3B. SB8 - Construction Cost Comparison Summary (Uncertainty Analysis)

Confidence Level	Project Cost ($)	Contingency ($)	Contingency (%)
0%	**$391,772,116**	**$28,134,116**	**7.74%**
5%	**$422,824,930**	**$59,186,930**	**16.28%**
10%	**$431,001,798**	**$67,363,798**	**18.52%**
15%	**$436,624,564**	**$72,986,564**	**20.07%**
20%	**$441,020,979**	**$77,382,979**	**21.28%**
25%	**$445,349,931**	**$81,711,931**	**22.47%**
30%	**$449,430,772**	**$85,792,772**	**23.59%**
35%	**$453,213,236**	**$89,575,236**	**24.63%**
40%	**$456,886,402**	**$93,248,402**	**25.64%**
45%	**$460,663,258**	**$97,025,258**	**26.68%**
50%	**$464,623,958**	**$100,985,958**	**27.77%**
55%	**$468,139,081**	**$104,501,081**	**28.74%**
60%	**$472,170,410**	**$108,532,410**	**29.85%**
65%	**$475,882,381**	**$112,244,381**	**30.87%**
70%	**$480,241,481**	**$116,603,481**	**32.07%**
75%	**$484,956,781**	**$121,318,781**	**33.36%**
80%	**$490,028,500**	**$126,390,500**	**34.76%**
85%	**$496,174,529**	**$132,536,529**	**36.45%**
90%	**$503,436,210**	**$139,798,210**	**38.44%**
95%	**$513,495,593**	**$149,857,593**	**41.21%**
100%	**$565,245,374**	**$201,607,374**	**55.44%**

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.

1. Key Cost Risk Drivers: The key cost risk drivers identified through sensitivity analysis for both Alternatives SB7 and SB8 are CA-3 (Availability of Qualified Contractors) and CA-1 (Multiple Construction Contracts), which together contribute some 75 percent of the statistical cost variance.

a) Availability of Qualified Contractors: There is inherent risk that the ultimate bidding climate at the time of award of future contracts will be unfavorable to the price, as compared to the current working estimates of contract price. The PDT should continue to perform market research and analysis of trends within the construction industry. Ultimately, this uncertainty cannot be mitigated until more information is available. This should be communicated to management, and an adequate amount of contingency should be reserved to capture this risk.

b) Multiple Construction Contracts (Funding Constraints): Project leadership should take proactive measures to obtain decisions regarding funding and acquisition strategy, as well as communication to management regarding the impact of those decisions on cost performance.

2. Key Schedule Risk Drivers: The he key schedule risk drivers identified through sensitivity analysis for both Alternatives SB7 and SB8 are CA-1 (Multiple Construction Contracts), PPM-2 (Vertical Team Review and Approval) and FL-1 (Funding Delays), which together contribute some 70 percent of the statistical schedule variance.

a) Multiple Construction Contracts (Funding): Project leadership should take proactive measures to obtain decisions regarding funding and acquisition strategy, as well as communication to management regarding the impact of those decisions on schedule performance.

b) Vertical Team Review and Approval: Project leadership should proactively coordinate and communicate with Management (both at the District, Division and Headquarters). Ultimately, an amount and duration for this issue should be included and protected within the contingency and/or management reserve.

c) Funding Delays: Project leadership should proactively coordinate and communicate with Management (both at the District, Division and Headquarters) keeping all parties aware of probable funding and any subsequent impacts.

3. 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.

4. 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).

APPENDIX A

Sutter Basin - SB7

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost Likelihood*	Project Cost Impact*	Project Cost Risk Level*	Project Schedule Likelihood*	Project Schedule Impact*	Project 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	Project competing with other priorities	PDT Design Resources. District has assigned key personnel to various projects. Sutter Pilot study is one of two pilot studies in the nation, so has become a higher priority project. Project Feasibility Study is only funded through FY 13. The schedule currently reflects a Sept 30 Chiefs Report.	With time "priority" status has diminished. Competition for resources will remain an issue through completion of feasibility study. At this point, September 30 competition is likely but review process and unforeseen issues remain possible. A delay into next FY could significantly impact schedule due to unknown availability of future feasibility study funding after September 30.	Very Unlikely	Negligible	LOW	Unlikely	Critical	MODERATE
PPM-2	Vertical Team Review / Approval Process	Vertical Team review and approval (outside of District control) is required to meet critical milestones.	High demands on vertical teams have created a backlog of projects and pilot projects have lost much of their "priority" status.	Very Unlikely	Negligible	LOW	Likely	Critical	HIGH
PPM-3	PED Phase Staffing / Funding	Majority of design is being performed as in-kind work by the sponsor. Non-Federal Sponsor funding is in place and has not been an issue; minimal risks design will be delayed for funding or staffing issues.	Because the sponsor is funding much of the design as in-kind work, funding delays are not a concern.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
PPM-4	Scope Changes	Given the inherient nature of Feasibility Studies, changes in the project scope can be anticipated.	The local sponsor's A/E is actively developing designs and is currently approaching the 90% level. The PDT has used the A/E's 65% plans in development of the NED and LPP plans and feels they are much better prepared than typical feasibility level designs.	Very Unlikely	Marginal	LOW	Very Unlikely	Marginal	LOW
	CONTRACT ACQUISITION RISKS								
CA-1	Multiple Construction Contracts	SB-8 Construction Contracts currently divided into 5 contracts with most ~$50-$60 Million. Contracts may need to be divided into smaller increments resulting in increased construction costs, government oversight and construction schedules.	Sponsor will proceed ahead with 221 Crediting agreement, working ahead of Federal Funding.	Likely	Significant	HIGH	Likely	Significant	HIGH
CA-2	Incremental Construction Schedule	Fixing the highest risk areas with long delays between projects (5 years or more) could result in last contracts not being completed due to B/C ratios no longer being beneficial.	Projects going beyond 5 years and subject to economic re-evaluation can become problematic.	Very Unlikely	Significant	LOW	Very Unlikely	Negligible	LOW
CA-3	Availability of qualified contractors.	Number of seepage cutoff wall contractors could be limited slowing either schedule (insufficient equipment) or increasing cost (limited competition).	It is the opinion of the PDT that equipment will be available, but limited qualified contractors could lead to moderately higher costs.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	TECHNICAL RISKS								
TL-1	Borrow Sources	It has been difficult to find willing landowners to acquire impermeable (clay) borrow material. Cost estimate assumes borrow sources are available and within 25-50miles round trip. Haul could be as much as 100 miles round trip or more. Sponsor may also require additional lengths of time finding "willing" borrow sites.	Real Estate estimate has included a relatively high contingency for procurement of borrow sites.	Likely	Significant	HIGH	Likely	Marginal	MODERATE
TL-2	Changes in Geomorphology	Riprap protection for scour issues has not been included in the current design.	It is assumed that any future scour issues, when they occur, will be covered with O&M funding and outside the scope of this project.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-3	Utility Crossings	Pipe penetrations will be removed and replaced but not necessarily to USACE current design guidance. For example, some large pump stations will not be remodeled to up-and-over type pipe penetrations.	Current project design is sufficient. Given the impracticality of meeting all criteria, design waivers will be acquired and USACE criteria will not dictate future design modifications.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-4	Utility Relocations	Time requirement for coordination of relocation of utility poles could be extensive.	Sponsor is confident relocations will not impact construction award schedules.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-5	O&M Access Road	Current design assumes a 10ft to 20ft land acquisition along the entire length of the toe of the levee for an O&M access road and vegetation free zone.	Real Estate estimate assumes a worst case cost (max land acquisition) but enough uncertainties remain that no potential cost savings will be included.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-6	Utility Corridor	Several areas will require relocation of existing utilities outside the flood critical areas.	Real Estate contingency accounts for additional reaches requiring utility corridor easements.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-7	Soil Bentonite Seepage Cutoff Wall	Design assumes Soil Bentonite Cutoff wall with jet grouting at bridge and railroad crossings. This design is robust enough that any changes in design methodology will not result in cost or schedule increases.	Cost estimate assumes long stick excavation for depths up to 75' design depth and Deep Soil mixing for deeper cutoff walls.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost Likelihood*	Project Cost Impact*	Project Cost Risk Level*	Project Schedule Likelihood*	Project Schedule Impact*	Project Schedule Risk Level*
	LANDS AND DAMAGES RISKS								
LD-1	Real Estate - Utility Corridors	Majority of work is on existing levee already owned by the sponsor. Real Estate has assumed 10 ft to 20 ft permanent real estate acquisition (riverside and landside) for O&M access road and vegetation free zone. Real Estate estimate does not include baseline costs for utility relocation corridors.	Real Estate contingency accounts for additional reaches requiring utility corridor easements. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-2	Real Estate - Irrigation Canal and Levee Relocations	Real Estate estimate does not include baseline costs for irrigation canal relocation corridors.	Real Estate contingency accounts for additional relocations. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-3	Real Estate - Structural Relocations	Real Estate estimate does not include demolition costs for potential structural relocations.	There is a placeholder cost of $1,920,460 in the appraisal. When buildings are impacted it is not unusual for agencies to acquire the entire property (land/building) and make necessary changes---altering or raising the buildings and than resale the remainder. This helps to alleviate the time and cost associated with litigation or working with property owners. It is less costly to acquire the entire property when improvements will be impacted versus trying to modify the existing improvements and compensating property owners for damages. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-4	Real Estate - Temporary Construction Easements	Temporary construction easements have been assumed along the length of the levee construction.	Staging areas have been identified already in the project area. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW

Risk | Risk/Opportunity Event | Concerns | Project Cost | Project Schedule

No.			PDT Discussions & Conclusions	Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	REGULATORY AND ENVIRONMENTAL RISKS								
RE-1	Air quality	Contractor will require newer equipment to meet air quality requirement, but air quality credits aren't anticipated.	Anticipate qualified California contractor will have worked previous projects with appropriate equipment.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-2	Known cultural Sites	Estimate includes 1% for cultural impacts.	Historical structures downtown will require vibration monitoring.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-3	Cultural discoveries	Cost Estimate includes 1% for cultural investigations.	Majority of work will occur in existing levees, but it is still possible cultural discoveries could be made during construction. Cultural reconnaissance will occur prior to construction and limit possibility of discovery during construction. If cultural discoveries are made, construction must stop in that area. Cultural discovery must be resolved before construction can resume in that reach. IF discovery is made anticipate 3 to 6 month impact. Some 3 miles of Levee and Canal Realignment are required through new previously untouched regions; but greater cultural reconnaissance will be conducted in these areas minimizing potential schedule impacts.	Unlikely	Significant	MODERATE	Likely	Marginal	MODERATE
RE-4	Endangered Species	Construction windows are constrained by Federal and State endangered species windows. Work is currently scheduled outside most species windows but Swainson's Hawk will nest in early spring and fledge in early September.	There is the possibility work could be halted around any nesting areas. Bird surveys may be conducted the prior year to determine risk. (Construction schedule for Irrigation canal Jan-March and Levee April - October).	Very Unlikely	Negligible	LOW	Likely	Marginal	MODERATE
RE-5	Historic Structures	There are a number of historical structures that may or may not need to be relocated, specifically in Yuba City.	Cultural inventories will identify historic structures and assess possible adverse effects. If a historic structure is identified for relocation mitigation for that resources would be governed by a Memorandum of Agreement coordinated with SHPO.	Likely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-6	HTRW	There may be HTRW sites that are unknown	It is unlikely that HTRW waste be encounter. If HTRW waste is encountered in would not affect cost but the schedule may be affected.	Unlikely	Marginal	LOW	Unlikely	Significant	MODERATE

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	CONSTRUCTION RISKS								
CON-1	Seepage Cutoff Wall and Utility Penetrations	Replacement construction of Utility Penetration can't begin until after seepage cutoff wall construction has been completed possibly resulting in long periods of temporary service. Costs have been included for temporary up-and-over services for a limited number of sites (4months each site) .	SB7 Levee has fewer gravity flow utilities (more up-and-over type levee crossings) so likely a marginal cost impact.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
CON-2	Availability of Bentonite	There is risk of escalation on bentonite, pea gravel and course sand. There may be come shortages that could impact the costs and schedule.	In the past, contractor for Mayhew Levee raise encountered difficulties procuring sufficient supplies of bentonite. Bentonite has many applications, including in oil drilling. If multiple other projects also requiring bentonite are under construction concurrently, this could be an issue. Pea gravel and course sand have also presented acquisition issue in the past as well.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW
CON-3	Cobbles	Cobbles in the area can slow or even prevent the construction of seepage cutoff walls.	Seepage berms have been included in the design and cost estimate to account for these problematic areas but could anticipate greater numbers required with only a minimal cost/schedule impacts.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
CON-4	Slurry Blowout During Construction	In the event of slurry blowout, would require greater levee degradation, suspension of work during cleanup and additional backfill required.	Worst case assume one blowout every 5 miles at a cost of $500,000 per blowout. The levee is far enough from the river that seepage into the river and potential environmental impacts is not anticipated.	Likely	Significant	HIGH	Very Unlikely	Negligible	LOW
CON-5	Vagrancy and Loitering Issues	There is the issue of vandalism and damage to the contractor, and there may be some risk transference to the contractor.	The likelihood of claims initiated by the contractor is negligible.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
CON-6	Soil Bentonite Wall - Backfill Material	Consistence of backfill material gradations, specification are reasonable per the drill logs and existing conditions at each site	Historically these types of SB wall contracts include a provision that the KTR use on-site material with a mix of import to meet the backfill requirements. This mixing and subsequent testing of the mix are performed on-site with laboratory results to follow in 3 days. By the time laboratory results are provided backfill has been placed and it becomes a battle on if we remove and replace or give the KTR consideration.	Very Likely	Significant	HIGH	Likely	Marginal	MODERATE
CON-MOD	Modifications and Claims	There is inherent risk of construction modifications and claims that arise after contract award due to issues such as weather, schedules dictated by O&M cycles, differing site conditions, user directed changes or omissions, inaccurate surveys, and variations in estimated quantities (minor).	Post-award construction contract modifications and claims could impact the ultimate contract costs and delay the overall schedule.	Likely	Significant	HIGH	Likely	Significant	HIGH

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	ESTIMATE AND SCHEDULE RISKS								
EST-1	Railroad Crossing	Railroad crossing is currently below crest of levee.	Estimate includes cost of stop log closure structure. May not include costs for establishing temporary railroad services or outages.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW
EST-2	Budget Estimate Adequacy	All feature codes are currently captured in the estimate. However, there may be some uncertainty in the disposition of some feature codes.	Crews, assemblies, productivities, and methodologies in the current PCE may not adequately capture ultimate actual contractor technique and costs.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
	ECONOMICS RISKS								
FL-1	Funding Delays	With extended funding lags could be multiple years before funding arrives. Protracted construction places the project at greater risks related to more stringent environmental restrictions, scope changes, political changes, escalation exceeding OMB projections, greater potential for extreme commodity availability	Much of this issue exists outside of the scope of the PDT's control, but it is anticipated there will likely be schedule delays and cost increases due to funding lags.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
INT-1	Internal Risk	There is inherent risk in all projects that could contribute to cost and schedule variance due to unknowns.	This could impact cost and schedule.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
	Programmatic Risks	(External Risk Items are those that are generated, caused, or controlled exclusively outside the PDT's sphere of influence.)							
PR-1	System Wide Improvement Framework (SWIF)	Agreement on ETL vegetation requirements will require negotiation and agreement between three parties (USACE, State of California, and Levee Sponsor) in addition to third party entities.	Cost estimate does not include cost for additional vegetation removal. It may be possible it will be decided this removal will be a project cost (as opposed to O&M).	Likely	Critical	HIGH	Very Unlikely	Negligible	LOW
PR-2	Central Valley Flood Protection Plan	A statewide systemwide program that includes the Sacramento Flood Control Project (study project levees).	Affects all Central Valley studies. Future efforts or alternatives of current studies coordinated as "no regrets actions."	Unlikely	Negligible	LOW	Unlikely	Negligible	LOW
EXT-1	External Risk	There is inherent risk in all projects that could contribute to cost and schedule variance due to unknowns.	This could impact cost and schedule.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE

Sutter Basin - SB8

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	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	Project competing with other priorities	PDT Design Resources. District has assigned key personnel to various projects. Sutter Pilot study is one of two pilot studies in the nation, so has become a higher priority project. Project Feasibility Study is only funded through FY 13. The schedule currently reflects a Sept 30 Chiefs Report.	With time "priority" status has diminished. Competition for resources will remain an issue through completion of feasibility study. At this point, September 30 competition is likely but review process and unforeseen issues remain possible. A delay into next FY could significantly impact schedule due to unknown availability of future feasibility study funding after September 30.	Very Unlikely	Negligible	LOW	Unlikely	Critical	MODERATE
PPM-2	Vertical Team Review / Approval Process	Vertical Team review and approval (outside of District control) is required to meet critical milestones.	High demands on vertical teams have created a backlog of projects and pilot projects have lost much of their "priority" status.	Very Unlikely	Negligible	LOW	Likely	Critical	HIGH
PPM-3	PED Phase Staffing / Funding	Majority of design is being performed as in-kind work by the sponsor. Non-Federal Sponsor funding is in place and has not been an issue; minimal risks design will be delayed for funding or staffing issues.	Because the sponsor is funding much of the design as in-kind work, funding delays are not a concern.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
PPM-4	Scope Changes	Given the inherient nature of Feasibility Studies, changes in the project scope can be anticipated.	The local sponsor's A/E is actively developing designs and is currently approaching the 90% level. The PDT has used the A/E's 65% plans in development of the NED and LPP plans and feels they are much better prepared than typical feasibility level designs.	Very Unlikely	Marginal	LOW	Very Unlikely	Marginal	LOW
	CONTRACT ACQUISITION RISKS								
CA-1	Multiple Construction Contracts	SB-8 Construction Contracts currently divided into 7 contracts with most ~$50-$60 Million. Contracts may need to be divided into smaller increments resulting in increased construction costs, government oversight and construction schedules.	Sponsor will proceed ahead with 221 Crediting agreement, working ahead of Federal Funding.	Likely	Significant	HIGH	Likely	Significant	HIGH
CA-2	Incremental Construction Schedule	Fixing the highest risk areas with long delays between projects (5 years or more) could result in last contracts not being completed due to B/C ratios no longer being beneficial.	Projects going beyond 5 years and subject to economic re-evaluation can become problematic.	Very Unlikely	Significant	LOW	Very Unlikely	Negligible	LOW
CA-3	Availability of qualified contractors.	Number of seepage cutoff wall contractors could be limited slowing either schedule (insufficient equipment) or increasing cost (limited competition).	It is the opinion of the PDT that equipment will be available, but limited qualified contractors could lead to moderately higher costs.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	TECHNICAL RISKS								
TL-1	Borrow Sources	It has been difficult to find willing landowners to acquire impermeable (clay) borrow material. Cost estimate assumes borrow sources are available and within 25-50miles round trip. Haul could be as much as 100 miles round trip or more. Sponsor may also require additional lengths of time finding "willing" borrow sites.	Real Estate estimate has included a relatively high contingency for procurement of borrow sites.	Likely	Significant	HIGH	Likely	Marginal	MODERATE
TL-2	Changes in Geomorphology	Riprap protection for scour issues has not been included in the current design.	It is assumed that any future scour issues, when they occur, will be covered with O&M funding and outside the scope of this project.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-3	Utility Crossings	Pipe penetrations will be removed and replaced but not necessarily to USACE current design guidance. For example, some large pump stations will not be remodeled to up-and-over type pipe penetrations.	Current project design is sufficient. Given the impracticality of meeting all criteria, design waivers will be acquired and USACE criteria will not dictate future design modifications.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-4	Utility Relocations	Time requirement for coordination of relocation of utility poles could be extensive.	Sponsor is confident relocations will not impact construction award schedules.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-5	O&M Access Road	Current design assumes a 10ft to 20ft land acquisition along the entire length of the toe of the levee for an O&M access road and vegetation free zone.	Real Estate estimate assumes a worst case cost (max land acquisition) but enough uncertainties remain that no potential cost savings will be included.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-6	Utility Corridor	Several areas will require relocation of existing utilities outside the flood critical areas.	Real Estate contingency accounts for additional reaches requiring utility corridor easements.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-7	Soil Bentonite Seepage Cutoff Wall	Design assumes Soil Bentonite Cutoff wall with jet grouting at bridge and railroad crossings. This design is robust enough that any changes in design methodology will not result in cost or schedule increases.	Cost estimate assumes long stick excavation for depths up to 75' design depth and Deep Soil mixing for deeper cutoff walls.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
TL-8	Abandoned Drainage penetrations	Cost included for removal of abandoned penetrations. Additional engineering effort will be required to justify no internal drainage issues will be caused.	Additional effort will have minimal impacts to design cost and schedule.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	LANDS AND DAMAGES RISKS								
LD-1	Real Estate - Utility Corridors	Majority of work is on existing levee already owned by the sponsor. Real Estate has assumed 10 ft to 20 ft permanent real estate acquisition (riverside and landside) for O&M access road and vegetation free zone. Real Estate estimate does not include baseline costs for utility relocation corridors.	Real Estate contingency accounts for additional reaches requiring utility corridor easements. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-2	Real Estate - Irrigation Canal and Levee Relocations	Real Estate estimate does not include baseline costs for irrigation canal relocation corridors.	Real Estate contingency accounts for additional relocations. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-3	Real Estate - Structural Relocations	Real Estate estimate does not include demolition costs for potential structural relocations.	There is a placeholder cost of $1,920,460 in the appraisal. When buildings are impacted it is not unusual for agencies to acquire the entire property (land/building) and make necessary changes---altering or raising the buildings and than resale the remainder. This helps to alleviate the time and cost associated with litigation or working with property owners. It is less costly to acquire the entire property when improvements will be impacted versus trying to modify the existing improvements and compensating property owners for damages. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
LD-4	Real Estate - Temporary Construction Easements	Temporary construction easements have been assumed along the length of the levee construction.	Staging areas have been identified already in the project area. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	REGULATORY AND ENVIRONMENTAL RISKS								
RE-1	Air quality	Contractor will require newer equipment to meet air quality requirement, but air quality credits aren't anticipated.	Anticipate qualified California contractor will have worked previous projects with appropriate equipment.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-2	Known cultural Sites	Estimate includes 1% for cultural impacts.	Historical structures downtown will require vibration monitoring.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-3	Cultural discoveries	Cost Estimate includes 1% for cultural investigations.	Majority of work will occur in existing levees, but it is still possible cultural discoveries could be made during construction. Cultural reconnaissance will occur prior to construction and limit possibility of discovery during construction. If cultural discoveries are made, construction must stop in that area. Cultural discovery must be resolved before construction can resume in that reach. IF discovery is made anticipate 3 to 6 month impact. Some 3 miles of Levee and Canal Realignment are required through new previously untouched regions; but greater cultural reconnaissance will be conducted in these areas minimizing potential schedule impacts.	Unlikely	Significant	MODERATE	Likely	Marginal	MODERATE
RE-4	Endangered Species	Construction windows are constrained by Federal and State endangered species windows. Work is currently scheduled outside most species windows but Swainson's Hawk will nest in early spring and fledge in early September.	There is the possibility work could be halted around any nesting areas. Bird surveys may be conducted the prior year to determine risk. (Construction schedule for Irrigation canal Jan-March and Levee April - October).	Very Unlikely	Negligible	LOW	Likely	Marginal	MODERATE
RE-5	Historic Structures	There are a number of historical structures that may or may not need to be relocated, specifically in Yuba City.	Cultural inventories will identify historic structures and assess possible adverse effects. If a historic structure is identified for relocation mitigation for that resources would be governed by a Memorandum of Agreement coordinated with SHPO.	Likely	Negligible	LOW	Very Unlikely	Negligible	LOW
RE-6	HTRW	There may be HTRW sites that are unknown	It is unlikely that HTRW waste be encounter. If HTRW waste is encountered in would not affect cost but the schedule may be affected.	Unlikely	Marginal	LOW	Unlikely	Significant	MODERATE

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	CONSTRUCTION RISKS								
CON-1	Seepage Cutoff Wall and Utility Penetrations	Replacement construction of Utility Penetration can't begin until after seepage cutoff wall construction has been completed possibly resulting in long periods of temporary service. Costs have been included for temporary up-and-over services for a limited number of sites (4months each site) .	SB8 Levee reach has multiple gravity flow lines that could be impacted.	Likely	Significant	HIGH	Likely	Marginal	MODERATE
CON-2	Availability of Bentonite	There is risk of escalation on bentonite, pea gravel and course sand. There may be come shortages that could impact the costs and schedule.	In the past, contractor for Mayhew Levee raise encountered difficulties procuring sufficient supplies of bentonite. Bentonite has many applications, including in oil drilling. If multiple other projects also requiring bentonite are under construction concurrently, this could be an issue. Pea gravel and course sand have also presented acquisition issue in the past as well.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW
CON-3	Cobbles	Cobbles in the area can slow or even prevent the construction of seepage cutoff walls. Seepage berms have been included in the design and cost estimate to account for these problematic areas but could anticipate greater numbers required.	Greater likelihood of encountering cobbles in SB8 regions, but larger numbers of seepage berms have also been included so assume minimal impacts.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
CON-4	Slurry Blowout During Construction	In the event of slurry blowout, would require greater levee degradation, suspension of work during cleanup and additional backfill required.	Worst case assume one blowout every 5 miles at a cost of $500,000 per blowout. The levee is far enough from the river that seepage into the river and potential environmental impacts is not anticipated.	Likely	Significant	HIGH	Very Unlikely	Negligible	LOW
CON-5	Vagrancy and Loitering Issues	There is the issue of vandalism and damage to the contractor, and there may be some risk transference to the contractor.	The likelihood of claims initiated by the contractor is negligible.	Very Unlikely	Negligible	LOW	Very Unlikely	Negligible	LOW
CON-6	Soil Bentonite Wall - Backfill Material	Consistence of backfill material gradations, specification are reasonable per the drill logs and existing conditions at each site	Historically these types of SB wall contracts include a provision that the KTR use on-site material with a mix of import to meet the backfill requirements. This mixing and subsequent testing of the mix are performed on-site with laboratory results to follow in 3 days. By the time laboratory results are provided backfill has been placed and it becomes a battle on if we remove and replace or give the KTR consideration.	Very Likely	Significant	HIGH	Likely	Marginal	MODERATE
CON-MOD	Modifications and Claims	There is inherent risk of construction modifications and claims that arise after contract award due to issues such as weather, schedules dictated by O&M cycles, differing site conditions, user directed changes or omissions, inaccurate surveys, and variations in estimated quantities (minor).	Post-award construction contract modifications and claims could impact the ultimate contract costs and delay the overall schedule.	Likely	Significant	HIGH	Likely	Significant	HIGH

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions & Conclusions	Project Cost			Project Schedule		
				Likelihood*	Impact*	Risk Level*	Likelihood*	Impact*	Risk Level*
	ESTIMATE AND SCHEDULE RISKS								
EST-1	Railroad Crossing	Railroad crossing is currently below crest of levee.	Estimate includes cost of stop log closure structure. May not include costs for establishing temporary railroad services or outages.	Likely	Marginal	MODERATE	Very Unlikely	Negligible	LOW
EST-2	Budget Estimate Adequacy	All feature codes are currently captured in the estimate. However, there may be some uncertainty in the disposition of some feature codes.	Crews, assemblies, productivities, and methodologies in the current PCE may not adequately capture ultimate actual contractor technique and costs.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
	ECONOMICS RISKS								
FL-1	Funding Delays	With extended funding lags could be multiple years before funding arrives. Protracted construction places the project at greater risks related to more stringent environmental restrictions, scope changes, political changes, escalation exceeding OMB projections, greater potential for extreme commodity availability	Much of this issue exists outside of the scope of the PDT's control, but it is anticipated there will likely be schedule delays and cost increases due to funding lags.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
INT-1	Internal Risk	There is inherent risk in all projects that could contribute to cost and schedule variance due to unknowns.	This could impact cost and schedule.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE
	Programmatic Risks	(External Risk Items are those that are generated, caused, or controlled exclusively outside the PDT's sphere of influence.)							
PR-1	System Wide Improvement Framework (SWIF)	Agreement on ETL vegetation requirements will require negotiation and agreement between three parties (USACE, State of California, and Levee Sponsor) in addition to third party entities.	Cost estimate does not include cost for additional vegetation removal. It may be possible it will be decided this removal will be a project cost (as opposed to O&M).	Likely	Critical	HIGH	Very Unlikely	Negligible	LOW
PR-2	Central Valley Flood Protection Plan	A statewide systemwide program that includes the Sacramento Flood Control Project (study project levees).	Affects all Central Valley studies. Future efforts or alternatives of current studies coordinated as "no regrets actions."	Unlikely	Negligible	LOW	Unlikely	Negligible	LOW
EXT-1	External Risk	There is inherent risk in all projects that could contribute to cost and schedule variance due to unknowns.	This could impact cost and schedule.	Likely	Marginal	MODERATE	Likely	Marginal	MODERATE

WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

Project No. 105638

SPK – Sutter Basin Project

Two Alternatives for the Sutter Basin Project, as presented by Sacramento District, have 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 scopes, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the 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 October 10, 2013, the Cost MCX certifies the estimated total project cost of the two alternatives:

ALTERNATIVE SB-7
FY 2014 Price Level: $391,840,000
Fully Funded Amount: $440,530,000

ALTERNATIVE SB-8
FY 2014 Price Level: $688,930,000
Fully Funded Amount: $791,970,000

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.

CALLAN.KIM.C.1231558221

Digitally signed by CALLAN.KIM.C.1231558221
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=CALLAN.KIM.C.1231558221
Date: 2013.10.11 07:30:26 -07'00'

Kim C. Callan, PE, CCE, PM1
Chief, Cost Engineering MCX
Walla Walla District

US Army Corps of Engineers®

****TOTAL PROJECT COST SUMMARY****

10/10/2013

THIS ESTIMATE IS BASED ON THE SCOPE CONTAINED IN THE DRAFT FEASIBILITY REPORT, ALT. SB-7
PROJECT: Sutter Basin TPCS
LOCATION: CALIFORNIA

U. S. ARMY CORPS OF ENGINEER, SACRAMENTO DISTRICT
P.O.C.: JEREMIAH A. FROST, CHIEF, COST ENGINEERING SECTION

Current MCACES Estimate Prepared: 25-Jul-2013
Effective Price Level (EPL): 1-Oct-2013

PROGRAM YEAR(BUDGET EC) 2014
EFF. PRICE LEVEL DATE:1-Oct-2013

TOTAL PROJECT COST (FULLY FUNDED)

WB NO.	Civil Works FEATURE DESCRIPTION	ESTIMATED COST: COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC. (%)	PROJECT FIRST COST: COST ($K)	CNTG ($K)	TOTAL ($K)	SPENT THRU: 1-Oct-2013 COST ($K)	ESC. MIDPT(%)	COST ($K)	CNTG ($K)	FULLY FUNDED ($K)
Index Codes: 0 - no esc. applied; A - Administration; C - Combined indexes; All other codes used coincides with the Code of Accounts.														
		Contingency Applied To Remaining Cost Only												
	FEDERAL COSTS													
6	FISH & WILDLIFE FACILITIES	5,032	1,006	20	6,038	0.00	5,032	1,006	6,038	0	12	5,611	1,122	6,733
11	LEVEES & FLOODWALLS	176,205	63,717	36	239,922	0.00	176,205	63,717	239,922	0	11	196,085	70,906	266,991
18	CULT. RESRC. PRESERV. (1	1,655	598		2,253		1,655	598	2,253	0		1,841	665	2,506
	Data Recovery	1,200	433		1,633	0.00	1,200	433	1,633	0		1,334	482	1,816
	Inventory/Evaluation/Mitigation Costs	455	165	36	620	0.00	455	165	620	0	11	507	183	690
	SUBTOTAL FEDERAL & NON-FEDERAL CONSTRUCTION COSTS	182,892	65,321		248,213		182,892	65,321	248,213	0		203,537	72,693	276,230
1	LANDS & DAMAGES, Admin (2	6,952	348	5	7,300	0.00	6,952	348	7,300	0	17	8,168	408	8,576
30	PLAN/ENGINEERING/DESIGN	32,622	11,797	36	44,419	0.00	32,622	11,797	44,419	0	18	38,534	13,934	52,468
31	CONSTRUCTION MANAGE'MT	15,406	5,570	36	20,976	0.00	15,406	5,570	20,976	0	23	18,943	6,849	25,792
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION	237,872	83,036		320,908		237,872	83,036	320,908	0		269,182	93,884	363,066
	NON-FEDERAL CONTRIBUTION (-)	-48,533	-17,105		-65,638		-48,533	-17,105	-65,638	0		-56,289	-19,847	-76,136
	TOTAL FEDERAL COSTS	**$189,339**	**$65,931**		**$255,270**		**$189,339**	**$65,931**	**$255,270**	**$0**		**$212,893**	**$74,037**	**$286,930**
	NON-FEDERAL COSTS													
1	LANDS AND DAMAGES	31,811	10,579	33	42,390	0.00	31,811	10,579	42,390	0	8.5	34,523	11,481	46,004
2	RELOCATIONS	20,962	7,580		28,542		20,962	7,580	28,542	0		23,105	8,355	31,460
	Relocations Construction Cost	16,376	5,922	36	22,298	0.00	16,376	5,922	22,298	0	10	18,074	6,536	24,610
	Plan/Engineering/Design	2,948	1,066	36	4,014	0.00	2,948	1,066	4,014	0	8.8	3,209	1,160	4,369
	Construction Mangement	1,638	592	36	2,230	0.00	1,638	592	2,230	0	11	1,822	659	2,481
	SUBTOTAL NON-FEDERAL	52,773	18,159		70,932		52,773	18,159	70,932	0		57,628	19,836	77,464
	NON-FEDERAL CONTRIBUTION (+)	48,533	17,105		65,638		48,533	17,105	65,638	0		56,289	19,847	76,136
	TOTAL NON-FEDERAL COSTS	**$101,306**	**$35,264**		**$136,570**		**$101,306**	**$35,264**	**$136,570**	**$0**		**$113,917**	**$39,683**	**$153,600**
	TOTAL FEDERAL AND NON-FEDERAL COSTS	**$290,645**	**$101,195**		**$391,840**		**$290,645**	**$101,195**	**$391,840**	**$0**		**$326,810**	**$113,720**	**$440,530**

GENERAL NOTES

(1 Cultural Resources Preservation costs was provided by Cultural Resources Archaeologist.
(2 Federal administrative costs for non-Federal land acquisition.
(3 The Fully Funded cost estimate was prepared in compliance with Indexes used from CWCCIS reflecting OMB future rates Mar. 31, 2013
(4 01 Account for Land and Damages cost are from Real Estates.
(5 06 Account Fish and Wildlife Cost was provided by SPK Environmental Planning.
(6 30 Account Planning, Engineering and Design and 31 Account Construction Management cost was provided by its respective organizations.

CONTINGENCY RATIONALE

(A CONTINGENCIES USED WAS DERIVED BY THE COST RISK ANALYSIS PROCESS AND IS BASED ON A 80% CONFIDENCE LEVEL

______________________ **CHIEF, COST ENGINEERING**

______________________ **PROJECT MANAGER**

______________________ **CHIEF, REAL ESTATE**

TOTAL FEDERAL COSTS	**$286,930**
TOTAL NON-FEDERAL COSTS	**$153,600**
THE MAXIMUM PROJECT COSTS	**$440,530**

****TOTAL PROJECT COST SUMMARY****

10/10/2013

THIS ESTIMATE IS BASED ON THE SCOPE CONTAINED IN THE DRAFT FEASIBILITY REPORT, ALT. SB-8
PROJECT: Sutter Basin TPCS
LOCATION: CALIFORNIA

U. S. ARMY CORPS OF ENGINEER, SACRAMENTO DISTRICT
P.O.C.: JEREMIAH A. FROST, CHIEF, COST ENGINEERING SECTION

Current MCACES Estimate Prepared: 25-Jul-2013
Effective Price Level (EPL): 1-Oct-2013

WBS NO.	Civil Works FEATURE DESCRIPTION		ESTIMATED COST COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	PROGRAM YEAR(BUDGET EC) 2014 EFF. PRICE LEVEL DATE:1-Oct-2013 ESC. (%)	PROJECT FIRST COST COST ($K)	CNTG ($K)	TOTAL ($K)	TOTAL PROJECT COST (FULLY FUNDED) SPENT THRU: 1-Oct-2013 COST ($K)	ESC. MIDPT(%)	COST ($K)	CNTG ($K)	FULLY FUNDED ($K)
Index Codes: 0 - no esc. applied; A - Administration; C - Combined indexes; All other codes used coincides with the Code of Accounts.															
			Contingency Applied To Remaining Cost Only												
	FEDERAL COSTS														
6	FISH & WILDLIFE FACILITIES		6,330	1,265	20	7,595	0.00	6,330	1,265	7,595	0	14	7,226	1,445	8,671
11	LEVEES & FLOODWALLS		306,367	106,488	35	412,855	0.00	306,367	106,488	412,855	0	13	347,604	120,821	468,425
18	CULT. RESRC. PRESERV. (1		3,030	1,076		4,106		3,030	1,076	4,106			3,399	1,207	4,606
	Federal Obligations from NED Cost.		1,655	598		2,253		1,655	598	2,253			1,841	665	2,506
	Data Recovery		1,200	433		1,633		1,200	433	1,633	0		1,334	482	1,816
	Inventory/Evaluation/Mitigation Costs		455	165		620		455	165	620	0		507	183	690
	Cost Beyond NED Cost.		1,375	478		1,853		1,375	478	1,853	0		1,558	542	2,100
	Data Recovery	18	1,000	348	35	1,348	0.00	1,000	348	1,348	0	13	1,134	394	1,528
	Inventory/Evaluation/Mitigation Costs	18	375	130	35	505	0.00	375	130	505	0	13	424	148	572
	SUBTOTAL FEDERAL & NON-FEDERAL CONSTRUCTION COSTS		315,727	108,829		424,556		315,727	108,829	424,556	0		358,229	123,473	481,702
1	LANDS & DAMAGES, Admin (2		11,143	557	5	11,700	0.00	11,143	557	11,700	0	22	13,549	677	14,226
30	PLAN/ENGINEERING/DESIGN		56,285	19,565	35	75,850	0.00	56,285	19,565	75,850	0	22	68,804	23,916	92,720
31	CONSTRUCTION MANAGE'MT		26,580	9,239	35	35,819	0.00	26,580	9,239	35,819	0	27	33,791	11,746	45,537
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION		409,735	138,190		547,925		409,735	138,190	547,925	0		474,373	159,812	634,185
	NON-FEDERAL CONTRIBUTION(-)		-220,396	-72,259		-292,655		-220,396	-72,259	-292,655	0		-261,480	-85,775	-347,255
	TOTAL FEDERAL NED COSTS		**$189,339**	**$65,931**		**$255,270**		**$189,339**	**$65,931**	**$255,270**	**$0**		**$212,893**	**$74,037**	**$286,930**
	NON-FEDERAL COSTS														
1	LANDS AND DAMAGES		41,795	11,751	28	53,546	0.00	41,795	11,751	53,546	0	11	46,222	12,995	59,217
2	RELOCATIONS		64,900	22,559		87,459		64,900	22,559	87,459	0		73,143	25,425	98,568
	Relocations Construction Cost		50,703	17,624	35	68,327	0.00	50,703	17,624	68,327	0	13	57,271	19,907	77,178
	Plan/Engineering/Design		9,127	3,172	35	12,299	0.00	9,127	3,172	12,299	0	11	10,123	3,519	13,642
	Construction Management		5,070	1,763	35	6,833	0.00	5,070	1,763	6,833	0	13	5,749	1,999	7,748
	SUBTOTAL NON-FEDERAL		106,695	34,310		141,005		106,695	34,310	141,005	0		119,365	38,420	157,785
	NON-FEDERAL CONTRIBUTION (+)		220,396	72,259		292,655		220,396	72,259	292,655	0		261,480	85,775	347,255
	Non-Federal Contribution - NED		48,533	17,105		65,638		48,533	17,105	65,638	0		56,289	19,847	76,136
	Additional Cost Above NED		171,863	55,154		227,017		171,863	55,154	227,017	0		205,191	65,928	271,119
	TOTAL NON-FEDERAL COSTS		**$327,091**	**$106,569**		**$433,660**		**$327,091**	**$106,569**	**$433,660**	**$0**		**$380,845**	**$124,195**	**$505,040**
	TOTAL FEDERAL AND NON-FEDERAL COSTS		**$516,430**	**$172,500**		**$688,930**		**$516,430**	**$172,500**	**$688,930**	**$0**		**$593,738**	**$198,232**	**$791,970**

GENERAL NOTES

(1 Cultural Resources Preservation costs was provided by Cultural Resources Archaeologist.
(2 Federal administrative costs for non-Federal land acquisition.
(3 The Fully Funded cost estimate was prepared in compliance with Indexes used from CWCCIS reflecting OMB future rates Mar. 31, 2013
(4 01 Account for Land and Damages cost are from Real Estates.
(5 06 Account Fish and Wildlife Cost was provided by SPK Environmental Planning.
(6 30 Account Planning, Engineering and Design and 31 Account Construction Management cost was provided by its respective organizations.

(A CONTINGENCIES USED WAS DERIVED BY THE COST RISK ANALYSIS PROCESS AND IS BASED ON A 80% CONFIDENCE LEVEL

	DOLLAR(K)
TOTAL FEDERAL COSTS	**$286,930**
TOTAL NON-FEDERAL COSTS	**$505,040**
THE MAXIMUM PROJECT COSTS	**$791,970**

______________________ **CHIEF, COST ENGINEERING**

______________________ **PROJECT MANAGER**

______________________ **CHIEF, REAL ESTATE**

US Army Corps
of Engineers®

Sacramento District

Engineering Division

Sutter Basin Pilot Feasibility Report - Environmental Impact Report / Supplemental Environmental Impacts Statement

Butte and Sutter Counties, California

Civil Design Appendix

October 2013

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REFERENCES

- USACE EM 1110-2-1913, “Design and Construction of Levees”, 30 April 2000
- CESPK-ED-G, SOP-EDG-03 “Geotechnical Levee Practice”, 11 April 2008

ACRONYMS

CCEL Cherokee Canal East Levee (Left Levee)
EIP Early Implementation Project (local sponsor’s Plan)
FRWL Feather River West Levee (Right Levee)
MEIP Modified EIP (COE’s modifications to the EIP)
O&M Operation and Maintenance
ROW Right of Way
SBEL Sutter Bypass East Levee (Left Levee)
SBLS Sutter Basin Levee System
WCEL Wadsworth Canal East Levee (Left Levee)
WCWL Wadsworth Canal West Levee (Right Levee)

CHAPTER 1 – INTRODUCTION

1.1 Project Description

The existing Sutter Basin Levee System (SBLS) consists of four mainline levees which are Feather River West Levee (FRWL or right levee), Sutter Bypass East Levee (SBEL or left levee), Wadsworth Canal East Levee (WCEL or left levee) and Wadsworth Canal West Levee (WCWL or right levee), and Cherokee Canal East Levee (CCEL or left levee) surrounding the communities of Yuba City, Live Oak, Gridley, Biggs and other smaller towns in Sutter and Butte Counties, California.

During the preliminary phase of this Feasibility Study, many potential remediation measures were considered and combined to form a preliminary array of conceptual alternatives. Through plan formulation process, the preliminary array was refined to a draft array that includes 8 potential alternatives:

- SB-1: No Action.
- SB-2: Minimal Fix-in-place the FRWL from Star Bend to Sunset Weir
- SB-3: Yuba City Ring Levee
- SB-4: Little "J" Levee
- SB-5: Fix-in-place the FRWL from Star Bend to Thermalito Afterbay
- SB-6: Fix-in-Place the FRWL, SBEL and WCEL
- SB-7: Fix-in-Place the FRWL from Laurel Avenue to Sunset Weir
- SB-8: Fix-in-Place the FRWL from Laurel Avenue to Thermalito Afterbay.

The draft array was analyzed and refined to a final array that includes 3 alternatives, SB-1, SB-7 and SB-8. During the final phase of this Feasibility Study, alternatives SB-7 and SB-8 were further evaluated to determine the Recommended Plan for final recommendation. (Plates 1-1 to 1-8 depict the extent of the potential alternatives included in the draft array).

1.2 Purpose and Scope

The purpose of this report is to provide a summary of the civil design evaluation of and consideration for the draft array. The evaluation is a refinement of the preliminary analysis completed for the conceptual alternatives and conforms to the minimum requirements for the development of Class 4 estimate for reconnaissance level analysis. (The preliminary analysis of the conceptual alternatives is documented in enclosure 1, Evaluation of Preliminary Array of Conceptual Alternatives. Classification of the estimate was in accordance with EM 1110-2-1302, Civil Works Cost Engineering, which was based on ASTM E 2516-06, Standard Classification for Cost Estimate Classification System.)

The civil design evaluation of and consideration for alternatives SB-7 and SB-8 of the final array are discussed in paragraph 2.9 of the Engineering Appendix and conform to the minimum requirements for the development of Class 3 estimate for feasibility level analysis. (Enclosure 2, Review & Incorporation of the EIP, of this report is an extension of paragraph 2.9 of the Engineering Appendix.)

(Enclosure 2, Review & Incorporation of the EIP, of this report is an extension of paragraph 2.9 of the Engineering Appendix.)

1.3 Coordination

Existing information and information from the local sponsor's Early Implementation Plan (EIP) were utilized for civil design considerations and evaluations. Close coordination with the local sponsor's design teams took place throughout the study.

CHAPTER 2 – DESIGN CONSIDERATIONS

2.1 General

This chapter provides a summary of the civil design evaluation of and consideration for the draft array of 8 potential alternatives, SB1 to SB8. Design considerations include engineering guidance or methodology used and assumptions.

2.2 Alignment and Stationing

Three levees considered were the FRWL (right levee), SBEL (left levee) and WCEL (left levee).

The project levee alignments and stationing for the SBEL and the WCEL were developed based on the surveyed data from the National Levee Data Base. The stationing for the SBEL begins with station 0+00 at the confluence of the SBEL at the FRWL and increases in an upstream (North) direction. The stationing for the WCEL begins with station 0+00 at the confluence of the WCEL at the SBEL and increases in an upstream (North) direction.

The project levee alignment and stationing for the FRWL, adopted from the 65% EIP, follows the existing levee centerline of the FRWL except at Star Bend where the levee alignment follows the centerline of the setback levee. The stationing begins with station 10+00 at the confluence of the FRWL at the SBEL and increases in an upstream (North) direction. This levee stationing conforms to the existing levee centerline and accounts for recent changes in the alignment, such as the Star Bend Setback Levee (between station 478+68 and station 512+00). At locations where levee relocations (e.g. roughly between station 1432+70 and station 1754+30 etc.) are proposed, supplementary levee alignments stationing necessary for designs and analyses were established.

2.3 Reaches and Alternatives

2.3.1 Reaches

A total of 28 reaches were considered. 16 of these reaches are the existing levee segments (see table 2-1). The other 12 reaches are either proposed setback or new levee segments (see table 2-2). The reaches are shown in figure 1-1.

Table 2-1 – Existing Levee Segments

Reach	Alignment	Type	STA. (Beg.)	STA. (End.)
S5-A-Upper	FRWL	Existing Levee	1958+00	2372+17
S5-A-Lower	FRWL	Existing Levee	1825+00	1958+00
S5-B	FRWL	Existing Levee	1432+00	1825+00
S5-C	FRWL	Existing Levee	1129+00	1432+00
S5-D	FRWL	Existing Levee	816+00	1129+00
S7-D	FRWL	Existing Levee	603+00	816+00
S7-E-Upper	FRWL	Existing Levee	512+00	603+00
S7-E-Middle	FRWL	Existing Levee	479+00	512+00
S7-E-Lower	FRWL	Existing Levee	420+00	479+00
S7-F-Upper	FRWL	Existing Levee	200+00	420+00
S7-F-Middle	FRWL	Existing Levee	47+00	200+00
S7-F-Lower	FRWL	Existing Levee	10+00	47+00
S7-G	SBEL	Existing Levee	0+00	400+00
S7-H	SBEL	Existing Levee	400+00	493+00
S7-I	SBEL	Existing Levee	493+00	922+16
S7-J	WCEL	Existing Levee	0+00	244+00

Table 2-2 – New Levee Segments

Reach	Alignment	Type	STA. (Beg.)	STA. (End.)
S4-South	YCRL	New Ring Levee	0+00	280+00
S4-West	YCRL	New Ring Levee	280+00	490+00
S4-North	YCRL	New Ring Levee	490+00	750+00
S6-South	YCJL	New “J” Levee	0+00	280+00
S6-West-lower	YCJL	New “J” Levee	280+00	490+00
S6-West-upper	YCJL	New “J” Levee	490+00	550+00
S9-G	SBEL	Setback Levee	0+00	400+00
S9-H	SBEL	Setback Levee	400+00	493+00
S9-I	SBEL	Setback Levee	493+00	922+16
S10	FRWL	Setback Levee	1958+00	2372+17
S11	FRWL	Setback Levee	47+00	200+00
S12	FRWL	Setback Levee	479+00	512+00

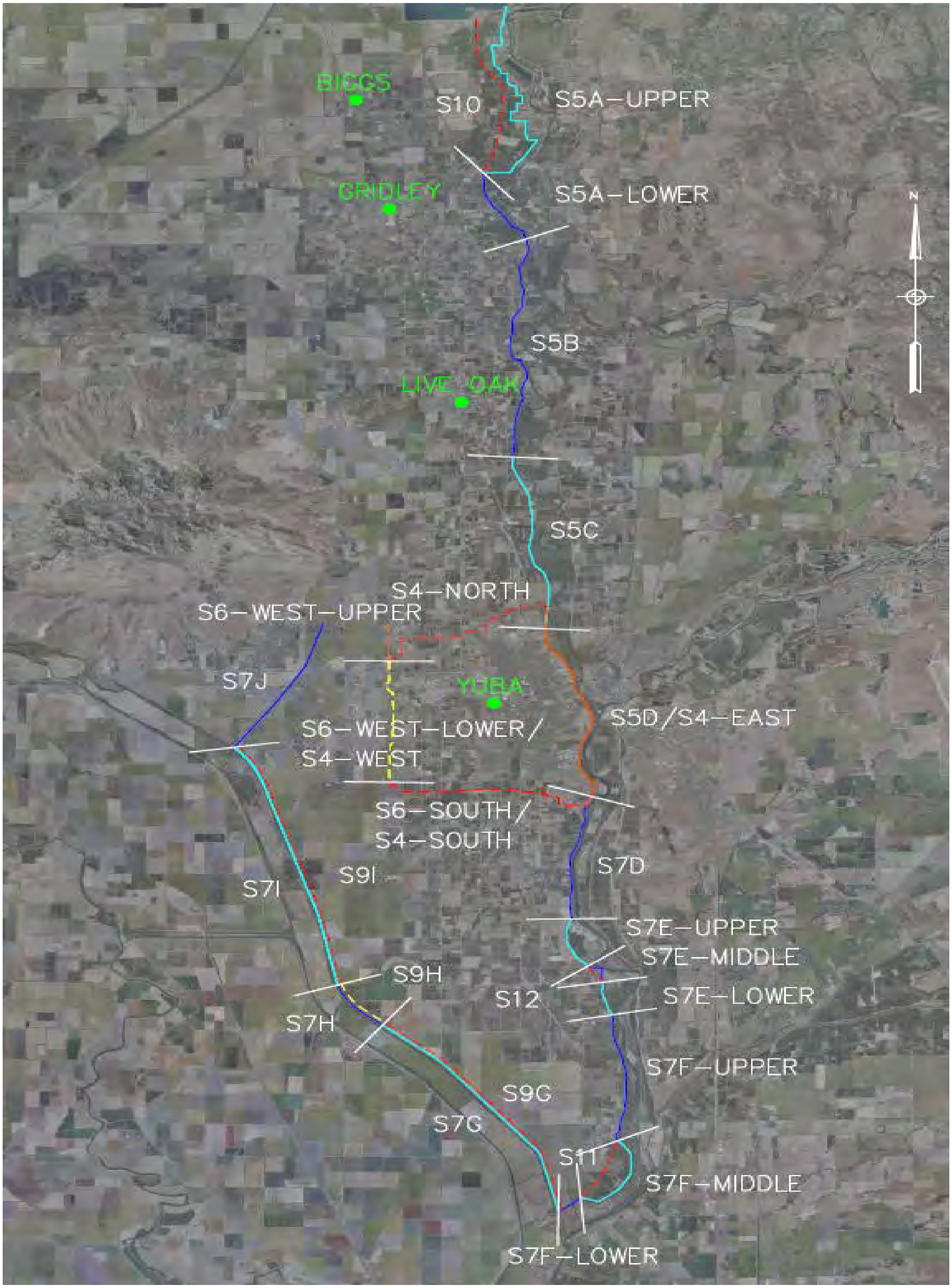


Figure 1-1 – Map of Reaches

2.3.2 Alternatives

Through plan formulation eight potential alternatives were retained from the preliminary array for further evaluation, these include:

- SB-1: No Action.
- SB-2: Minimal Fix-in-place the FRWL from Star Bend to Sunset Weir
- SB-3: Yuba City Ring Levee
- SB-4: Little "J" Levee
- SB-5: Fix-in-place the FRWL from Star Bend to Thermalito Afterbay
- SB-6: Fix-in-Place the FRWL, SBEL and WCEL
- SB-7: Fix-in-Place the FRWL from Laurel Avenue to Sunset Weir
- SB-8: Fix-in-Place the FRWL from Laurel Avenue to Thermalito Afterbay.

Table 2-3 summarizes the reaches included in each of the 8 potential alternatives.

Table 2-3 – Draft Array of Potential Alternatives

Reach	SB-1	SB-2	SB-3	SB-4	SB-5	SB-6	SB-7	SB-8
S5-A-Upper				X	X	X		X
S5-A-Lower				X	X	X		X
S5-B				X	X	X		X
S5-C		X		X	X	X	X	X
S5-D		X	X	X	X	X	X	X
S7-D		X			X	X	X	X
S7-E-Upper		X			X	X	X	X
S7-E-Middle		X			X	X	X	X
S7-E-Lower						X	X	X
S7-F-Upper						X	X	X
S7-F-Middle						X		
S7-F-Lower						X		
S7-G						X		
S7-H						X		
S7-I						X		
S7-J						X		
S4-South			X					
S4-West			X					
S4-North			X					
S6-South				X				
S6-West-lower				X				
S6-West-upper				X				
S9-G								
S9-H								
S9-I								
S10								
S11								
S12								

2.4 Existing Condition and Remediation Measures

2.4.1 Existing Condition

Based on the result of preliminary geotechnical investigations, the average geometry of the existing levees were defined and shown in table 2-4.

Table 2-4 – Average Geometry of Existing Levee Segments

Reach	Length (LF)	Height (LF)	Crest Width (LF)	LS Slope (H:V)	WS Slope (H:V)	Base Width (LF)
S5-A-Upper	41,417	17.5	20	2:1	3:1	107.5
S5-A-Lower	13,300	17.5	20	2:1	3:1	107.5
S5-B	39,300	12.5	20	2:1	3:1	82.5
S5-C	30,300	17.5	16	2:1	3:1	103.5
S5-D	31,300	25	15	2:1	3:1	140
S7-D	21,300	25	15	2:1	3:1	140
S7-E-Upper	9,100	22.5	17	2:1	3:1	127.5
S7-E-Middle	3,300	22.5	17	2:1	3:1	127.5
S7-E-Lower	5,900	22.5	17	2:1	3:1	127.5
S7-F-Upper	22,000	22.5	13	2:1	3:1	125.5
S7-F-Middle	15,300	22.5	13	2:1	3:1	125.5
S7-F-Lower	3,700	22.5	13	2:1	3:1	125.5
S7-G	40,000	22.5	22	2:1	3:1	134.5
S7-H	9,300	20	22	2:1	3:1	122
S7-I	42,916	20	22	2:1	3:1	122
S7-J	24,400	15	24	2:1	3:1	99

2.4.2 Proposed Levee Remediation Measures

Based on preliminary geotechnical design recommendations, 9 conceptual typical levee remediation measures were developed and shown in figure 2-1 through 2-9.

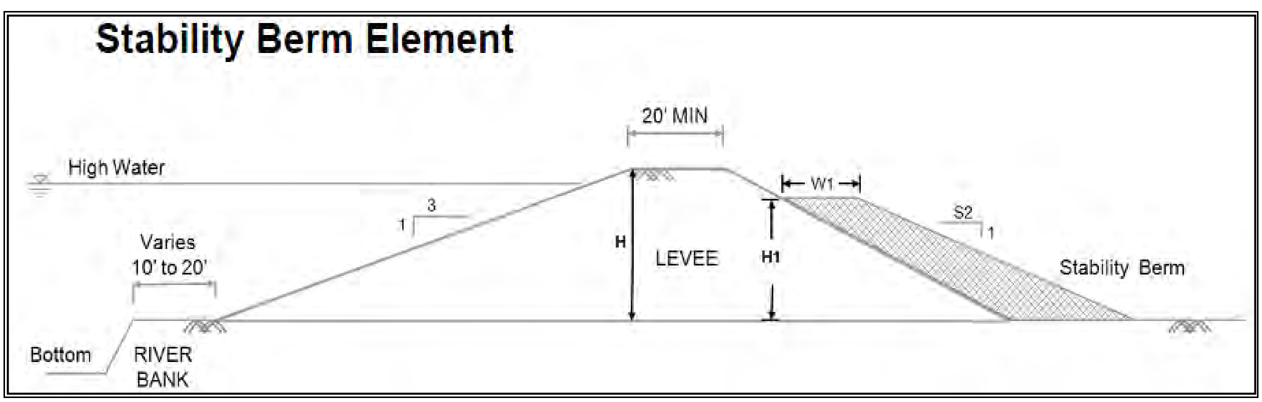


Figure 2-1 – Levee Improvement Type 1

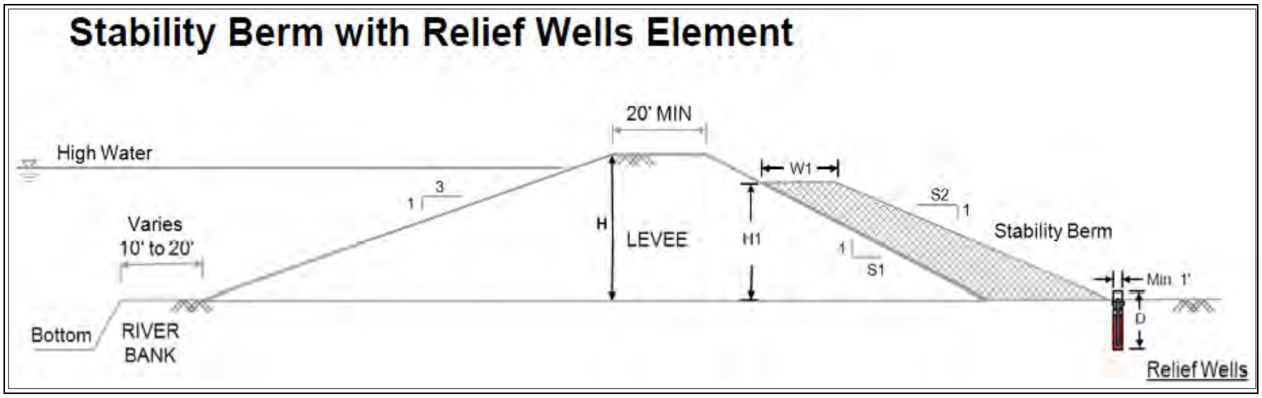


Figure 2-2 – Levee Improvement Type 2

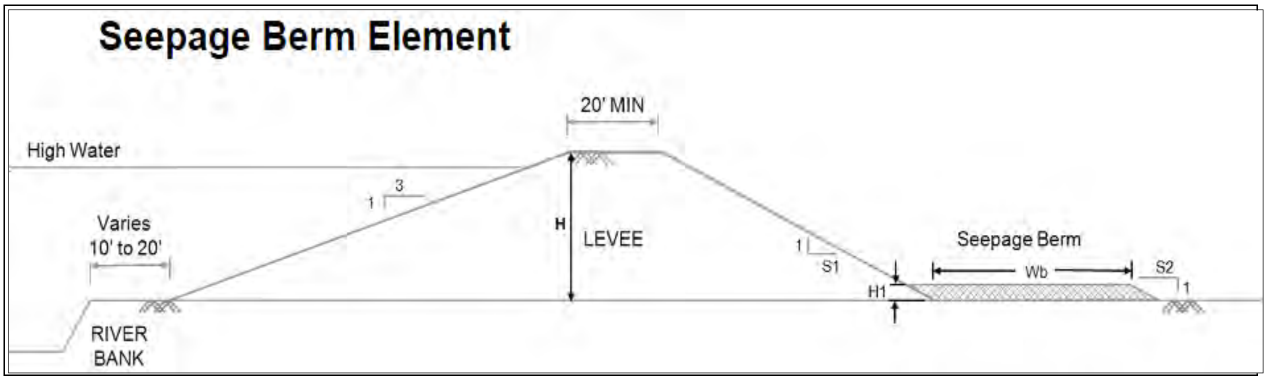


Figure 2-3 – Levee Improvement Type 3

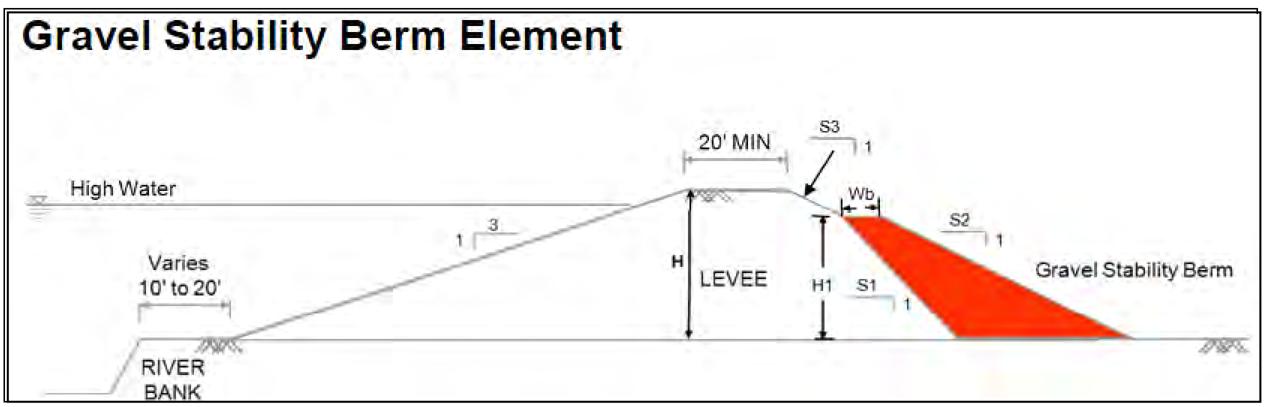


Figure 2-4 – Levee Improvement Type 4

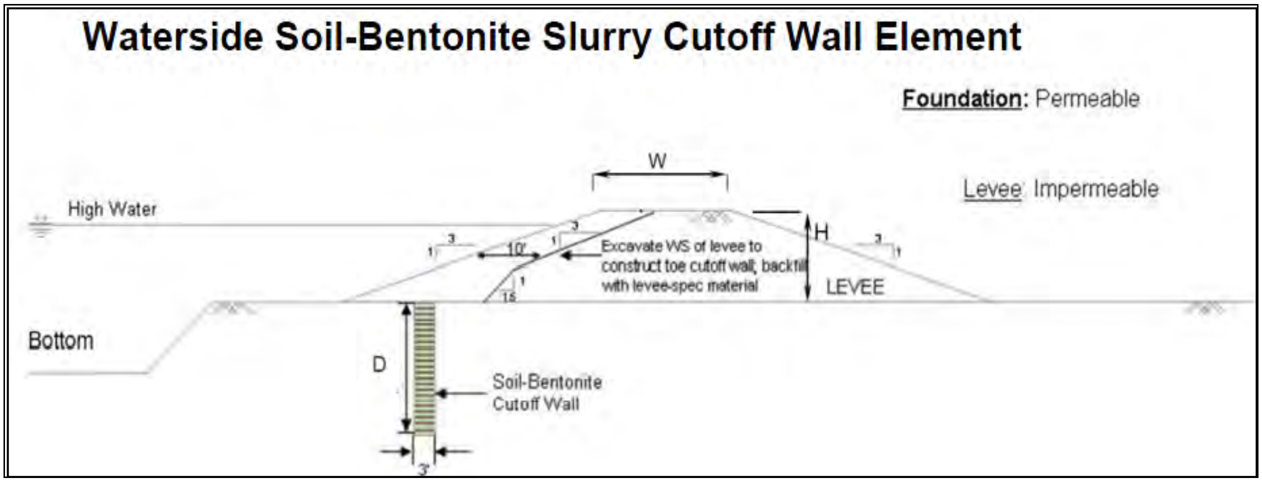


Figure 2-5 – Levee Improvement Type 5

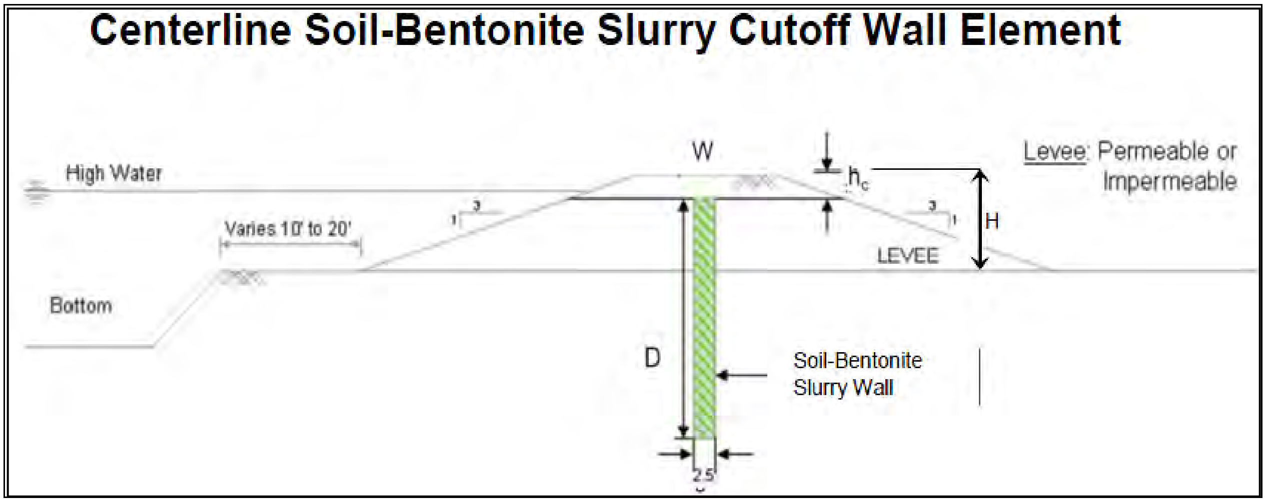


Figure 2-6 – Levee Improvement Type 6

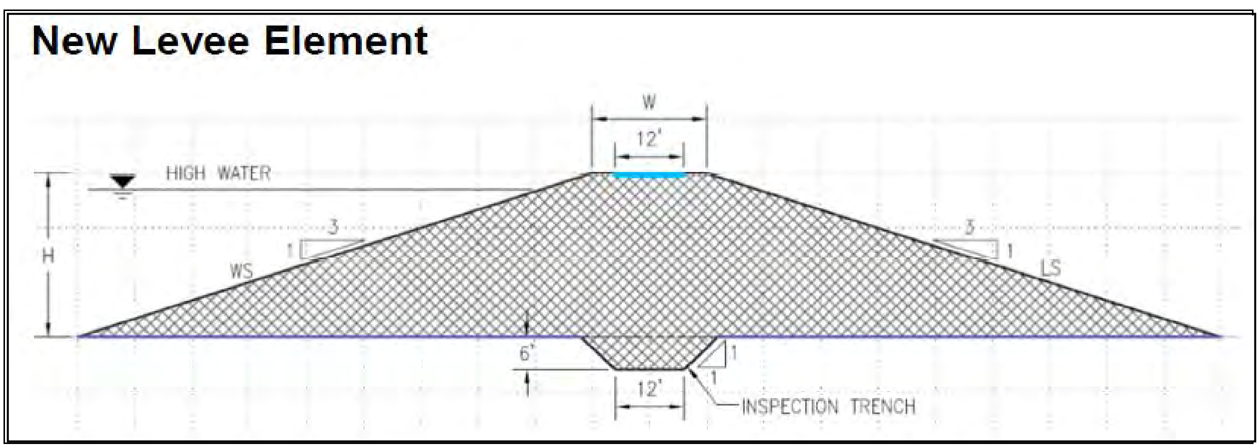


Figure 2-7 – Levee Improvement Type 7

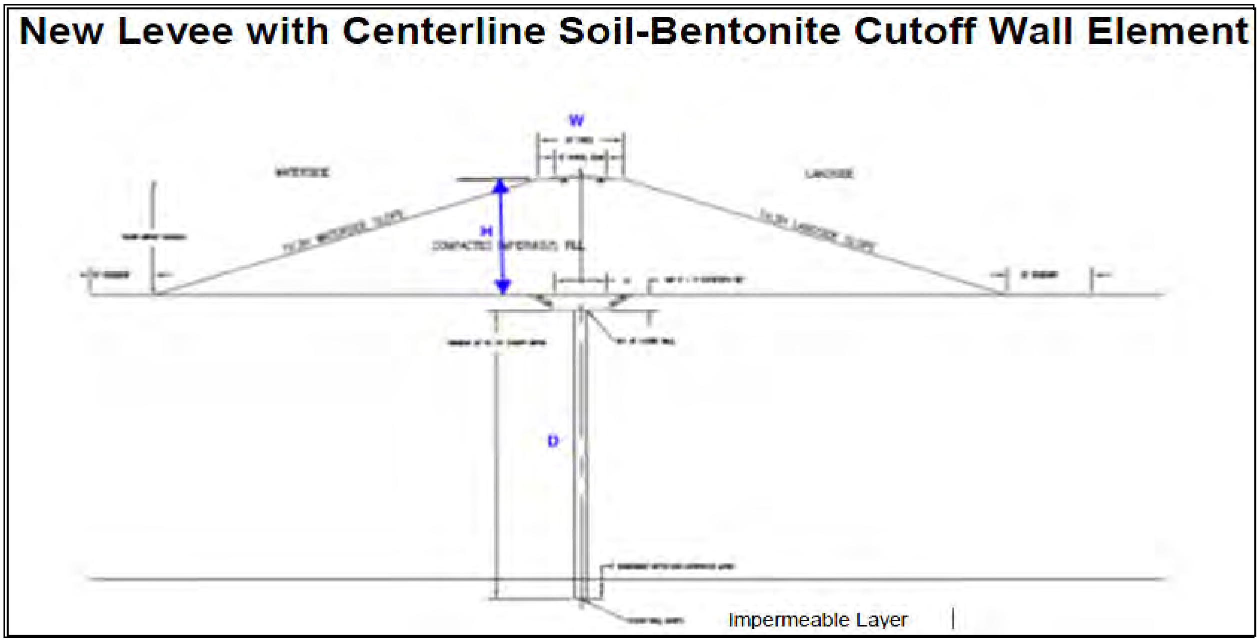


Figure 2-8 – Levee Improvement Type 8

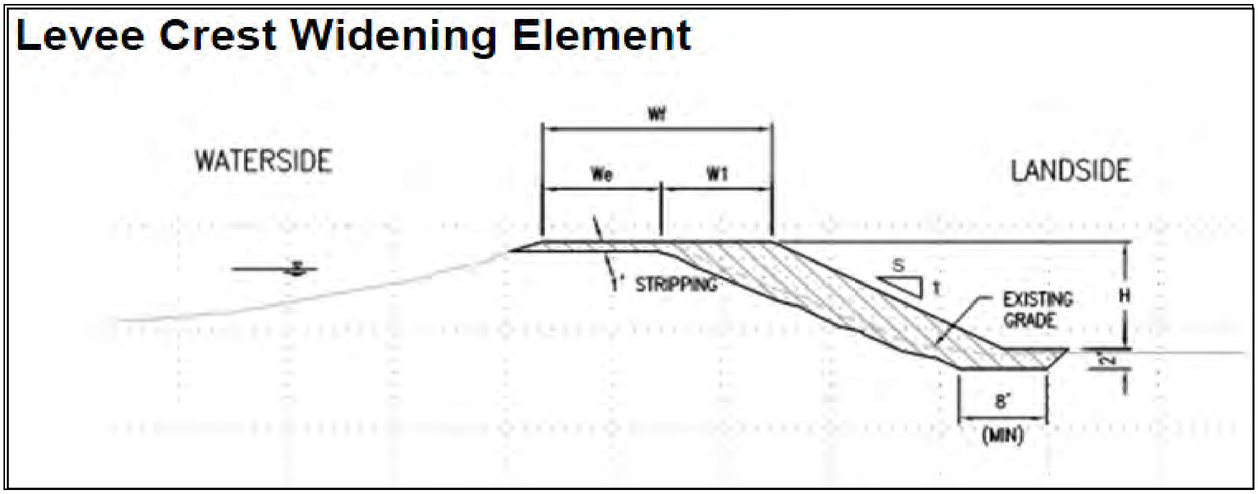


Figure 2-9 – Levee Improvement Type 9

The typical levee remediation measures (shown in figure 2-1 through 2-9) were assigned to each of the 28 reaches as shown in table 2-5A and 2-5B:

Table 2-5A – Levee Remediation Measures (by Percentage of Reach Length)

Reach	Length (LF)	Type 1	Type 2	Type 3	Type 4	Type 5	Type 6	Type 7	Type 8	Type 9
S5-A-Upper	41,417				25%		100%			
S5-A-Lower	13,300				25%		100%			
S5-B	39,300				75%		75%			
S5-C	30,300				25%		75%			100%
S5-D	31,300				10%		50%			100%
S7-D	21,300				10%		50%			100%
S7-E-Upper	9,100				10%		75%			100%

S7-E-Middle	3,300				10%		75%			100%
S7-E-Lower	5,900				10%		75%			100%
S7-F-Upper	22,000				10%		75%			100%
S7-F-Middle	15,300				10%		75%			100%
S7-F-Lower	3,700				10%		75%			100%
S7-G	40,000				10%		100%			
S7-H	9,300				10%		100%			
S7-I	42,916				10%		100%			
S7-J	24,400				10%		50%			
S4-South	28,000							50%	50%	
S4-West	21,000							75%	25%	
S4-North	26,000							50%	50%	
S6-South	28,000							50%	50%	
S6-West-lower	21,000							75%	25%	
S6-West-upper	6,000							75%	25%	
S9-G	40,000								100%	
S9-H	9,300								100%	
S9-I	42,916								100%	
S10	41,417								100%	
S11	15,300							50%	50%	
S12	3,300						25%		75%	

Table 2-5B – Levee Remediation Measures (by Length in Linear Feet)

Reach	Length (LF)	Type 1	Type 2	Type 3	Type 4	Type 5	Type 6	Type 7	Type 8	Type 9
S5-A-Upper	41,417				10,354		41,417			
S5-A-Lower	13,300				3,325		13,300			
S5-B	39,300				29,475		29,475			
S5-C	30,300				7,575		22,725			30,300
S5-D	31,300				3,130		15,650			31,300
S7-D	21,300				2,130		10,650			21,300
S7-E-Upper	9,100				910		6,825			9,100
S7-E-Middle	3,300				330		2,475			3,300
S7-E-Lower	5,900				590		4,425			5,900
S7-F-Upper	22,000				2,200		16,500			22,000
S7-F-Middle	15,300				1,530		11,475			15,300
S7-F-Lower	3,700				370		2,775			3,700
S7-G	40,000				4,000		40,000			
S7-H	9,300				930		9,300			
S7-I	42,916				4,292		42,916			
S7-J	24,400				2,440		12,200			
S4-South	28,000							14,000	14,000	
S4-West	21,000							15,750	5,250	
S4-North	26,000							13,000	13,000	
S6-South	28,000							14,000	14,000	
S6-West-lower	21,000							15,750	5,250	
S6-West-upper	6,000							4,500	1,500	
S9-G	40,000								40,000	

S9-H	9,300								9,300	
S9-I	42,916								42,916	
S10	41,417								41,417	
S11	15,300							7,650	7,650	
S12	3,300						825		2,475	

Assignment (dimension and extent) of the remediation measures (figure 2-1 to 2-9) for each reach are graphically presented in figure 2-10A through 2-29B. Also shown in these figure are the 20-foot landside and 15-foot waterside O&M corridors. The outer most limits of the O&M corridors define the project ROW. The heights of new levee segments (shown in figure 2-21A to 2-23B for Ring and J levee segments defined in table 2-2) were based on hydraulic design recommendations (enclosure 3, Design of New Levee Segments).

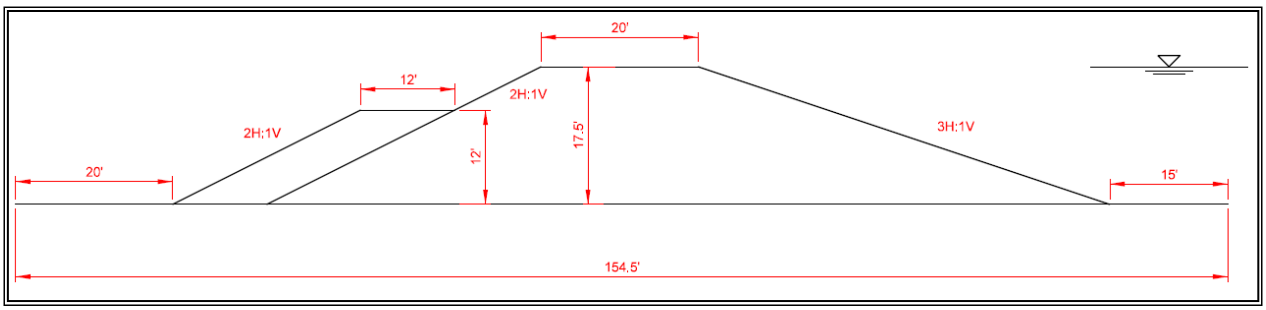


Figure 2-10A – Reach S5-A (Improvement Type 4 for 25% of Reach Length)

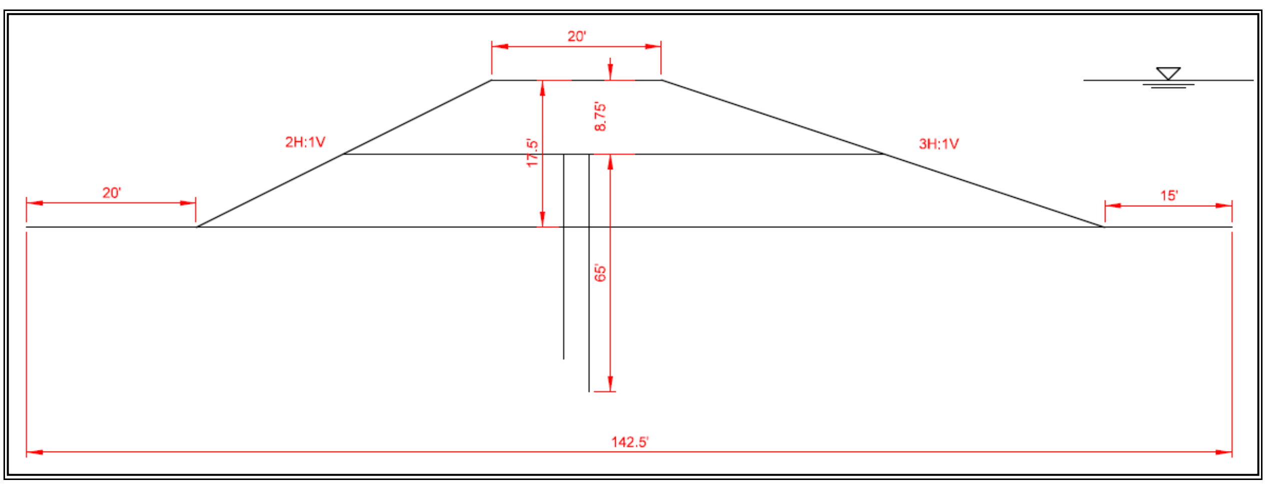


Figure 2-10B – Reach S5-A (Improvement Type 6 for 100% of Reach Length)

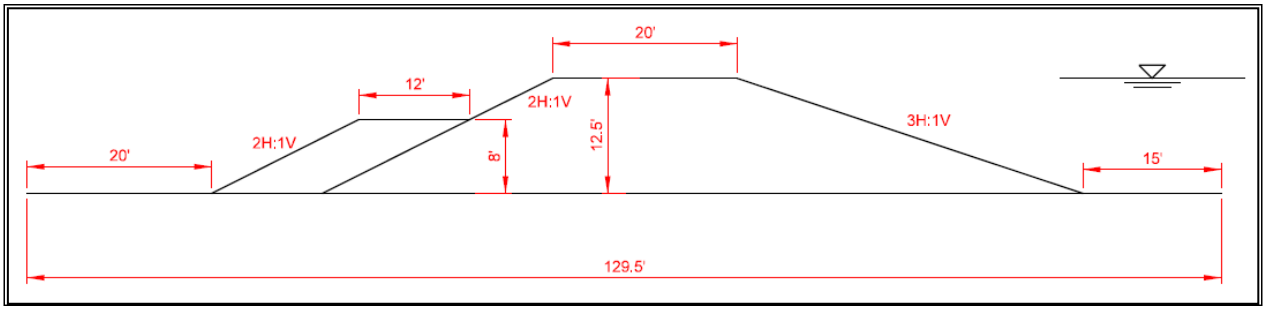


Figure 2-11A – Reach S5-B (Improvement Type 4 for 75% of Reach Length)

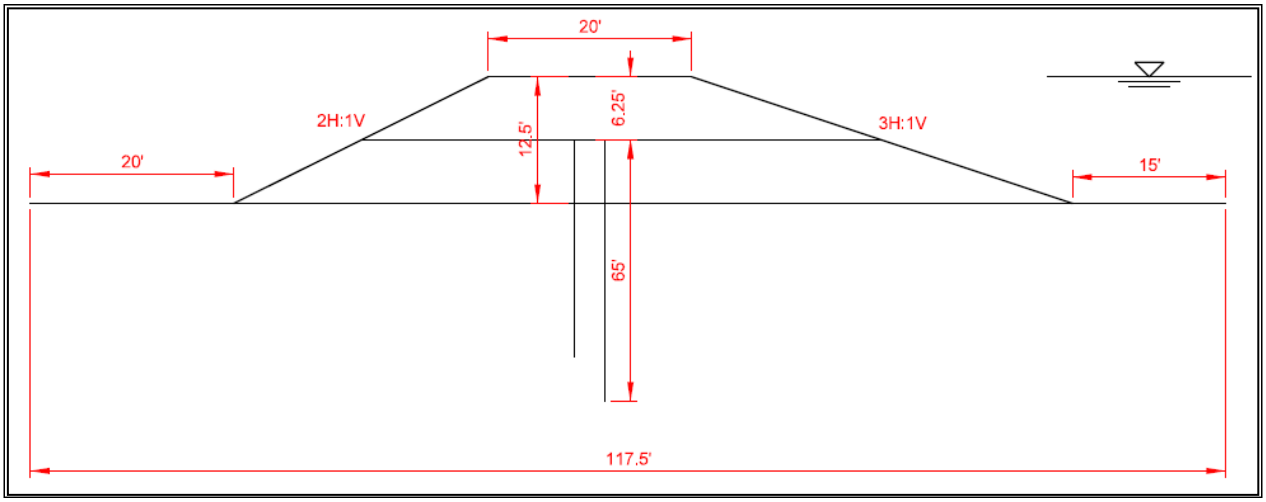


Figure 2-11B – Reach S5-B (Improvement Type 6 for 75% of Reach Length)

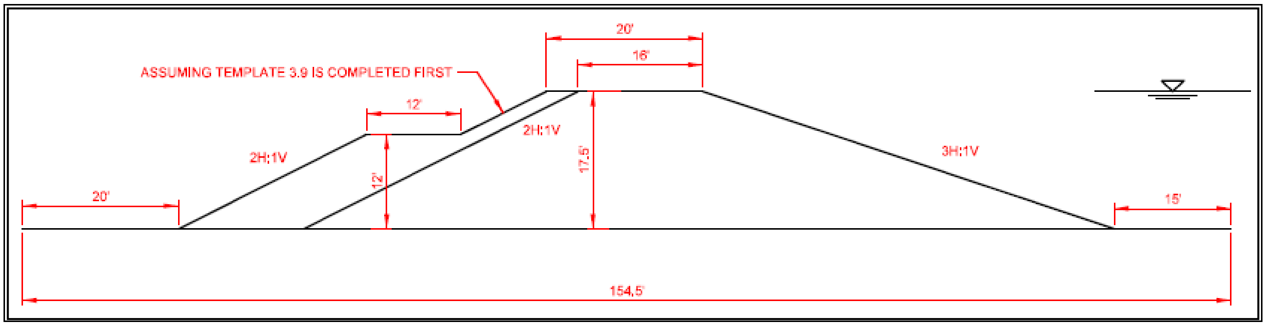


Figure 2-12A – Reach S5-C (Improvement Type 4 for 25% of Reach Length)

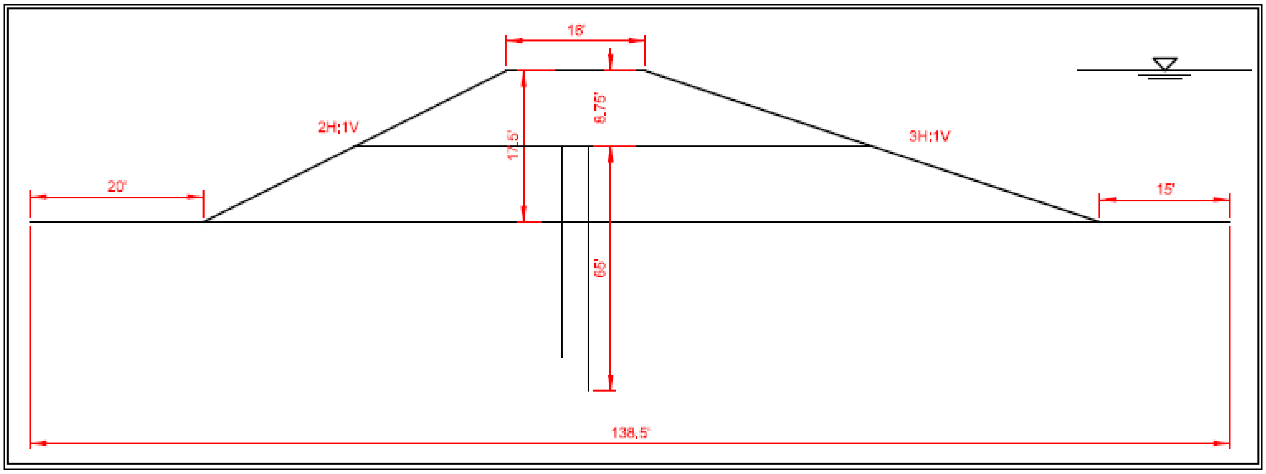


Figure 2-12B – Reach S5-C (Improvement Type 6 for 75% of Reach Length)

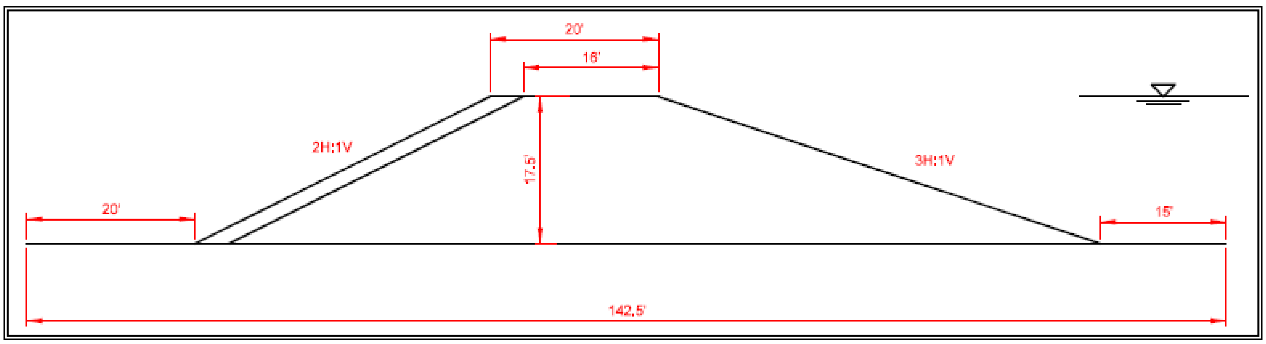


Figure 2-12C – Reach S5-C (Improvement Type 9 for 100% of Reach Length)

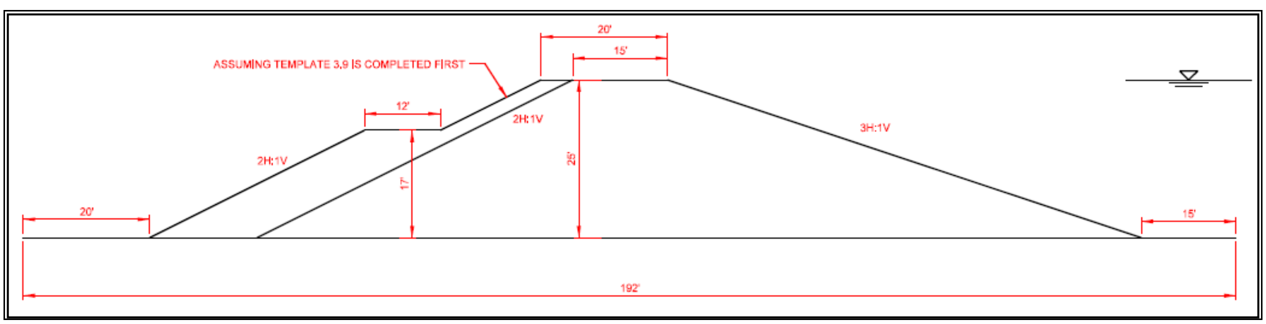


Figure 2-13A – Reach S5-D (Improvement Type 4 for 10% of Reach Length)

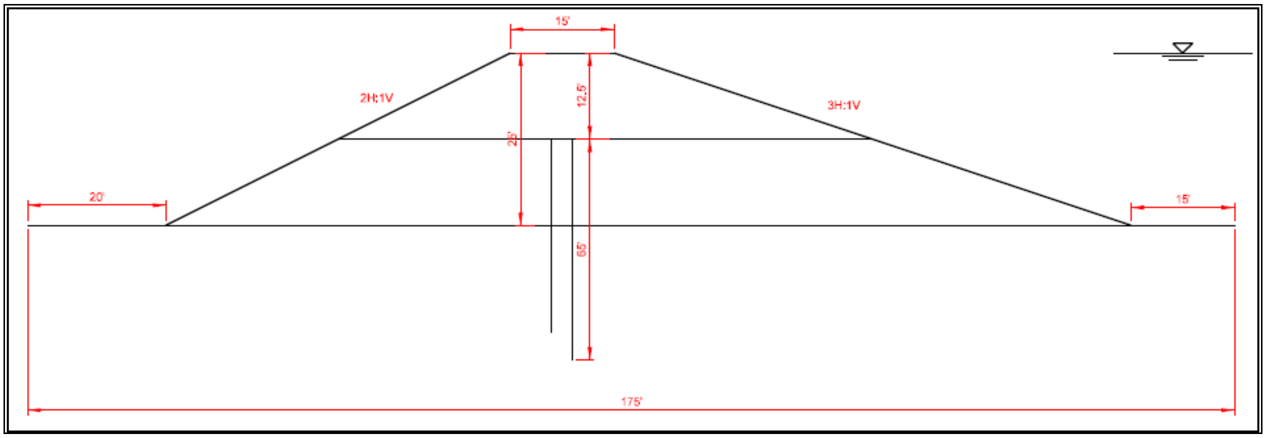


Figure 2-13B – Reach S5-D (Improvement Type 6 for 50% of Reach Length)

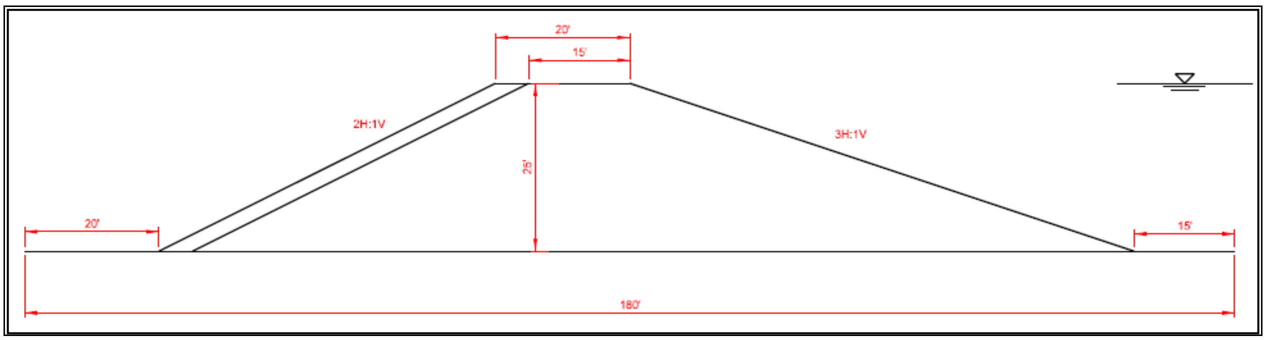


Figure 2-13C – Reach S5-D (Improvement Type 9 for 100% of Reach Length)

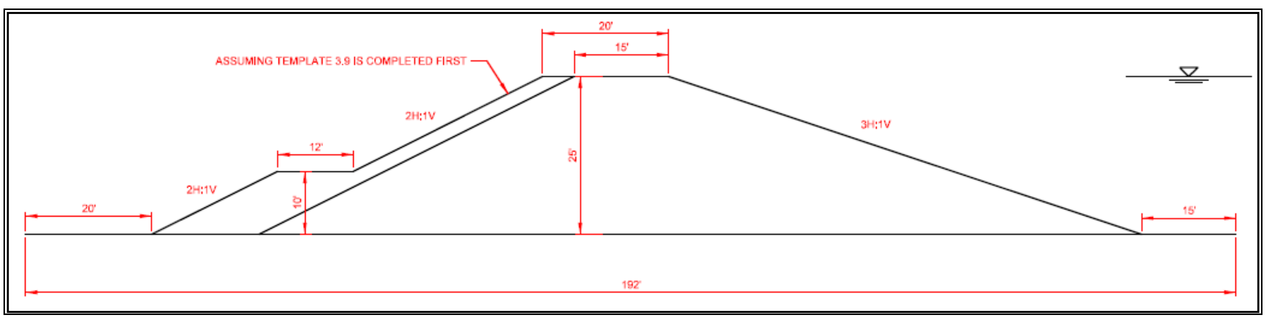


Figure 2-14A – Reach S7-D (Improvement Type 4 for 10% of Reach Length)

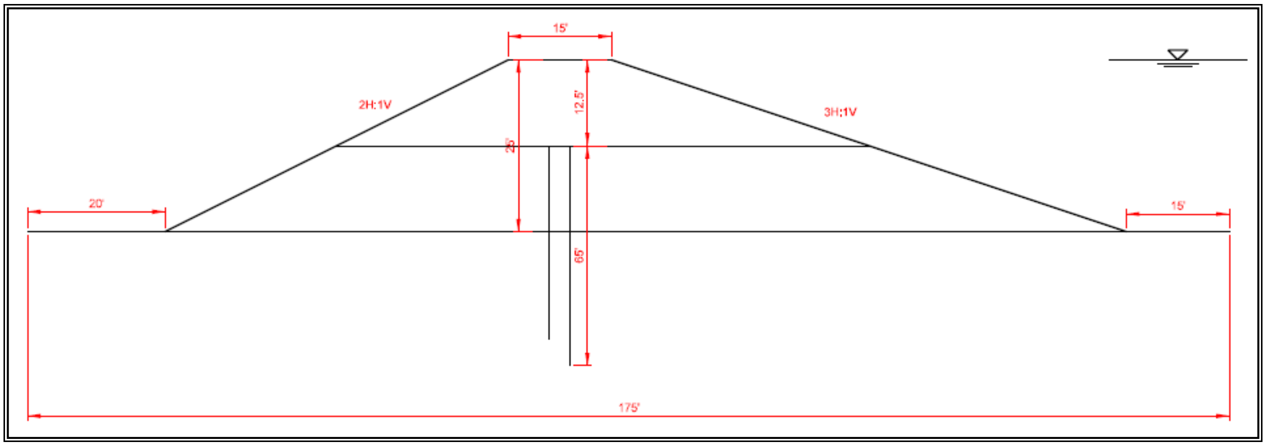


Figure 2-14B – Reach S7-D (Improvement Type 6 for 50% of Reach Length)

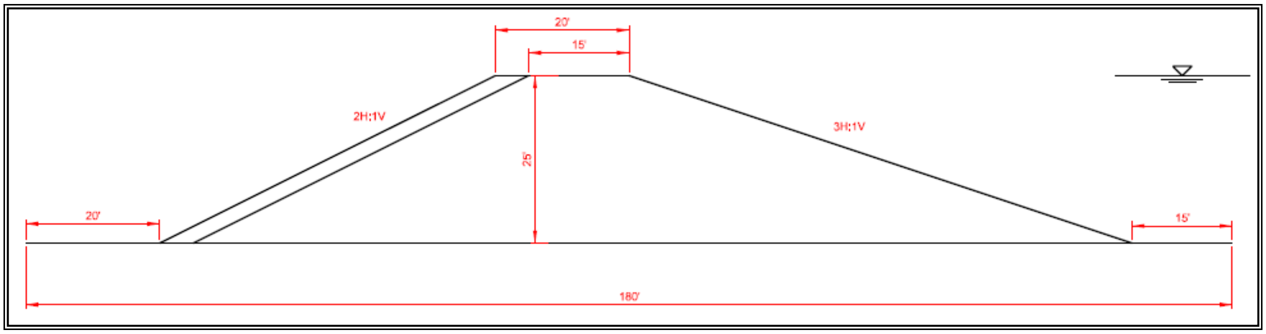


Figure 2-14C – Reach S7-D (Improvement Type 9 for 100% of Reach Length)

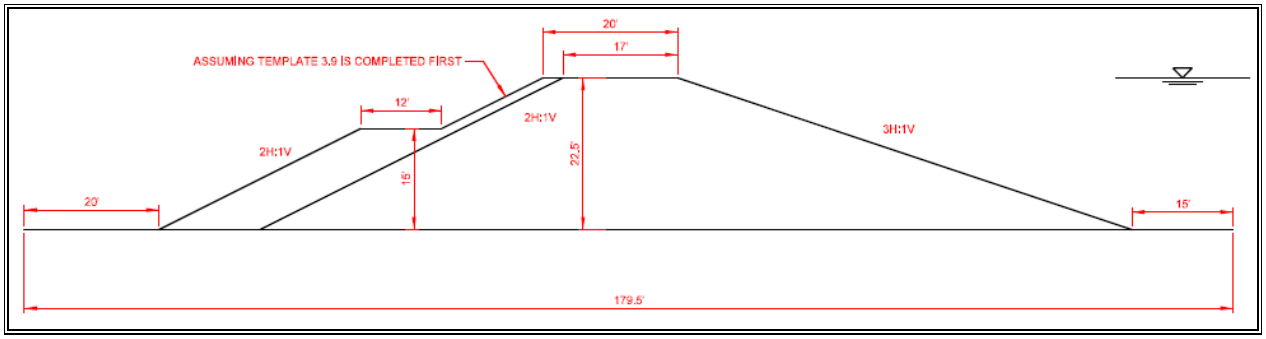


Figure 2-15A – Reach S7-E (Improvement Type 4 for 10% of Reach Length)

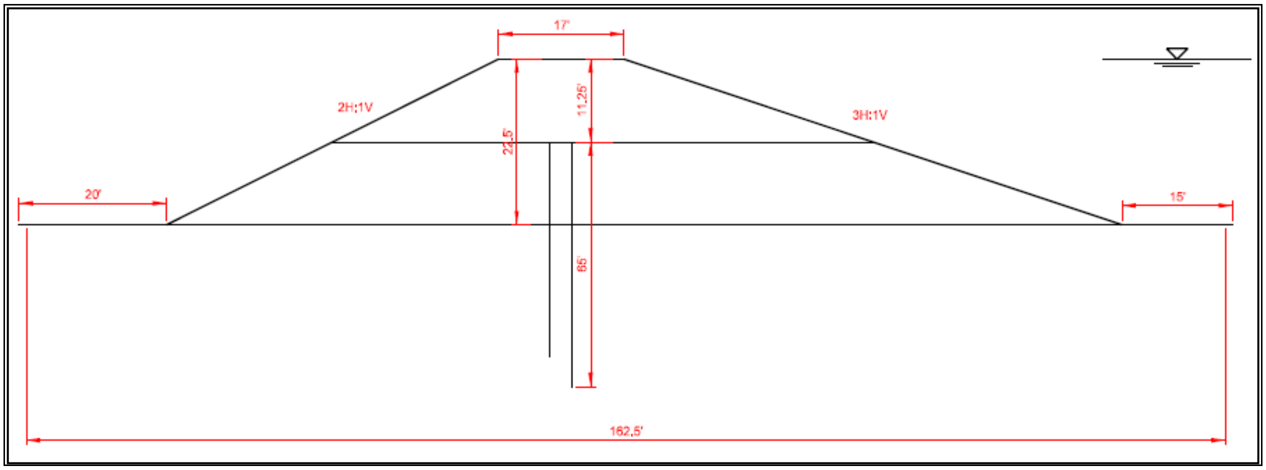


Figure 2-15B – Reach S7-E (Improvement Type 6 for 75% of Reach Length)

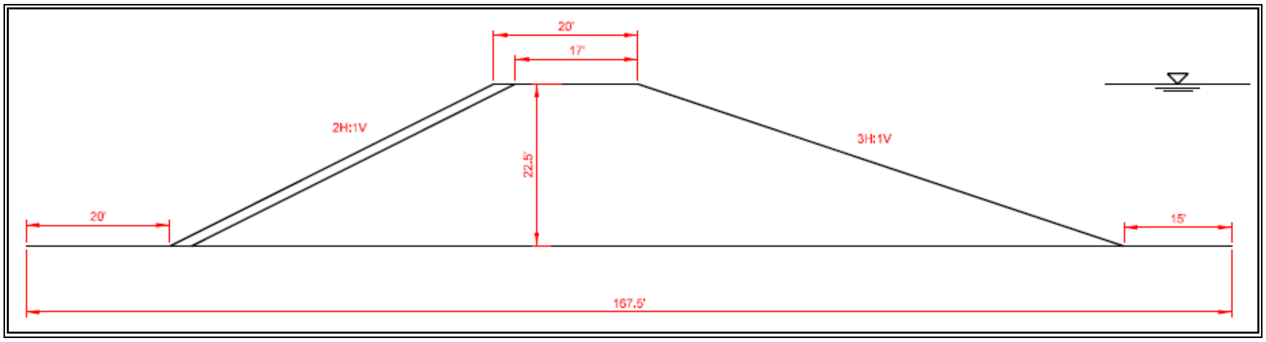


Figure 2-15C – Reach S7-E (Improvement Type 9 for 100% of Reach Length)

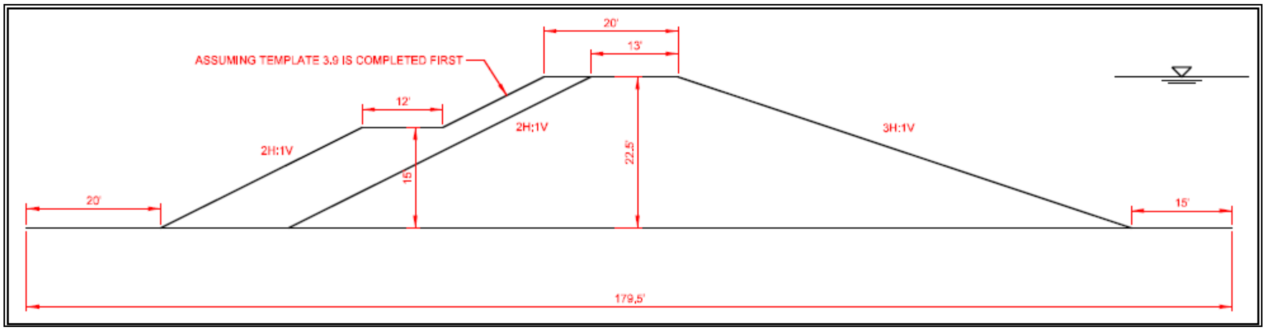


Figure 2-16A – Reach S7-F (Improvement Type 4 for 10% of Reach Length)

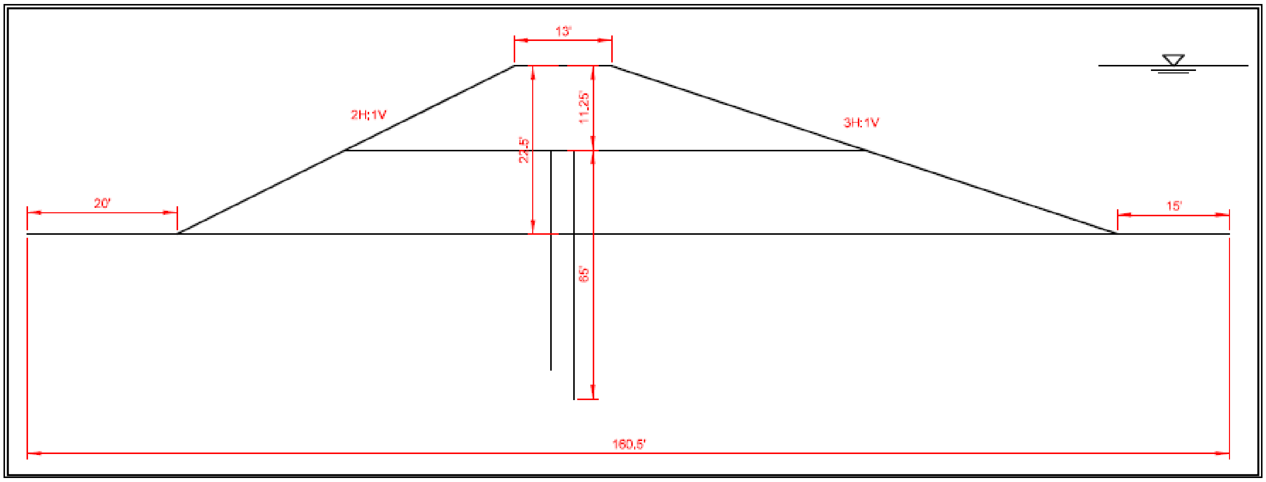


Figure 2-16B – Reach S7-F (Improvement Type 6 for 75% of Reach Length)

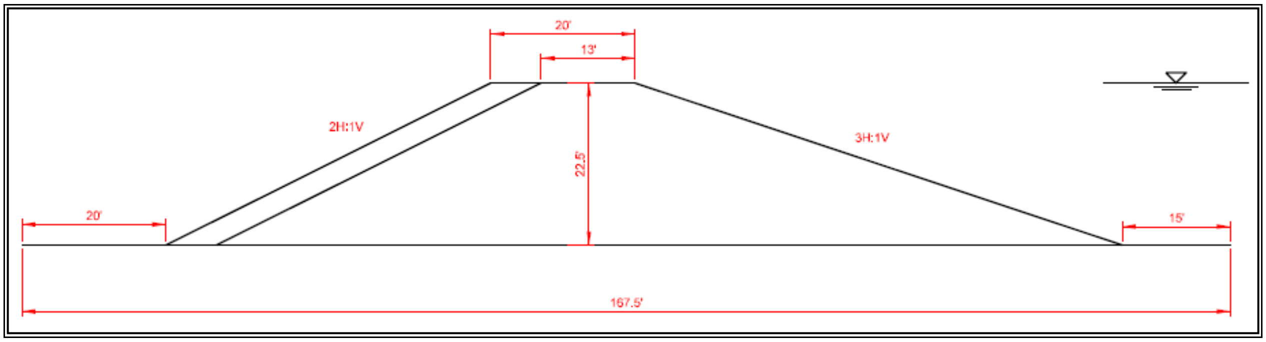


Figure 2-16C – Reach S7-F (Improvement Type 9 for 100% of Reach Length)

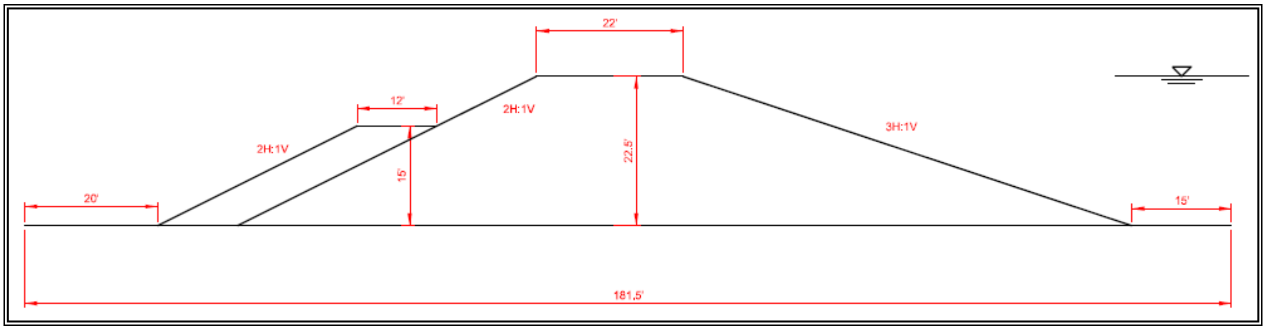


Figure 2-17A – Reach S7-G (Improvement Type 4 for 10% of Reach Length)

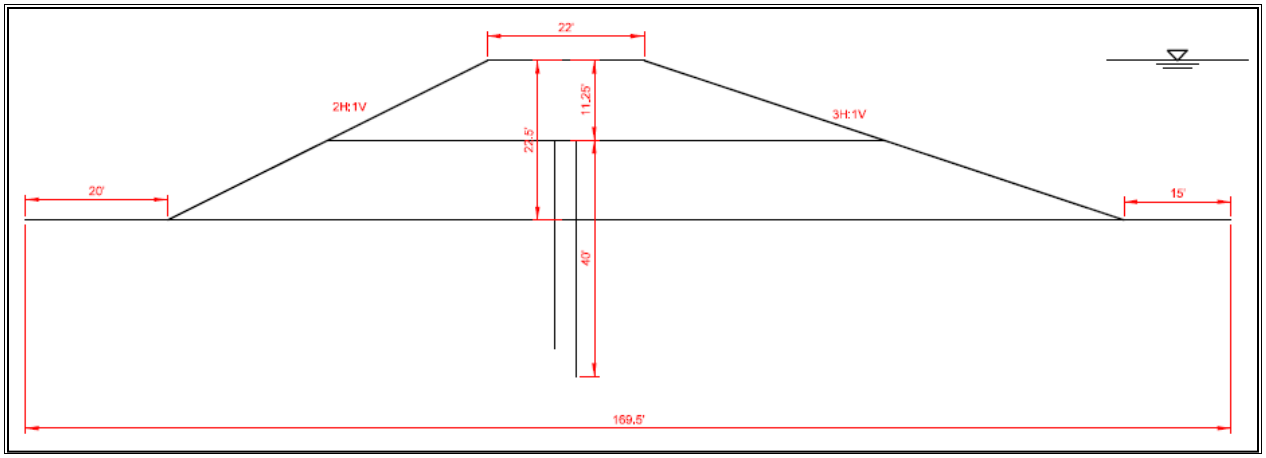


Figure 2-17B – Reach S7-G (Improvement Type 6 for 100% of Reach Length)

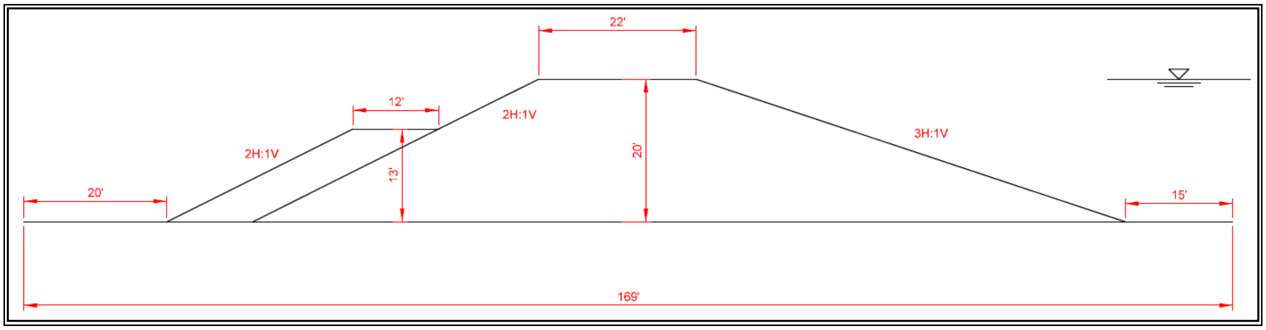


Figure 2-18A – Reach S7-H (Improvement Type 4 for 10% of Reach Length)

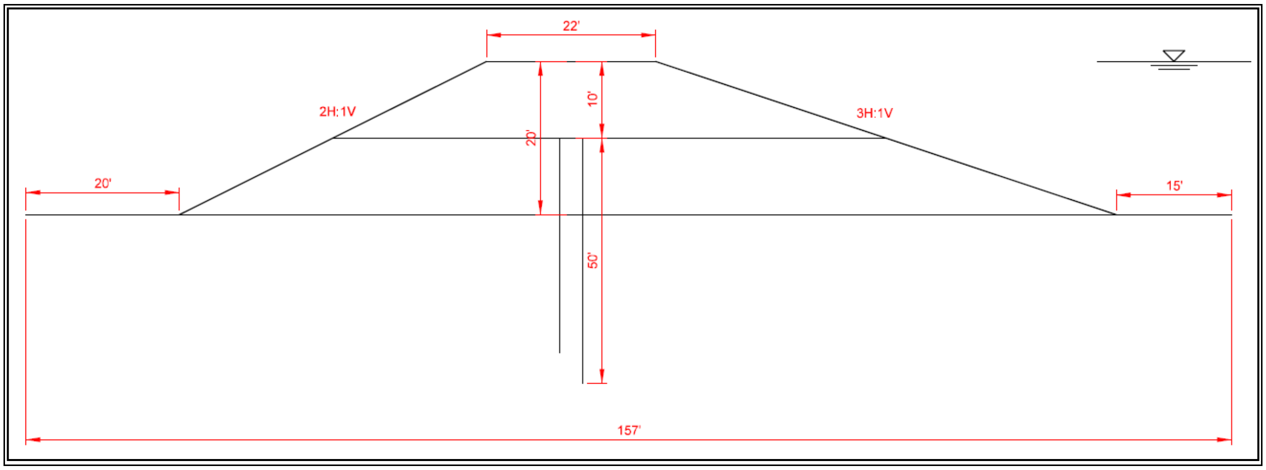


Figure 2-18B – Reach S7-H (Improvement Type 6 for 100% of Reach Length)

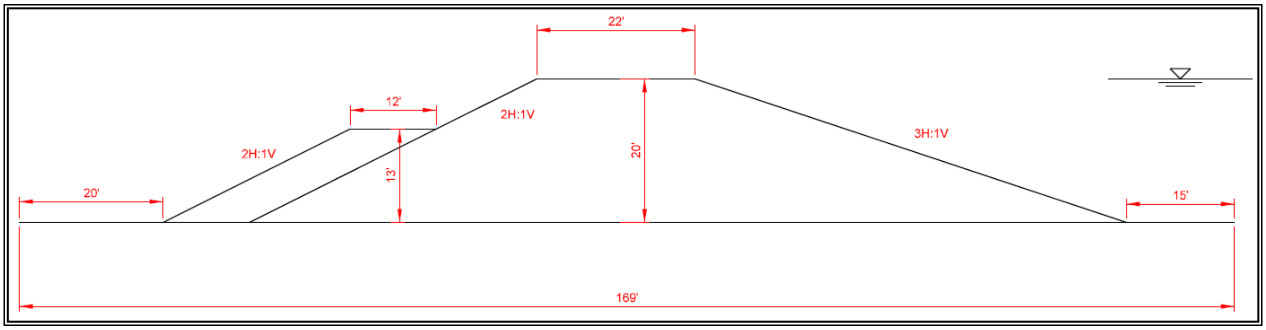


Figure 2-19A – Reach S7-I (Improvement Type 4 for 10% of Reach Length)

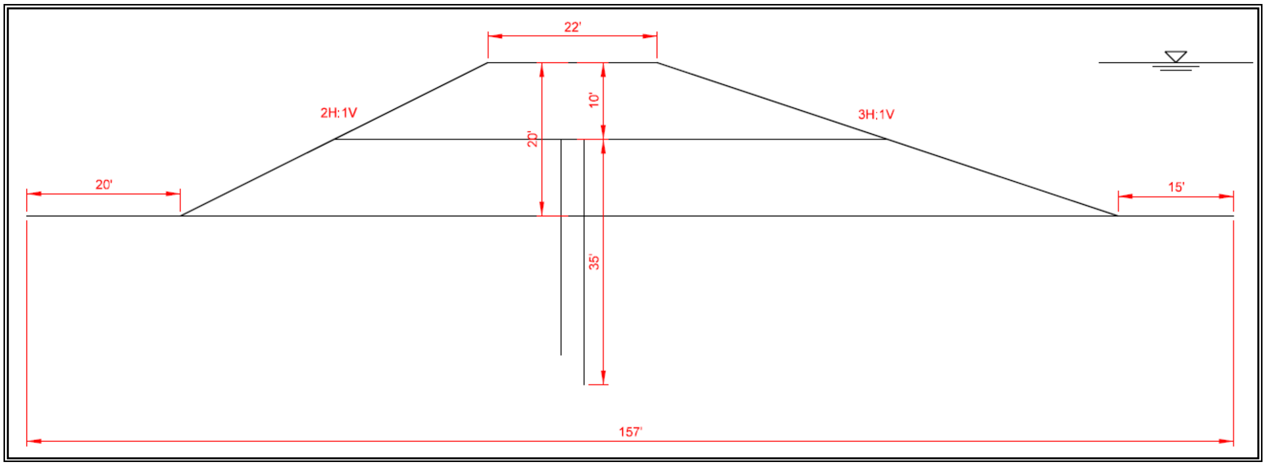


Figure 2-19B – Reach S7-I (Improvement Type 6 for 100% of Reach Length)

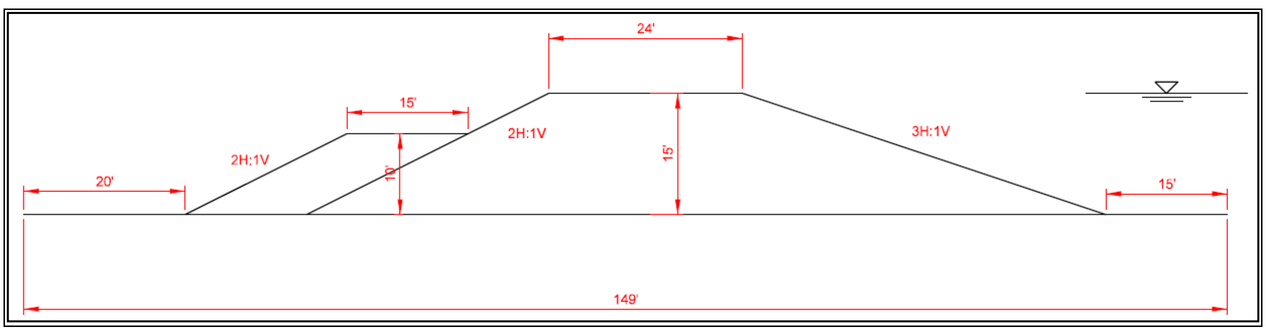


Figure 2-20A – Reach S7-J (Improvement Type 4 for 10% of Reach Length)

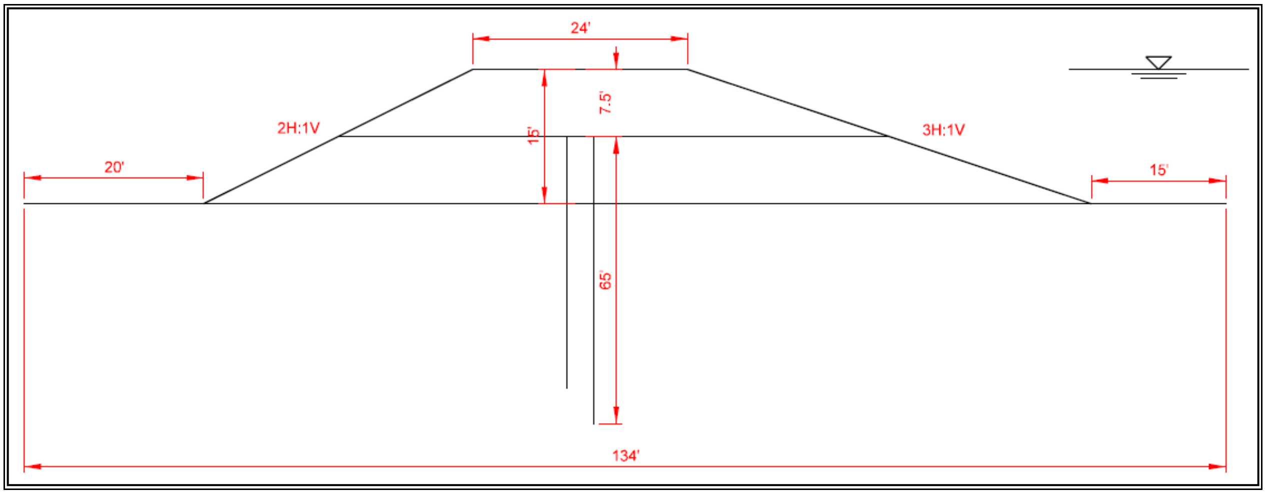


Figure 2-20B – Reach S7-J (Improvement Type 6 for 50% of Reach Length)

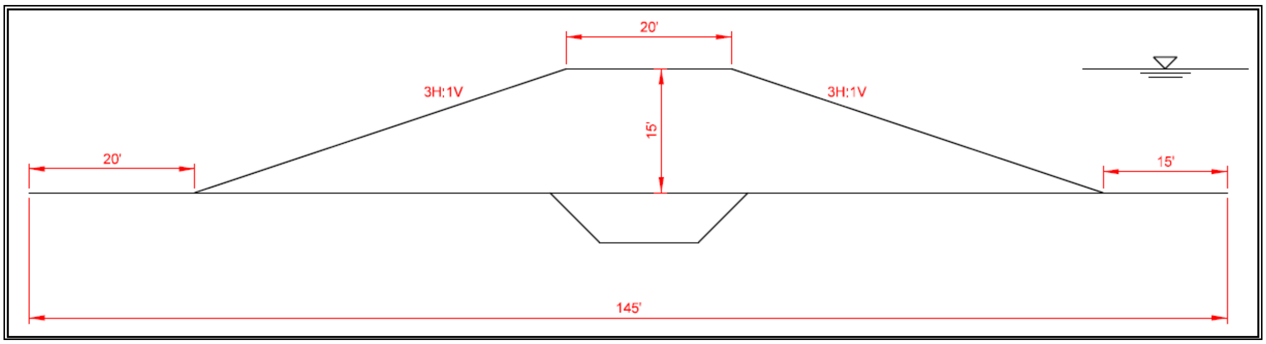


Figure 2-21A – Reach S4-South/S6-South (Improvement Type 7 for 50% of Reach Length)

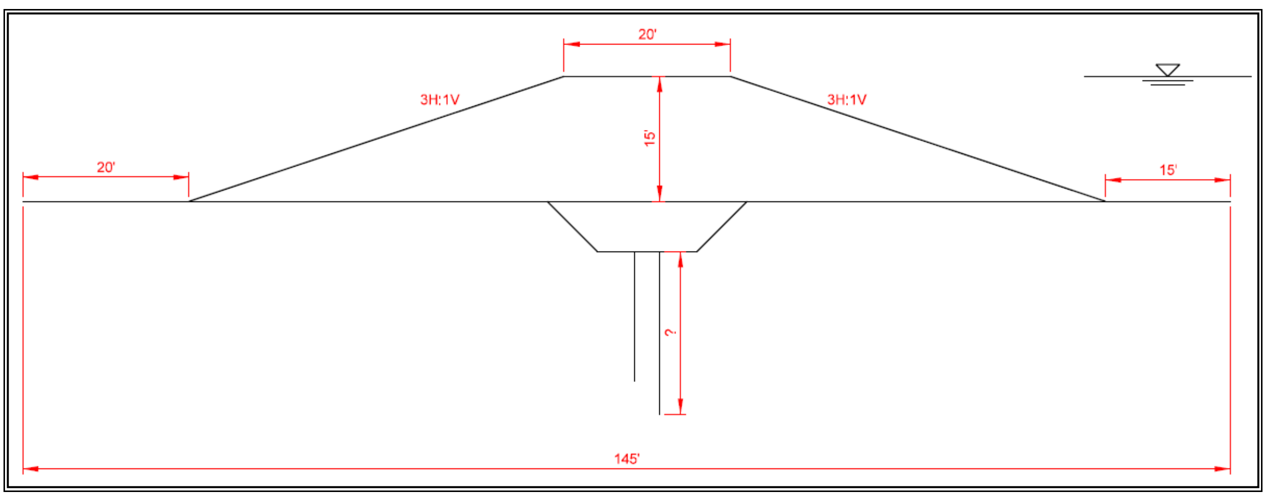


Figure 2-21B – Reach S4-South/S6-South (Improvement Type 8 for 50% of Reach Length)

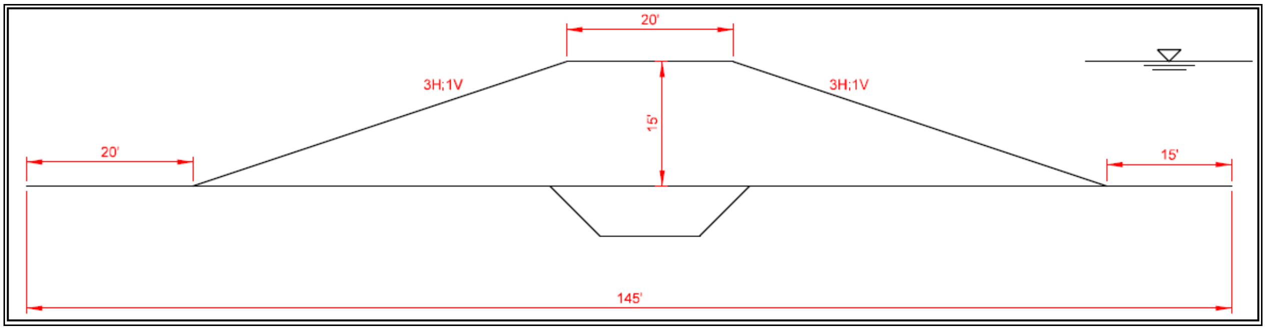


Figure 2-22A – Reach S4-West/S6-West (Improvement Type 7 for 75% of Reach Length)

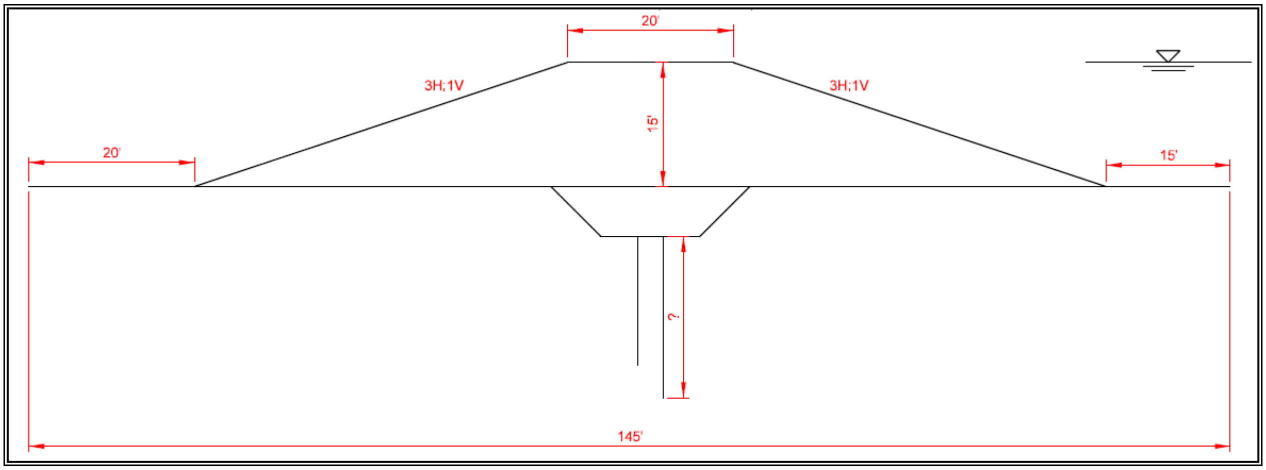


Figure 2-22B – Reach S4-West/S6-West (Improvement Type 8 for 25% of Reach Length)

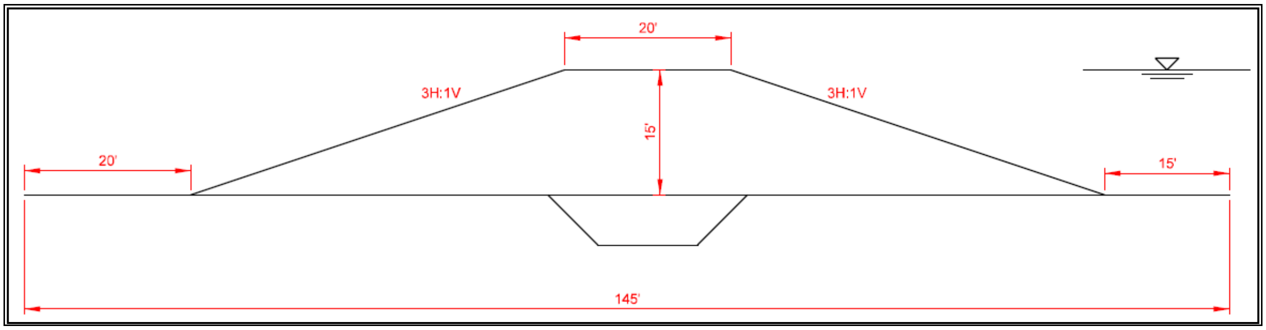


Figure 2-23A – Reach S4-North (Improvement Type 7 for 50% of Reach Length)

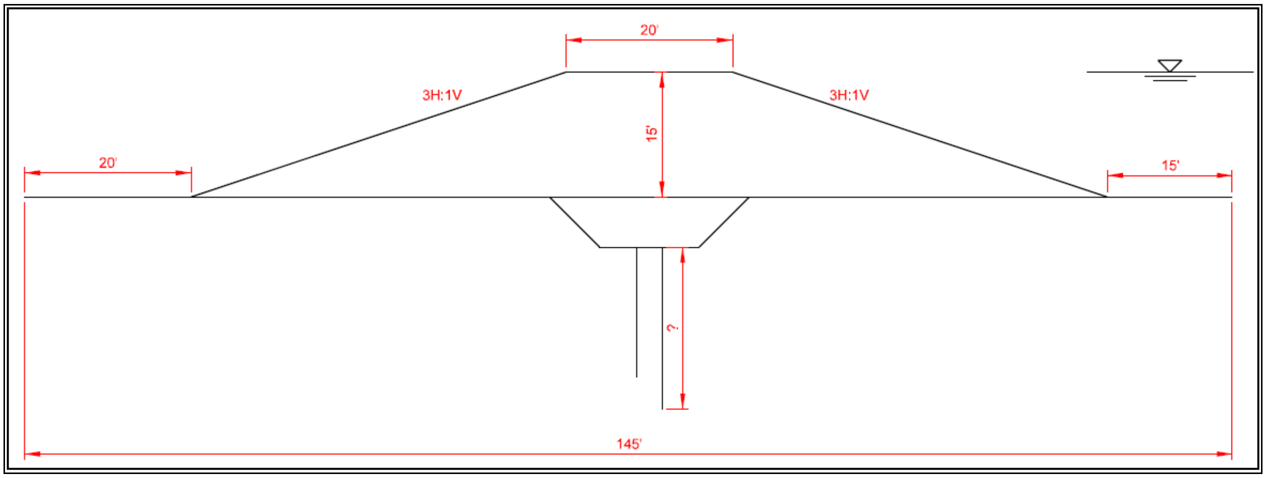


Figure 2-23B – Reach S4-North (Improvement Type 8 for 50% of Reach Length)

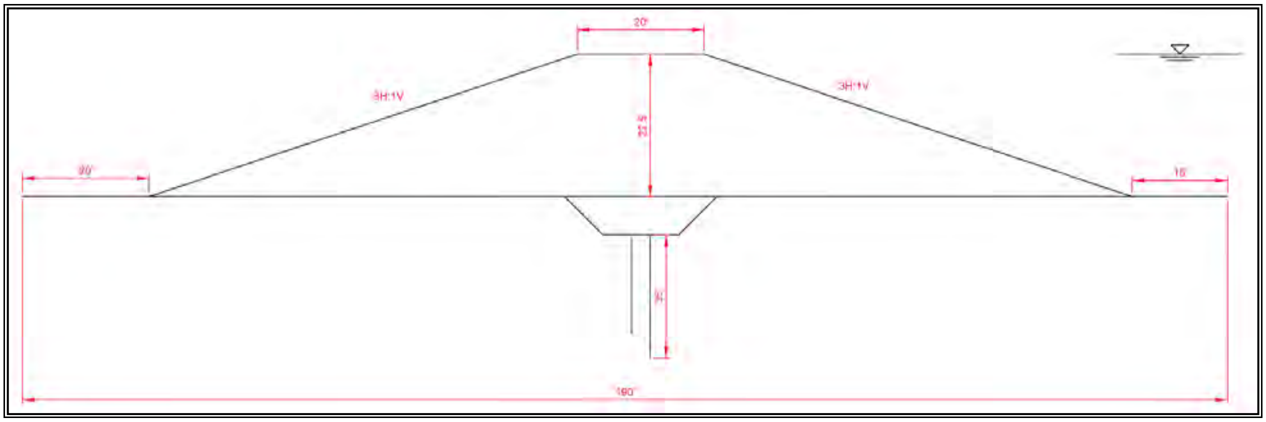


Figure 2-24 – Reach S9-G (Improvement Type 8 for 100% of Reach Length)

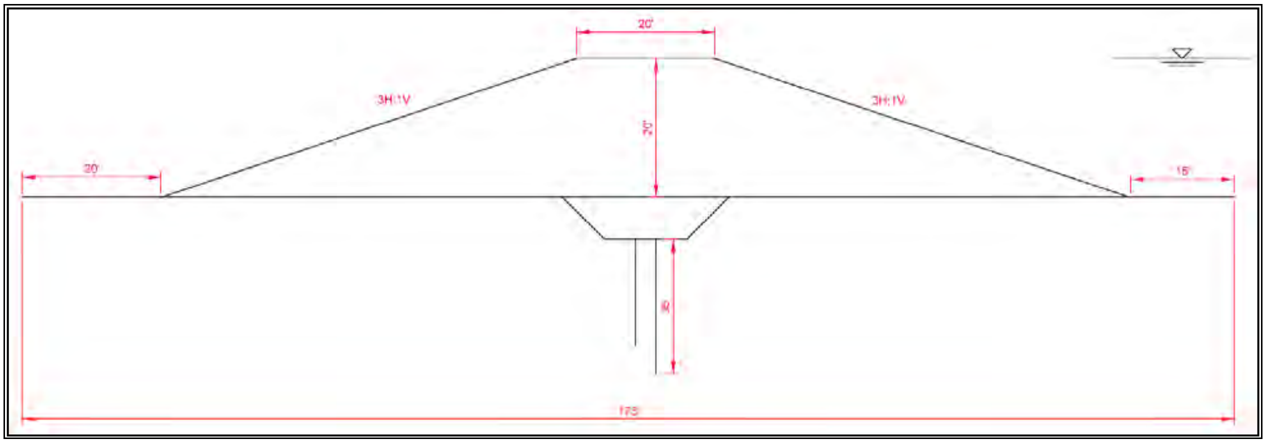


Figure 2-25 – Reach S9-H (Improvement Type 8 for 100% of Reach Length)

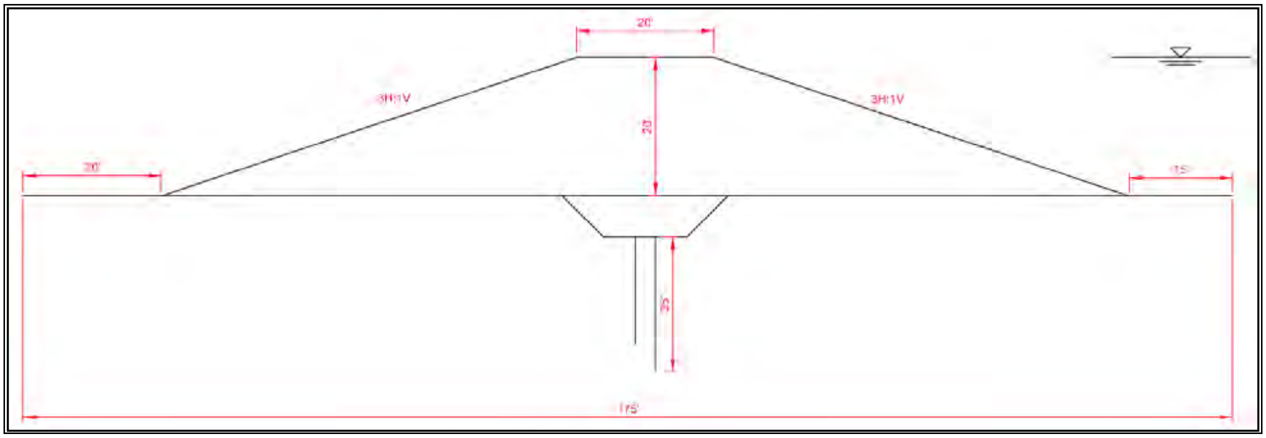


Figure 2-26 – Reach S9-I (Improvement Type 8 for 100% of Reach Length)

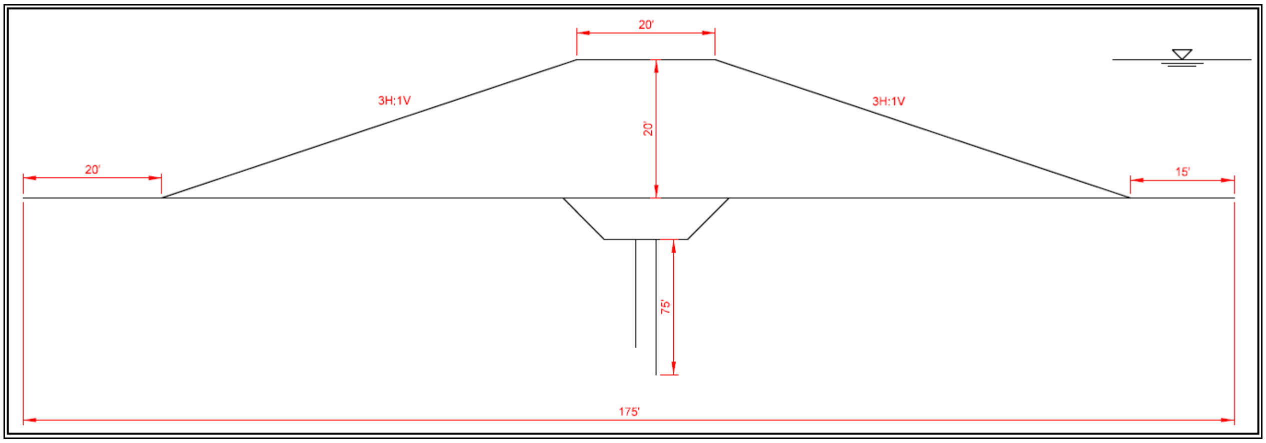


Figure 2-27 – Reach S10 (Improvement Type 8 for 100% of Reach Length)

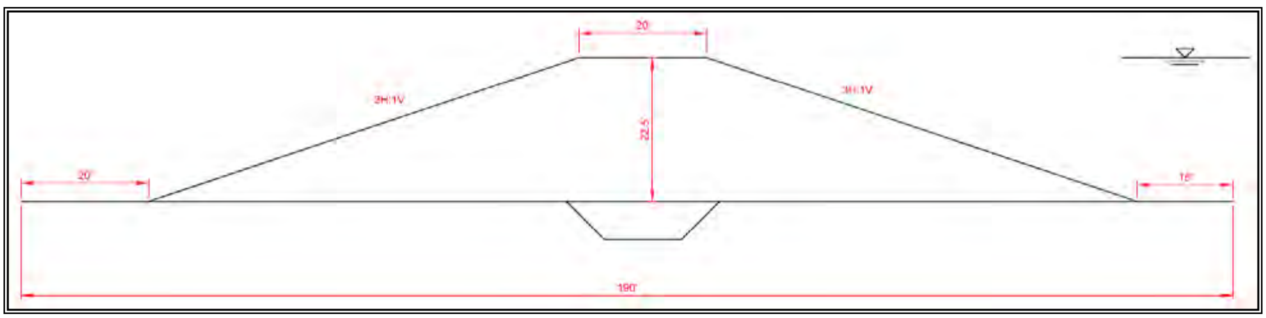


Figure 2-28A – Reach S11 (Improvement Type 7 for 50% of Reach Length)

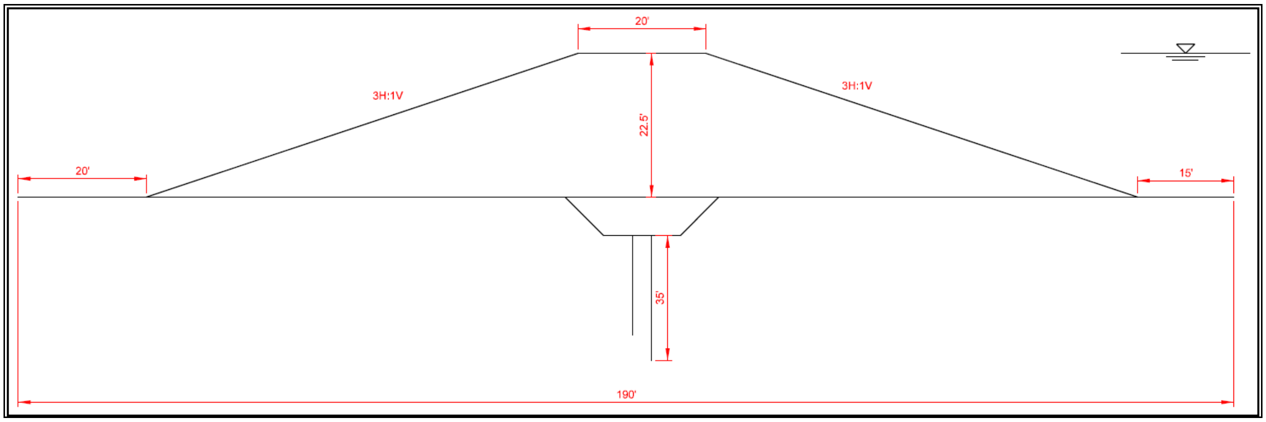


Figure 2-28B – Reach S11 (Improvement Type 8 for 50% of Reach Length)

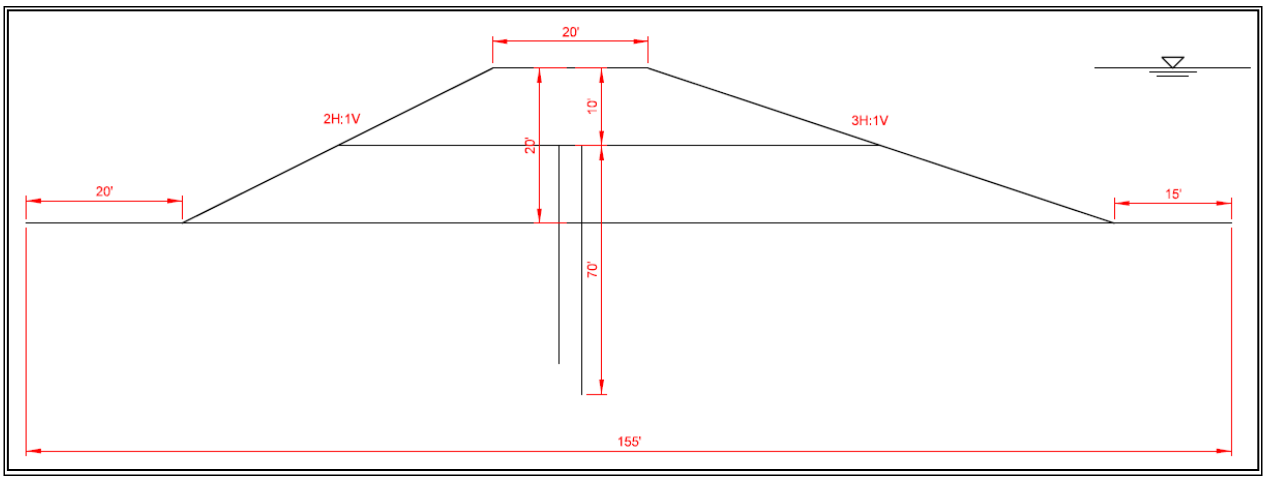


Figure 2-29A – Reach S12 (Improvement Type 6 for 25% of Reach Length)

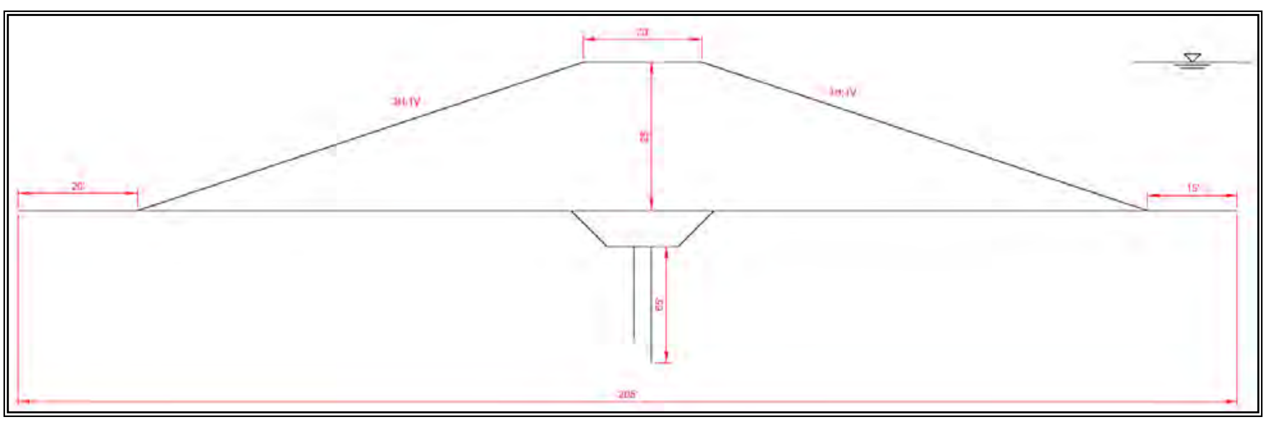


Figure 2-29B – Reach S12 (Improvement Type 8 for 75% of Reach Length)

2.5 Encroachments

The utilities (pipelines and conduits only) located within the proposed ROW for new levee segments (e.g. setback levees and ring levee segments) were not specifically addressed during this phase of the study and estimated as a lump sum percentage of the total utility cost. Physical structures located within the proposed ROW, roads and canals crossing the alignment of new levee segments were specifically addressed during this phase. New levee segments were defined in table 2-2 and shown in figure 1-1 of section 2.3.1.

A comprehensive inventory of all encroachments (utilities, physical structures and woody vegetations) located within the proposed ROW of the existing levee segments (see figure 2-10A to 2-20B) was completed based on existing data and field investigations. The existing encroachment data came from multiple sources including the CVFPB encroachment list, the USACE Periodic Inspection report and as-built of various projects located along the FRWL alignment. Field investigations were conducted to validate and improve the existing inventories.

The final encroachment list (enclosure 4, Encroachment Improvements & Estimates) shows numerous pipelines (both gravity and pressurized lines) and conduits (cables, electrical lines etc.) crossing the existing alignments of the FRWL, SBEL and WCEL. The record also indicated a number of utilities running parallel to the alignments (power poles, irrigation ditches, pipelines etc.), physical structures (public, residential and commercial buildings), and woody vegetation (mature trees) currently located within the proposed ROW of the existing levee segments. These encroachments were divided into 12 groups/types.

The following paragraphs outline the approach for addressing each type of encroachment. To avoid interference with construction of other project features, it is assumed that all levee penetrations will be removed prior to levee construction and disposed/replaced after the levee construction is completed. It is also assumed that temporary bypass will be provided at each utility improvement sites to avoid impacts to existing operations. All pipelines and conduits crossing the levee alignment will be modified to include positive closure devices and meet the USACE design criteria for levee penetrations in accordance with EM 1110-2-1913.

Refer to enclosure 4, Encroachment Improvements & Estimates, for the complete inventory, classification and remediation measures for all encroachments located within the proposed ROW of the existing levee segments.

2.5.1 Type 1

This group includes the major utilities those are crossing the levee prism and still in good condition. Relocation of these utility crossings above the DWSE would result in high construction cost and impacts. Therefore, the proposed remediation method is to construct jet grouting cutoff wall around the penetrations.

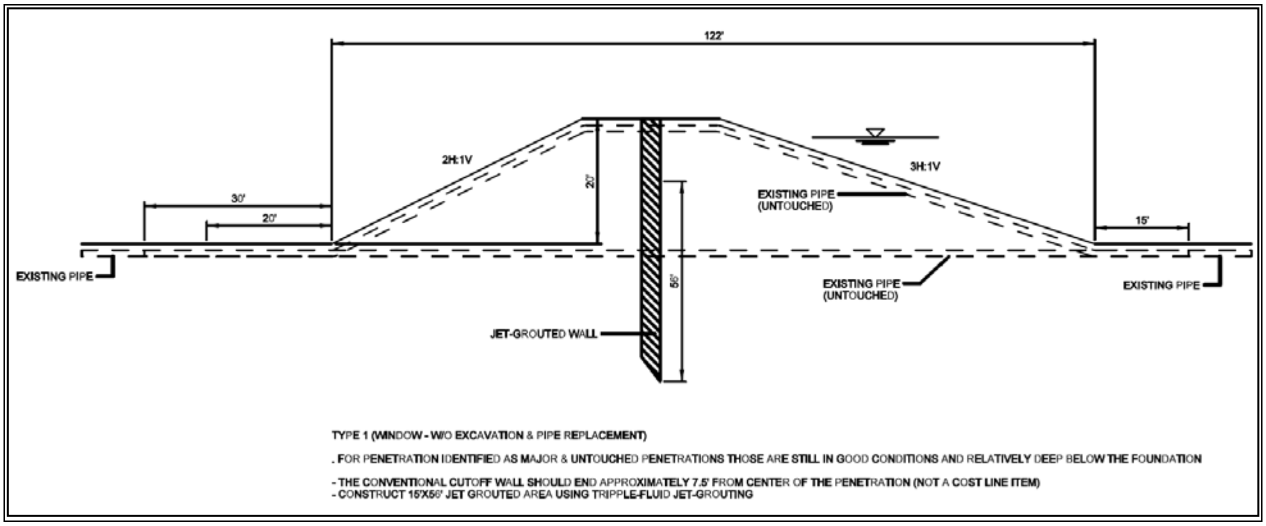


Figure 2-30A – Encroachment Type 1 – Section

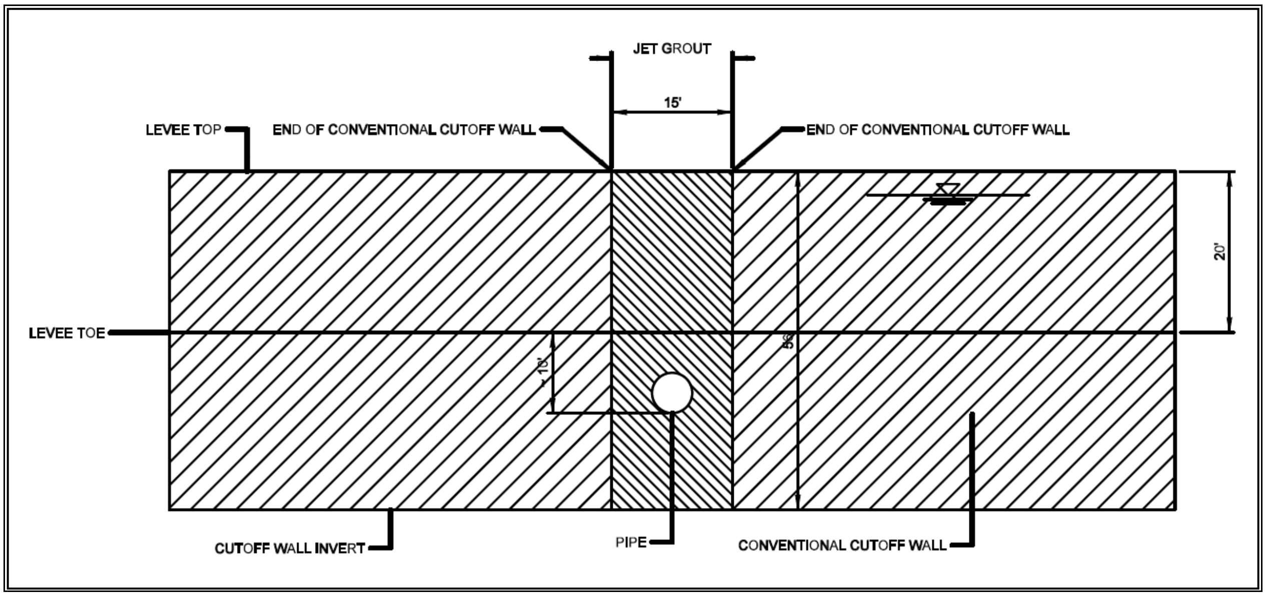


Figure 2-30B – Encroachment Type 1 – Profile

2.5.2 Type 2

This group includes the utilities those are crossing the levee prism (raised and through pipes/conduits) and abandoned. The proposed remediation method is to remove these abandoned penetrations.

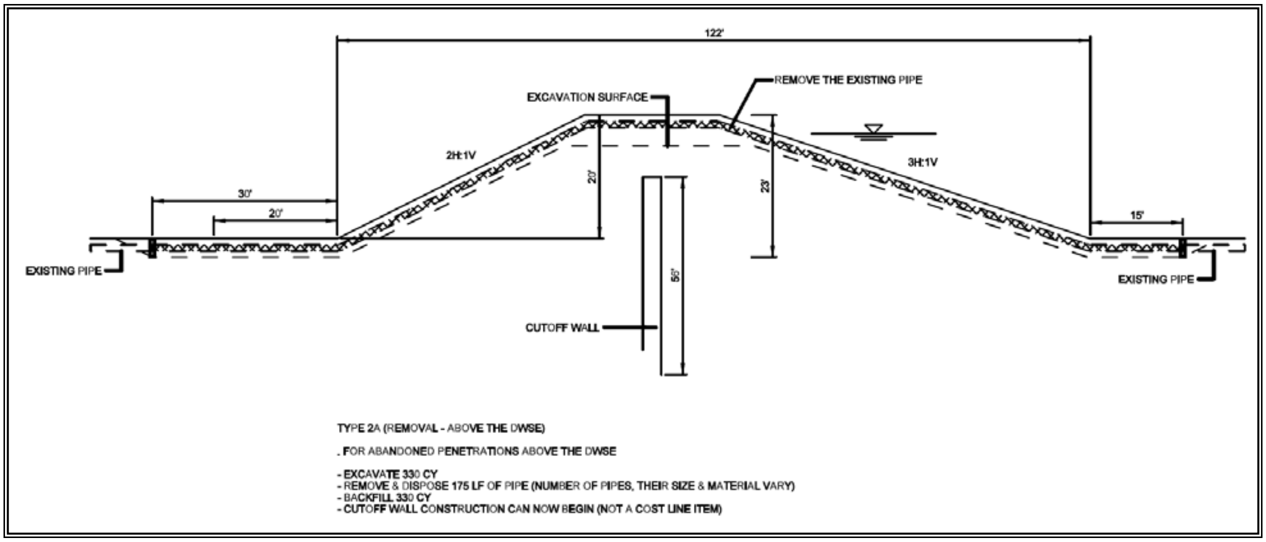


Figure 2-31A – Encroachment Type 2A – Section

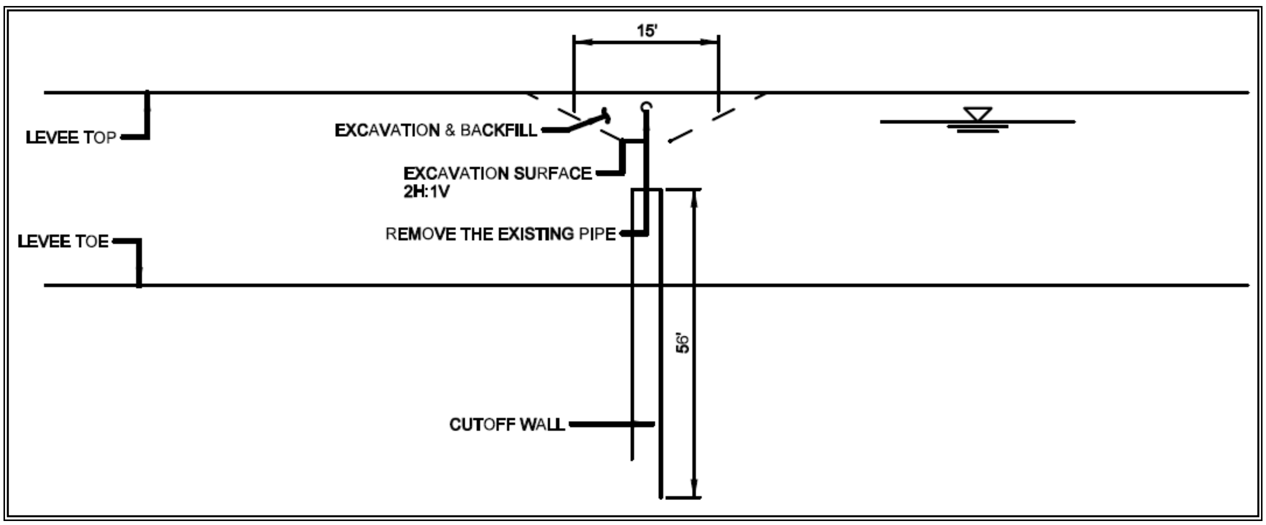


Figure 2-31B – Encroachment Type 2A – Profile

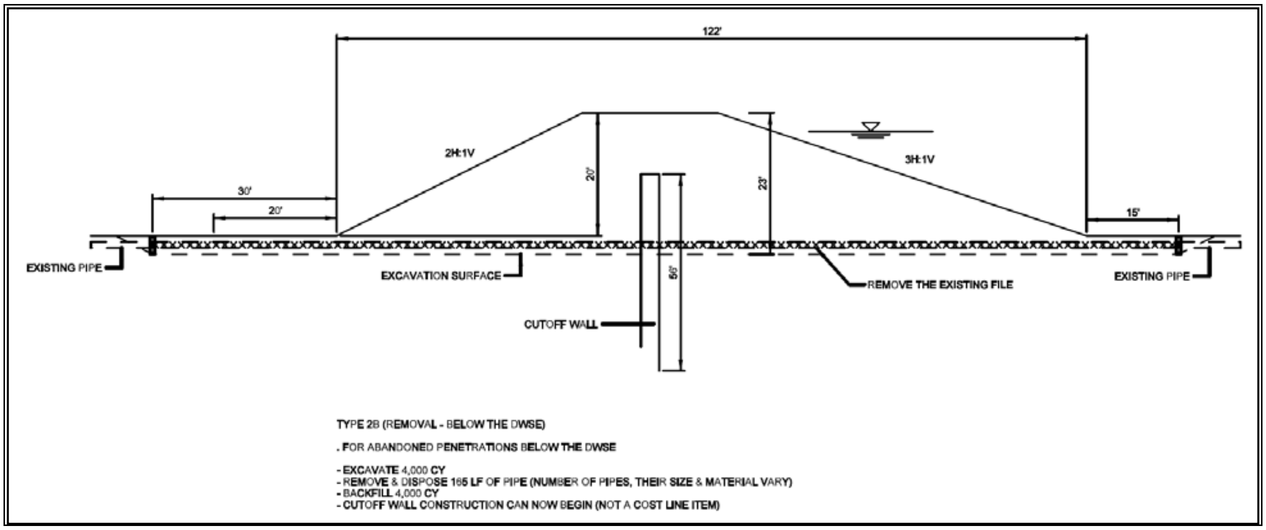


Figure 2-32A – Encroachment Type 2B – Section

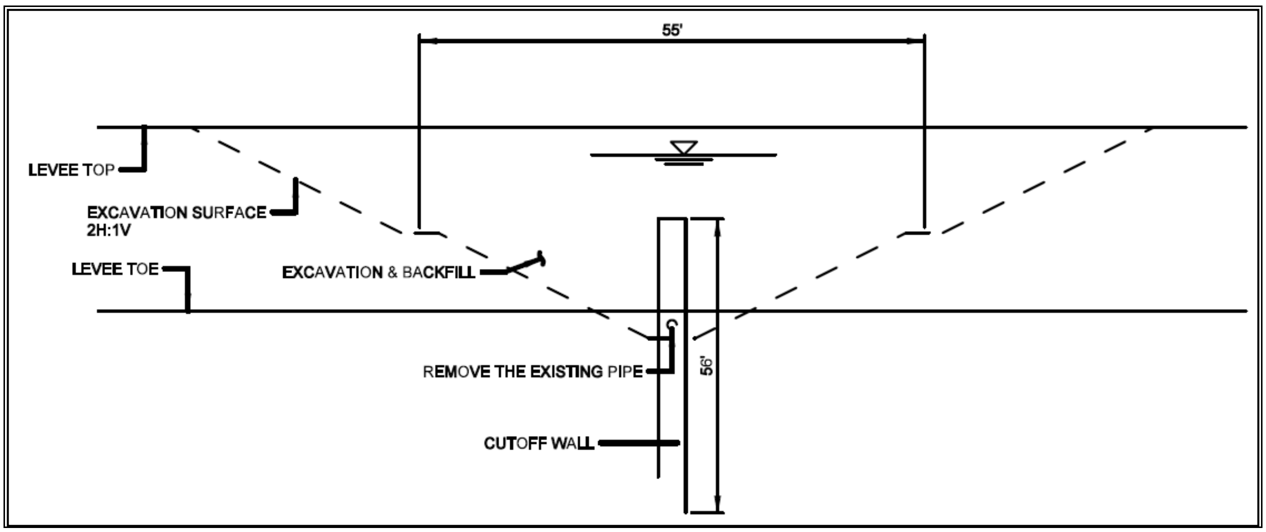


Figure 2-32B – Encroachment Type 2B – Profile

2.5.3 Type 3

This group includes utilities those are crossing the levee prism, dated and don't meet the current standard, include: (1) Communication conduits crossing the levee prism above the DWSE, (2) Minor pressurized pipelines crossing the levee prism above the DWSE, (3) Major pressurized pipelines crossing the levee prism below the DWSE, and (4) Gravity pipelines crossing the levee prism below the DWSE. These pipelines and conduits will be removed (before the cutoff wall construction begins) and replaced in-place (after the cutoff wall construction completes) with proper pipe materials and positive closure devices.

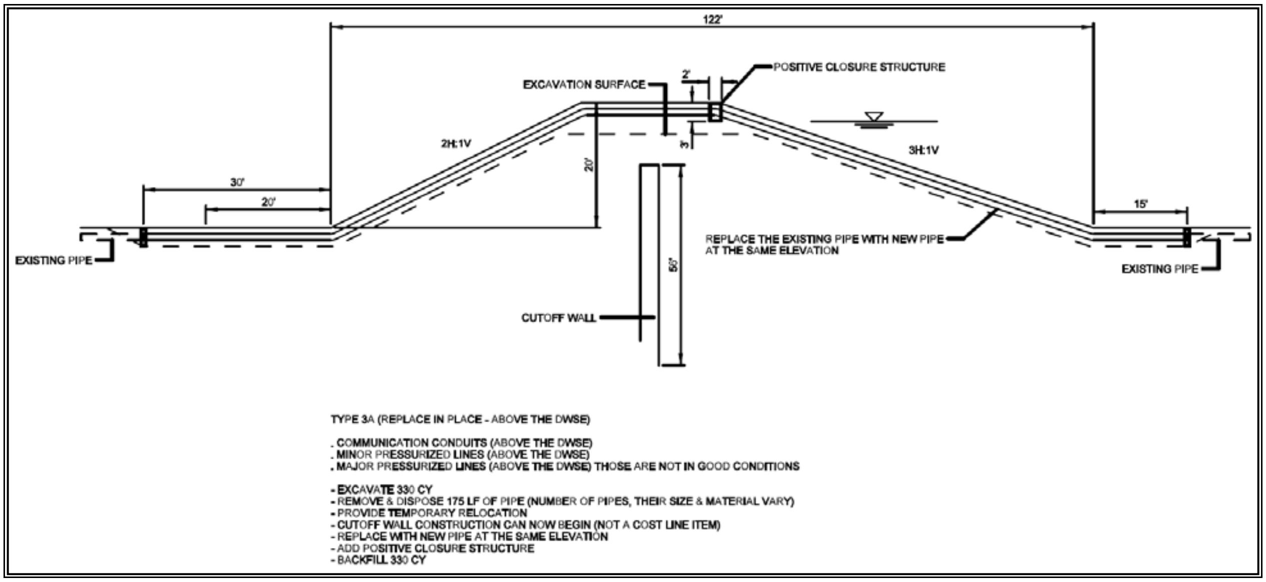


Figure 2-33A – Encroachment Type 3A – Section

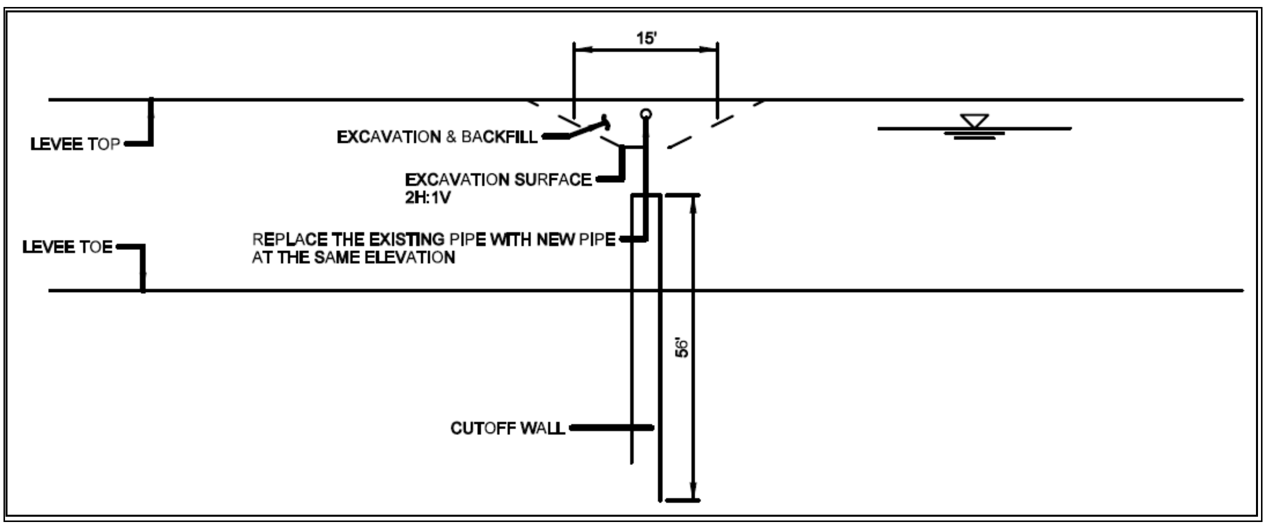


Figure 2-33B – Encroachment Type 3A – Profile

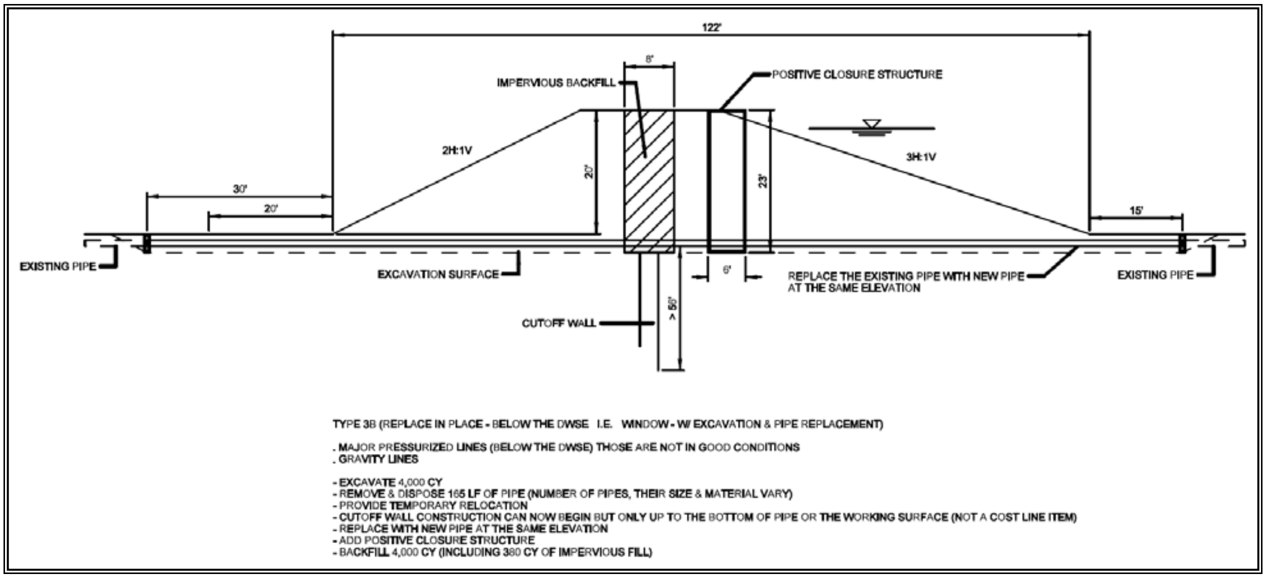


Figure 2-34A – Encroachment Type 3B – Section

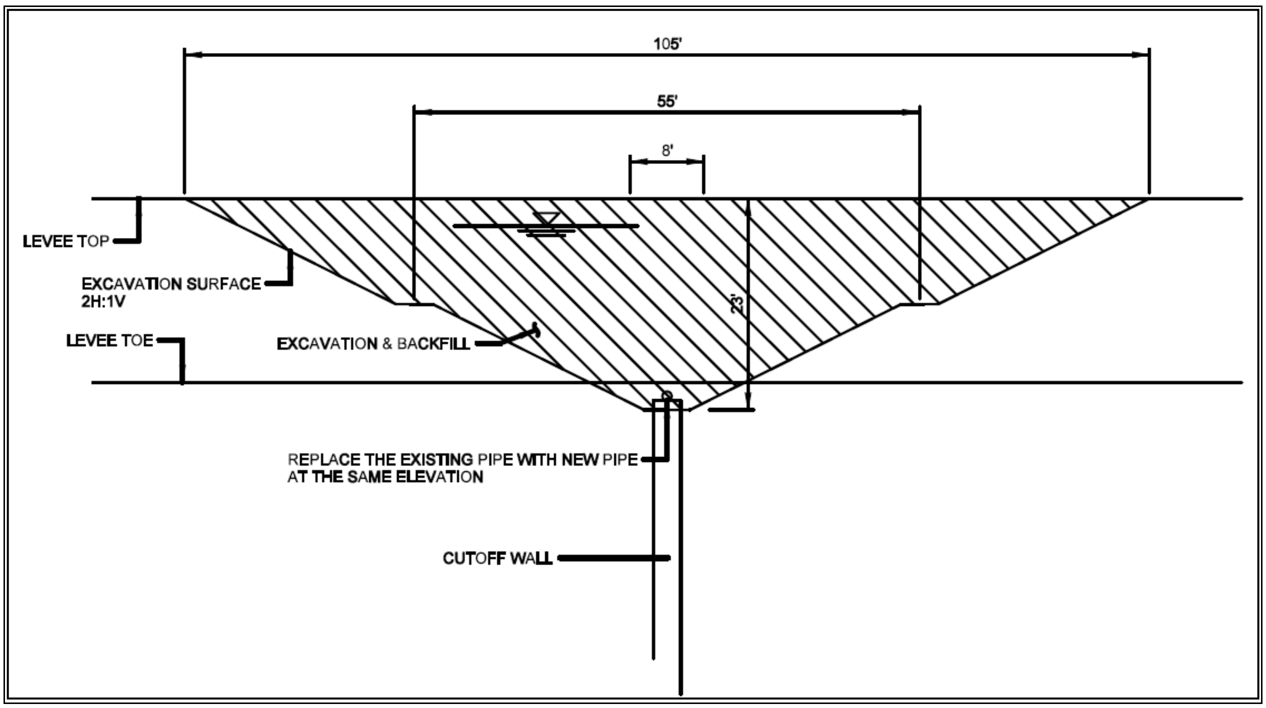


Figure 2-34B – Encroachment Type 3B – Profile

2.5.4 Type 4

This group includes utilities those are crossing the levee prism, dated and don't meet the current standard, include: (1) Communication conduits crossing the levee prism below the DWSE, and (2) Minor pressurized pipelines crossing the levee prism below the DWSE. These pipelines and conduits will be removed (before the cutoff wall construction begins) and replaced and relocated above the DWSE (after the cutoff wall construction completes) with proper pipe materials and positive closure devices.

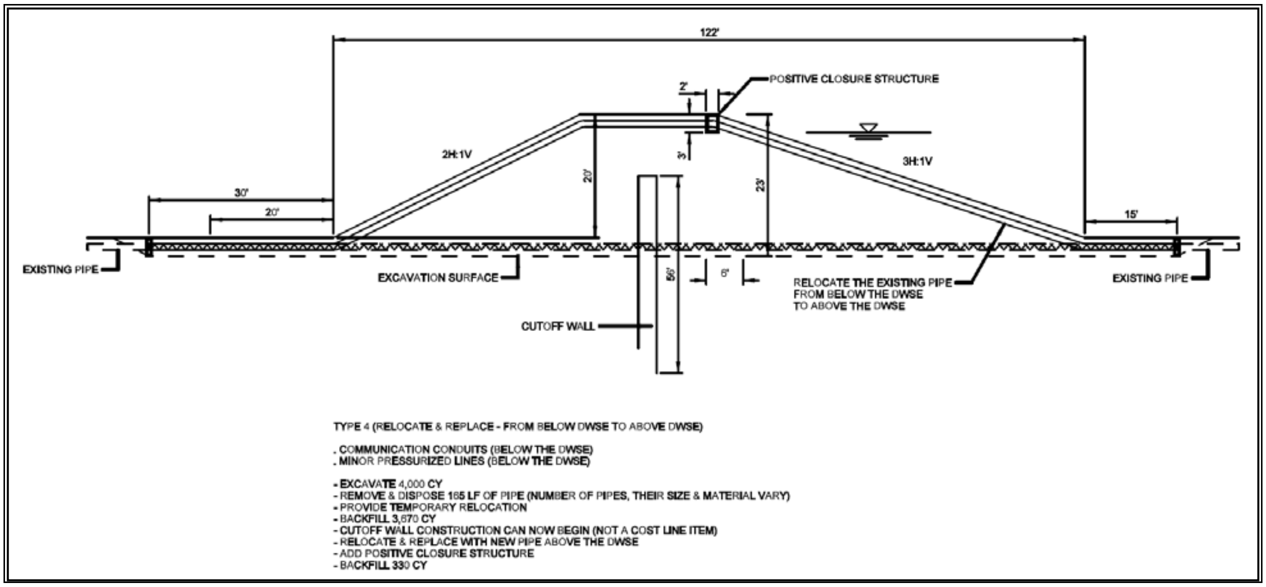


Figure 2-35A – Encroachment Type 4 – Section

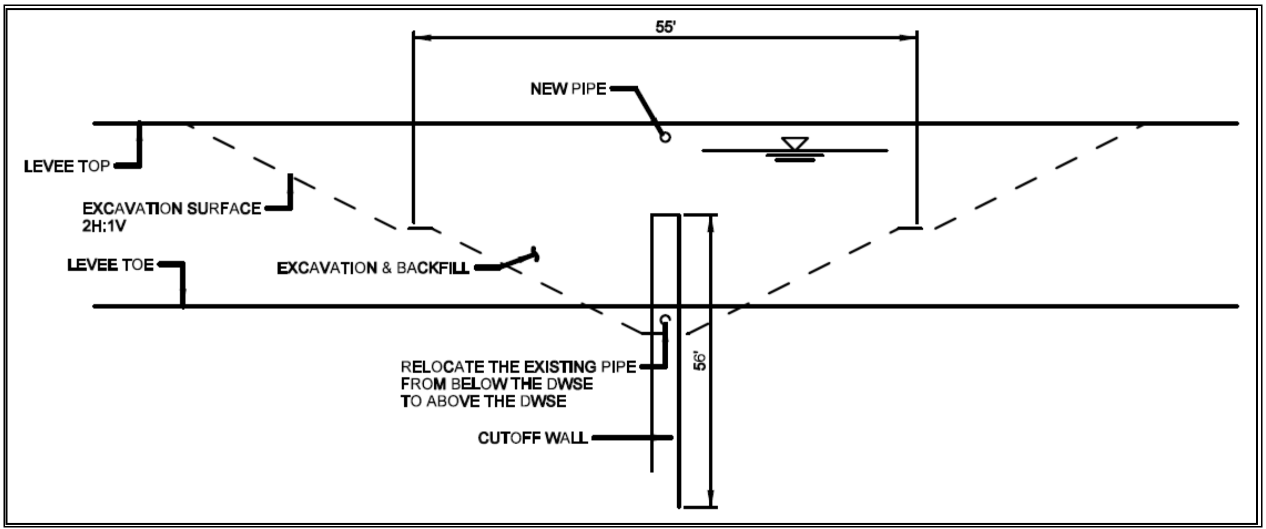


Figure 2-35B – Encroachment Type 4 – Profile

2.5.5 Type 5

This group includes bridges and railroads crossing the alignment of the existing levee. Deep Soil Mix (DSM) cutoff wall will be constructed at these locations.

2.5.6 Type 6

This group includes roads crossing the alignment of the new tall levee segments. Flood gate was initially considered as an option; however, because of the deep flood depth anticipated at these locations, these roads will be elevated up to the new top of levee.

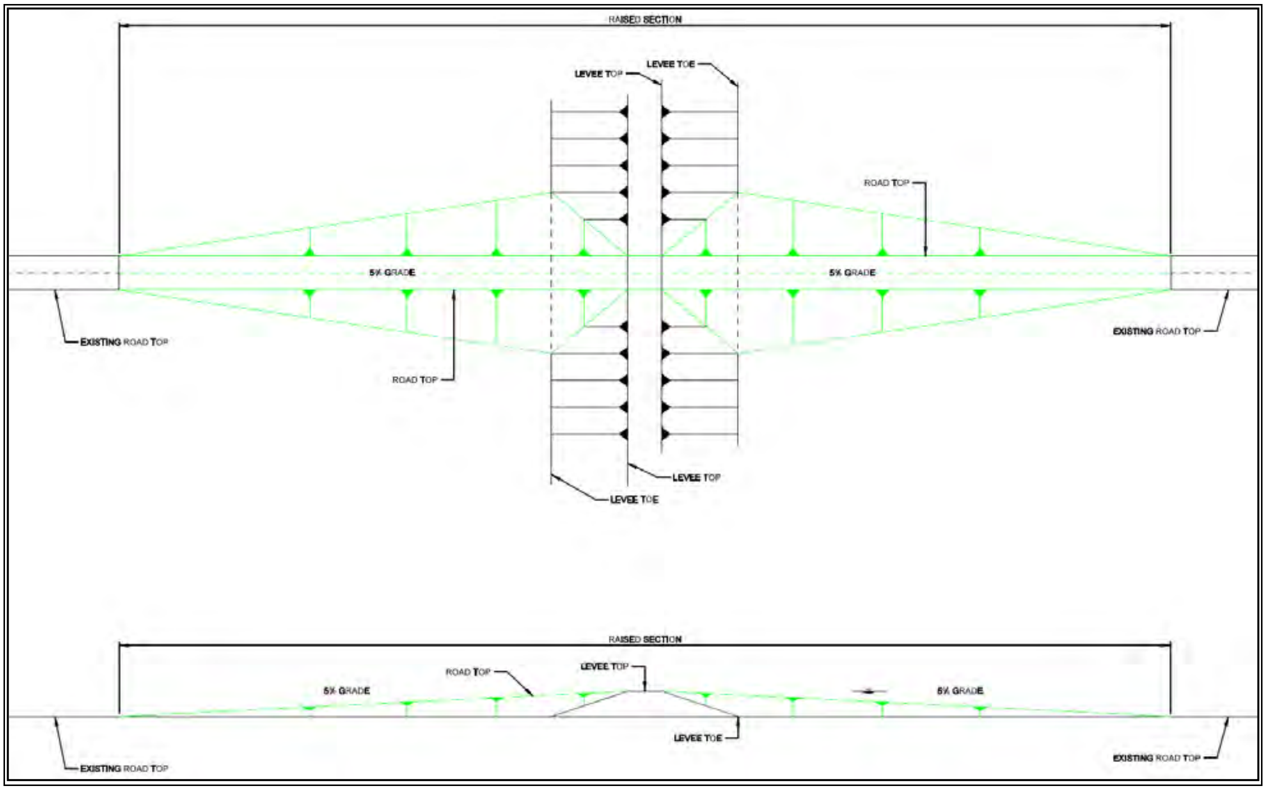


Figure 2-36 – Encroachment Type 6 – Plan and Section

2.5.7 Type 7

This group includes roads crossing the alignment of the new shallow levee segments. Because of the shallow flood depth anticipated at these locations, flood gate will be installed at these locations.

2.5.8 Type 8

This group includes canals crossing the alignment of the new levee segments. Relocation of these canals would result in high cost and impact. Therefore, the proposed remediation measure is to construct automatic closure structures at these canal crossings.

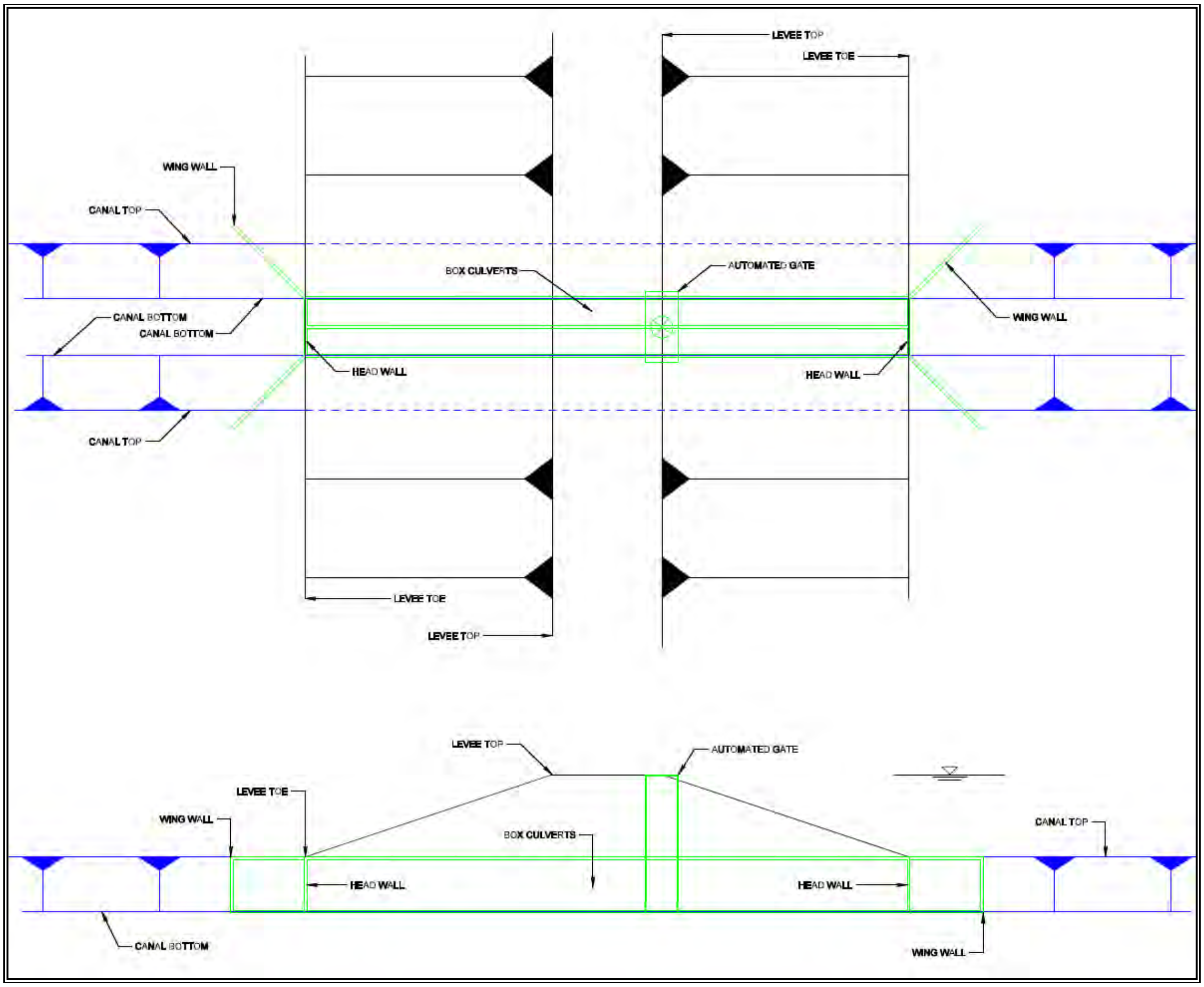


Figure 2-37 – Encroachment Type 8 – Plan and Section

2.5.9 Type 9

This group includes overhead power lines crossing the levee alignment. Temporary cutoff will be required to provide clearance for construction equipments where necessary. Power poles located within the proposed ROW will be relocated outside the proposed ROW, into a utility corridor.

2.5.10 Type 10

This group includes all physical structures (buildings, residential homes etc.) located within the proposed ROW of the existing and new levee segments. These structures will be relocated outside the proposed ROW.

2.5.11 Type 11

This group includes minor ditches and ponds located within the proposed ROW of the existing and new levee segments. These structures will be relocated outside the proposed ROW.

The Sutter Butte Main Canal (SBMC) falls within the proposed ROW at four locations along the FRWL alignment. Per Geotechnical Design recommendation, the SBMC encroachment was not specifically addressed during this phase of the study, however, captured as a part of the project's cost contingency during the Cost & Schedule Risk Analysis.

2.5.12 Type 12

This group includes all other overhead power poles, utility pipelines and conduits that are not crossing the levee alignment but located within the proposed ROW. These utilities will be relocated outside the proposed ROW, into a utility corridor.

2.6 Real Estate Requirement

The general Land, Easements, Rights-of-way, Relocation and Disposal Areas (LERRD)'s requirements include land acquisitions for levee footprint, O&M roads, utility corridors, temporary work areas, borrow and mitigation areas. The LERRD's requirements also include the relocation of physical structures (buildings, residential homes etc.) currently encroaching into the ROW.

The land acquisitions for levee footprint and O&M roads are necessary for construction, operation and maintenance of project features. The levee's and O&M road's footprints were established based on the final levee geometry (shown in figure 2-10A to 2-29B) and based on the distributions of typical levee improvement measures (shown in table 2-5A and 2-5B). In the figure, the levee footprint is the base width from the landside toe to the waterside toe of levee/berm. The landside O&M road is a 20-foot corridor along the landside toe of the levee/berm. The waterside O&M road is a 15-foot O&M corridor along the waterside toe of the levee/berm.

Additional land acquisitions for utility corridors, temporary work areas, borrow and mitigation areas were considered but not specifically addressed during this phase of the study. The utility corridor (approximately 20ft beyond the PRE for O&M roads) may be needed for relocation of utilities parallel to the project's alignment outside of the proposed ROW. Temporary work areas, borrow and mitigation areas are necessary for construction of the project features. These additional real estate requirements were not specifically identified and estimated as lump sum percentages of the total real estate requirements.

The number of physical structures to be relocated was estimated based on the ROW requirements (see paragraph 2.4.2).

2.7 Quantity Development

2.7.1 Levee and Cutoff Wall Constructions

The quantity estimates for levee and cutoff wall constructions (e.g. excavation and backfill, cutoff wall area etc.) were completed using the parametric approach. In this approach, the quantities were estimated as products of sectional area and length of different types of levee improvements. The sectional areas of levee improvements were based on the levee geometry shown in figure 2-10A to 2-29B. The lengths of the levee segment where a typical improvement measure applied were based on the distribution shown in table 2-5A and 2-5B. Refer to the URS Parametric Cost Estimating MII Toolbox for the quantity estimates for levee and cutoff wall constructions.

2.7.2 Improvements and Relocations of Encroachments

The quantity estimates for encroachments (type 1 through 12) are shown in enclosure 4, Encroachment Improvements & Estimates, based on the recommendations provided in paragraph 2.5.

CHAPTER 3 – ALTERNATIVE DESCRIPTIONS

3.1 General

Based on table 2-3 and 2-5B, the project features included in each potential alternative will be as follows:

Table 3-1 – Draft Array of Potential Alternatives

Reach	SB-1	SB-2	SB-3	SB-4	SB-5	SB-6	SB-7	SB-8
Stability Berm								
Stability Berm with Relief Wells								
Seepage Berm								
Gravel Stability Berm		14,075	3,130	10,705	57,229	73,581	16,865	60,019
Waterside Soil-Bentonite Slurry Cutoff Wall								
Centerline Soil-Bentonite Slurry Cutoff Wall		58,325	15,650	38,375	142,517	282,108	79,250	163,442
New Levee			42,750	34,250				
New Levee w/ Centerline SB Slurry Cutoff Wall			32,250	20,750				
Levee Crest Widening		95,300	31,300	61,600	95,300	142,200	123,200	123,200

Detailed description of the alternatives is discussed in paragraph 3.2

3.2 Alternative Descriptions

3.2.1 Alternative SB-1

Under this alternative, the Federal government would take no action toward implementing a specific flood risk remediation measures. See plate 1-1.

3.2.2 Alternative SB-2

This alternative includes fix-in-place Feather River levees from Sunset Weir to Star Bend (see plate 1-2), and includes fix-in-place levee structural measures and non-structural measures. The structural measures are shown in table 3-1.

3.2.3 Alternative SB-3

This is a primarily non-structural alternative that includes the construction of a new levee surrounding Yuba City (see plate 1-3) and utilizing fixed-in-place eastern sections of the existing levee, and includes fix-in-place levee, new ring levee structural measures and non-structural measures. The structural measures are shown in table 3-1. Two new pump stations were assumed to be required to address interior drainage.

3.2.4 Alternative SB-4

This alternative is a non-structural/structural hybrid that includes fixing-in-place the Feather River levees north of Yuba City from Shangahi Bend to Thermalito, and the construction of a new levee on the south and west of Yuba City (little J). See plate 1-4. Fix-in-place levee and new levee structural measures and non-structural measures are included in this alternative. The structural measures are shown in table 3-1. This alternative assumes two new pump stations to address interior drainage.

3.2.5 Alternative SB-5

This alternative is inclusive of alternative SB-2, and further extends levee fix-in-place improvements north to Thermalito Afterbay (see plate 1-5), and includes fix-in-place levee structural measures and non-structural measures. The structural measures are shown in table 3-1.

3.2.6 Alternative SB-6

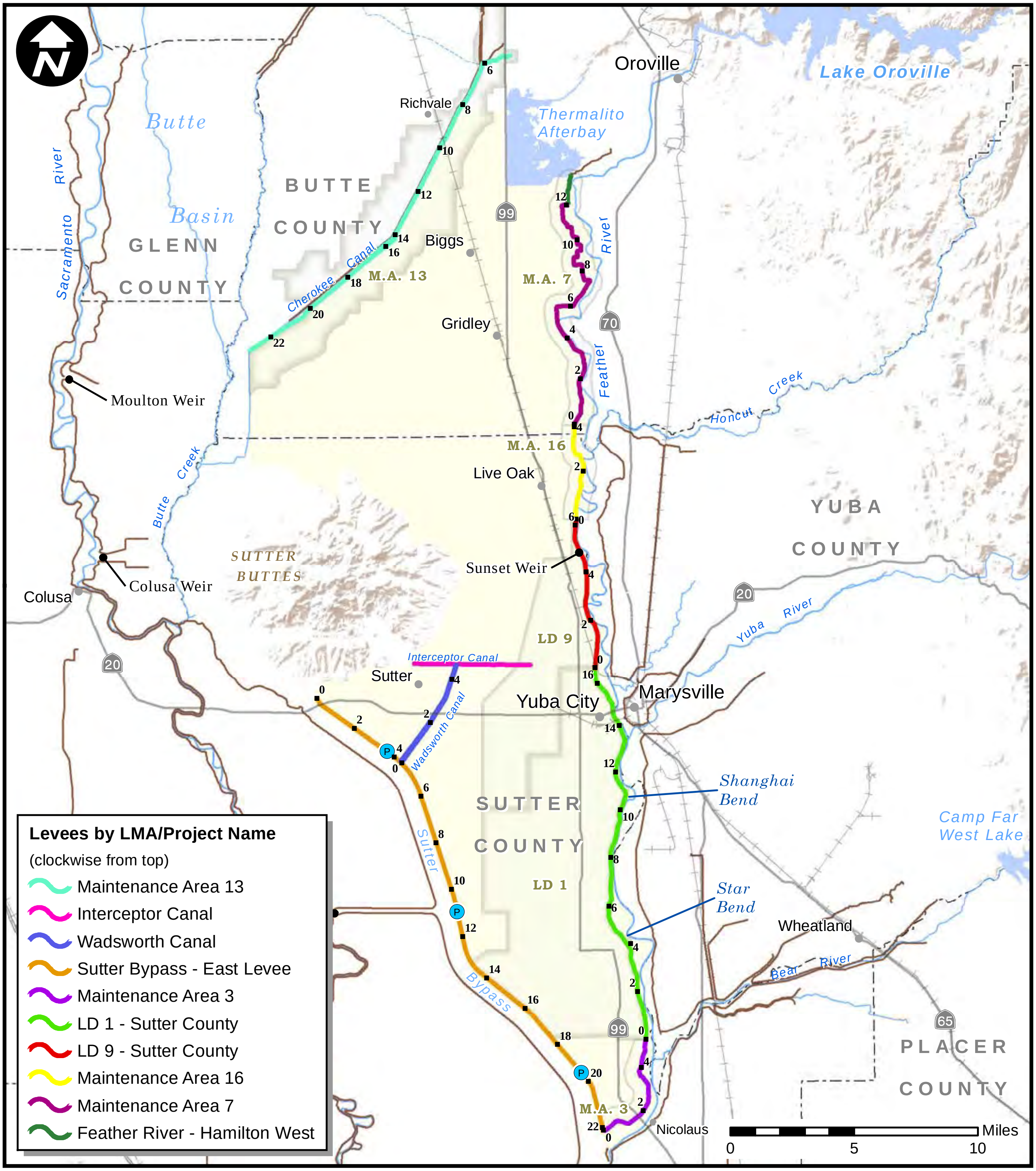
This alternative consists of the Sutter Bypass/Wadsworth Canal Levee fix-in-place improvements and fix-in-place levee improvements to all Feather River Levees (see plate 1-6), and includes fix-in-place levee structural measures and non-structural measures. The structural measures are shown in table 3-1.

3.2.7 Alternative SB-7

This alternative includes Alternative SB-2 and extends Feather River fix-in-place levee improvements south of Yuba City to Laurel Ave (see plate 1-7), and includes fix-in-place levee structural measures and non-structural measures. The structural measures are shown in table 3-1.

3.2.8 Alternative SB-8

This alternative is inclusive of Alternative SB-7 and extends Feather River levee improvements north to Thermalito (see plate 1-8), and includes fix-in-place levee structural measures and non-structural measures. The structural measures are shown in table 3-1.



Legend

- Study Area Extent
- Levee Maintenance Agency
- Federal Levee
- Levee Mile
- Pump Station
- City or Town
- Lake or Reservoir
- River or Stream
- Highway
- Railroad
- County Boundary

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

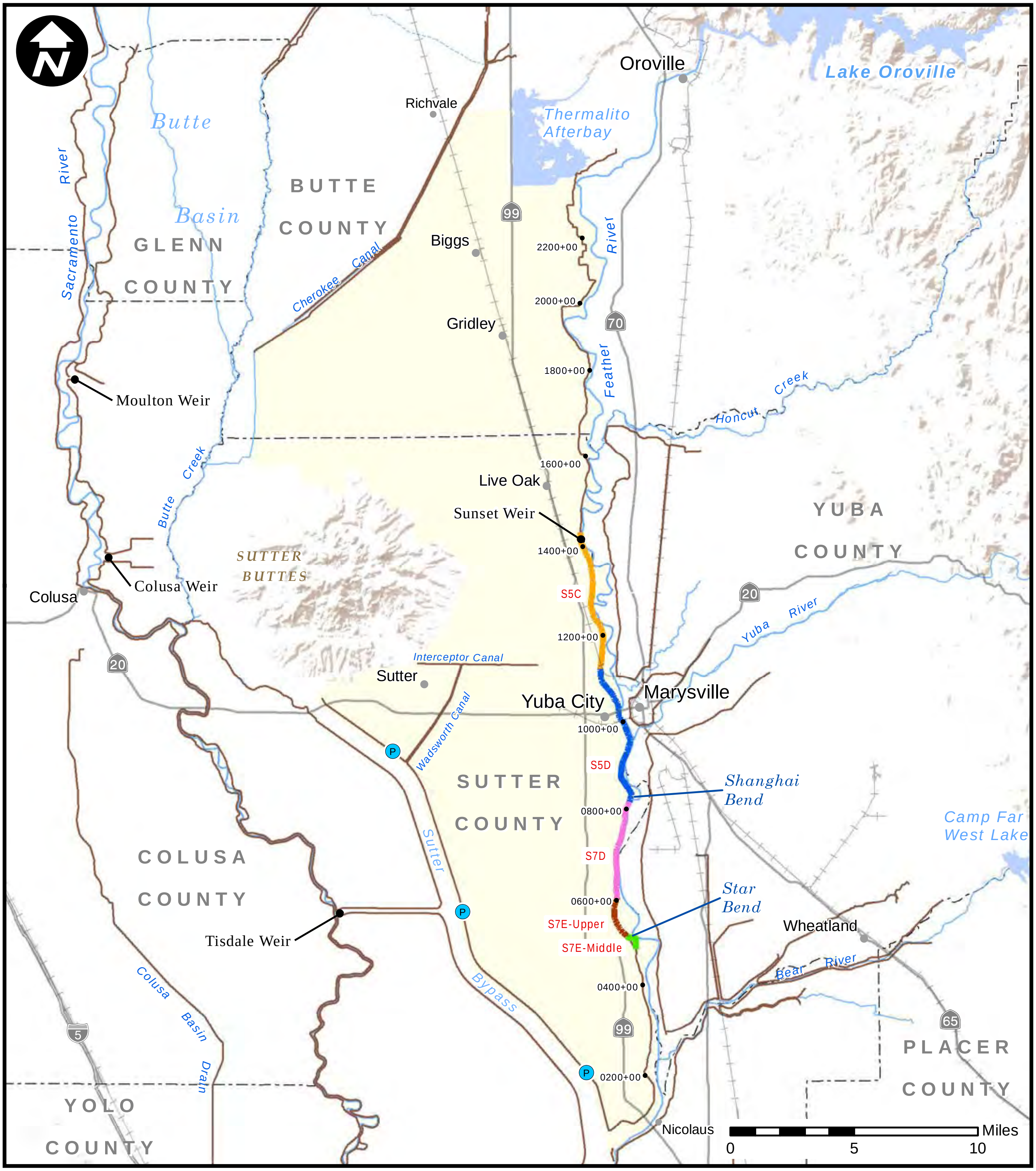
SUTTER BASIN PROJECT AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by Jim Mars

APR 2013

PLATE 1-1



Legend

- Study Area Extent
- S5B Project Reach
- Federal Levee
- Stationing Point
- P Pump Station
- City or Town
- Lake or Reservoir
- River or Stream
- Highway
- Railroad
- County Boundary

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-2: MINMAL FIX-IN-PLACE FEATHER RIVER LEVEES: SUNSET WEIR TO STAR BEND

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SACRAMENTO DISTRICT

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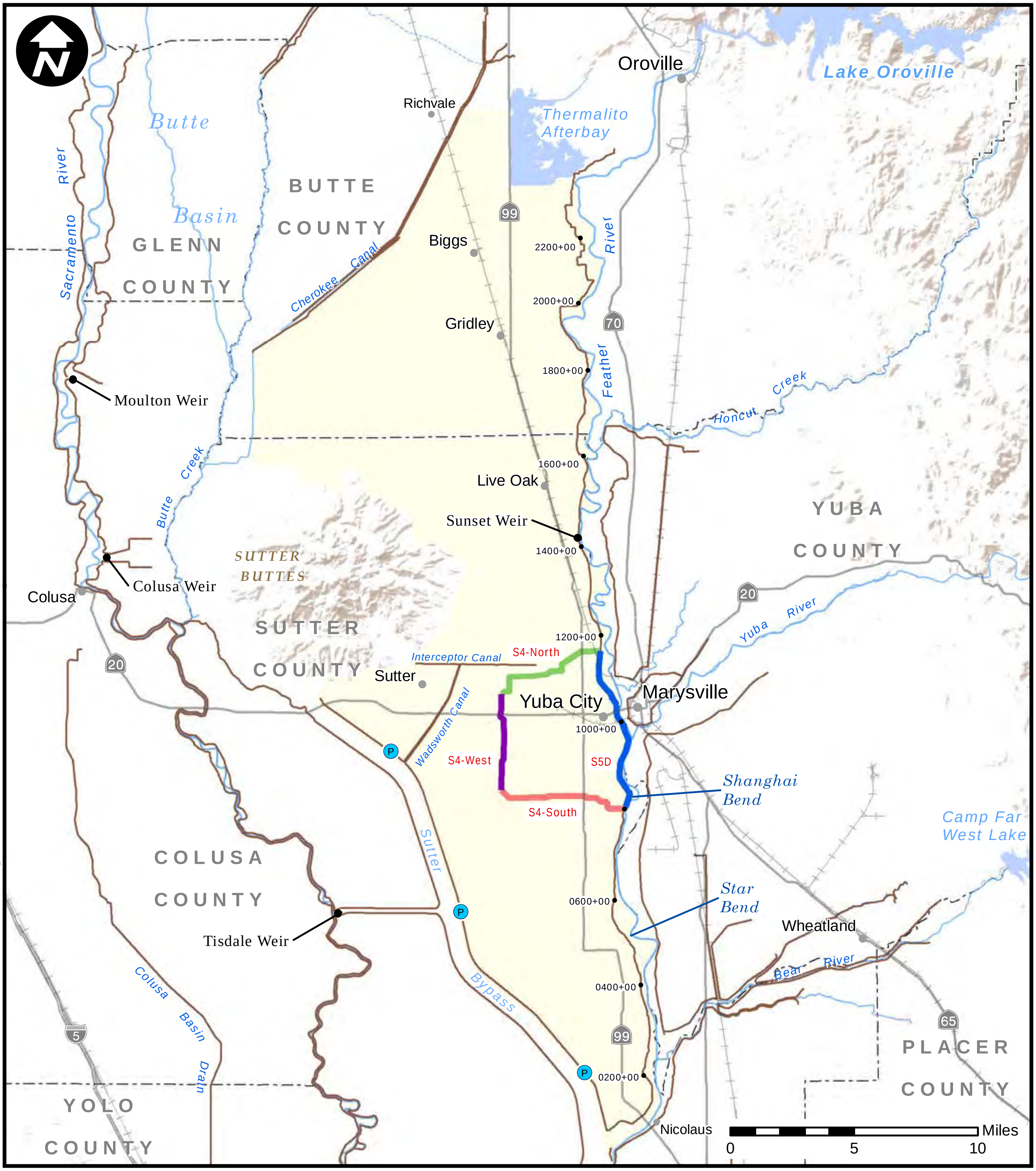

Legend

Study Area Extent
S5B Project Reach
Federal Levee
Stationing Point
Pump Station
City or Town
Lake or Reservoir
River or Stream
Highway
Railroad
County Boundary

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-3:
YUBA CITY RING LEVEE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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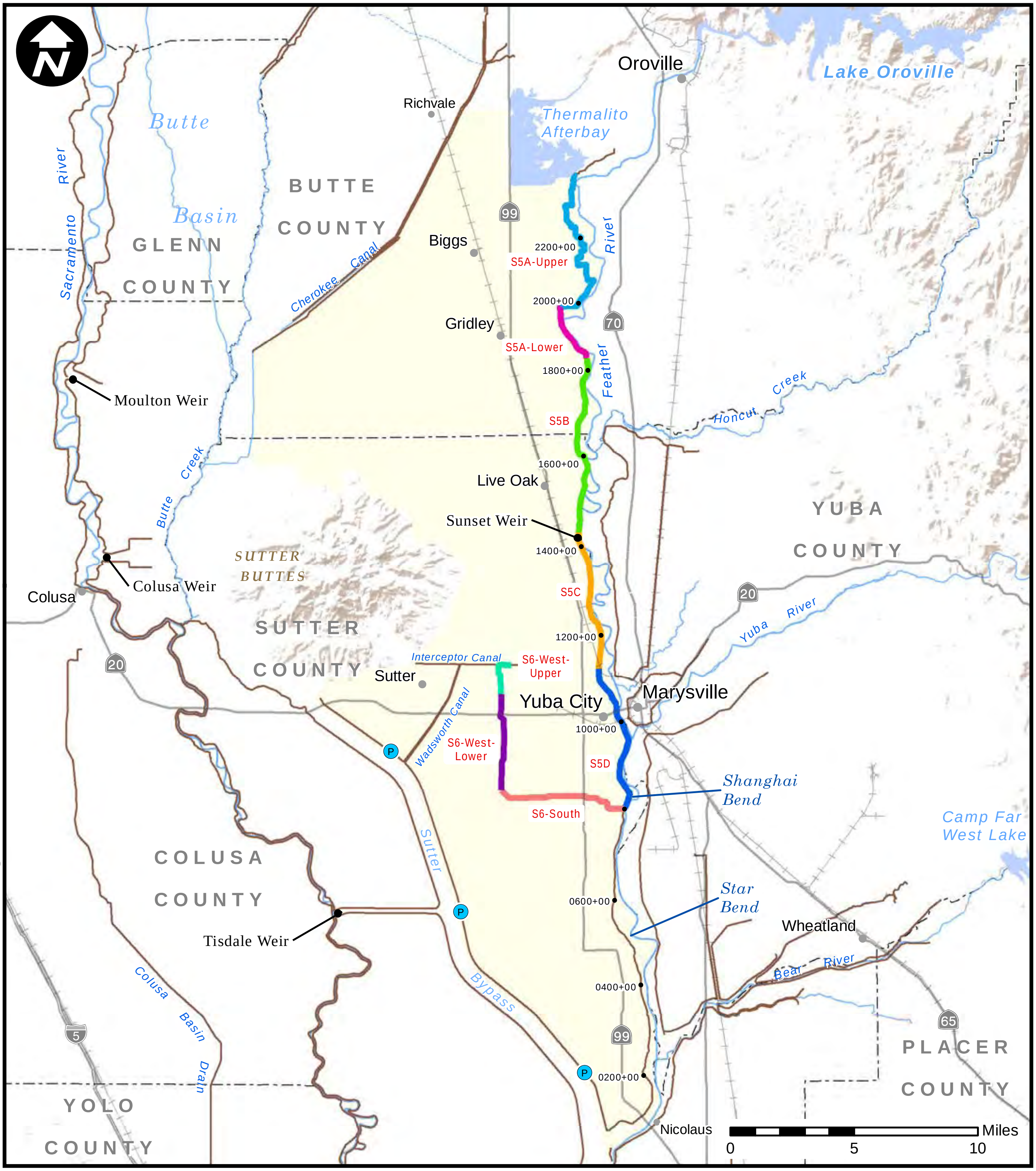
PLATE 1-3



Legend

- Study Area Extent
- S5B Project Reach
- Federal Levee
- Stationing Point
- P Pump Station
- City or Town
- Lake or Reservoir
- River or Stream
- Highway
- Railroad
- County Boundary

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-4: LITTLE J LEVEE

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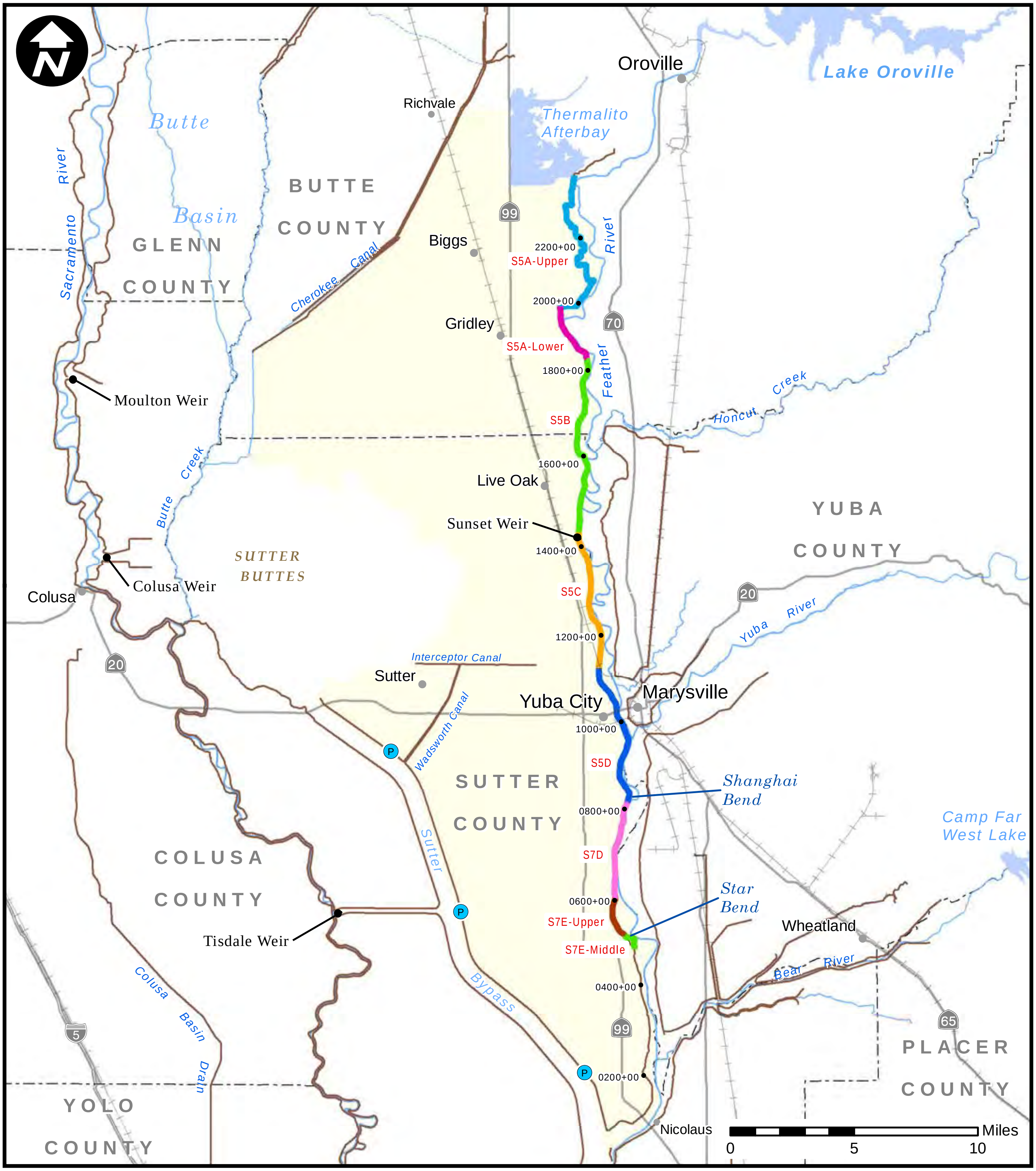

Legend

Study Area Extent	Lake or Reservoir
S5B Project Reach	River or Stream
Federal Levee	Highway
Stationing Point	Railroad
Pump Station	County Boundary
City or Town	

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-5: FIX-IN-PLACE FEATHER RIVER LEVEES: THERMAILTO TO STAR BEND

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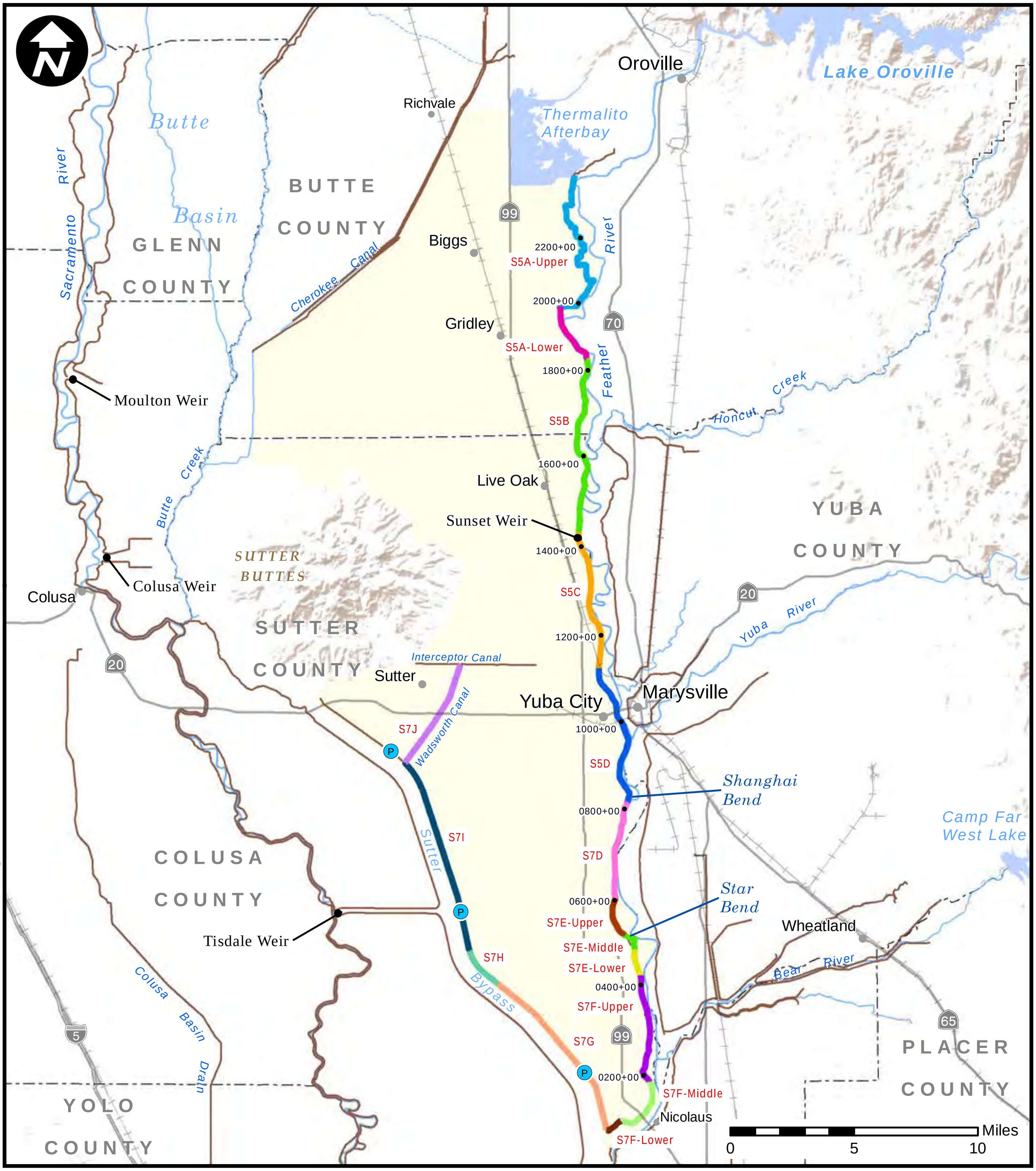

Legend

Study Area Extent	Lake or Reservoir
S5B Project Reach	River or Stream
Federal Levee	Highway
Stationing Point	Railroad
P Pump Station	County Boundary
City or Town	

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-6: FIX-IN-PLACE FEATHER RIVER, SUTTER BYPASS, AND WADSWORTH CANAL LEVEES

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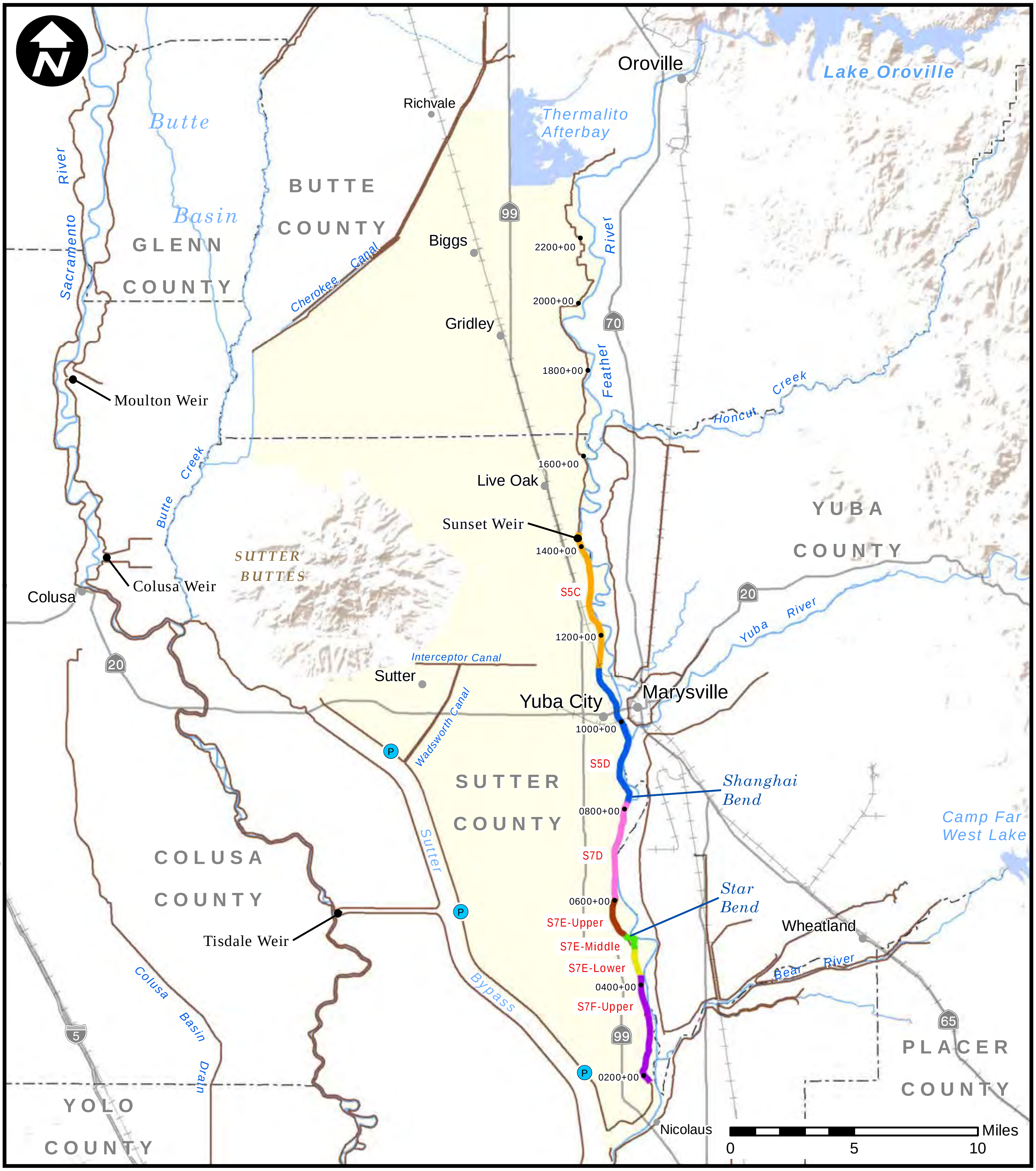


Legend

Study Area Extent	Lake or Reservoir
S5B Project Reach	River or Stream
Federal Levee	Highway
Stationing Point	Railroad
Pump Station	County Boundary
City or Town	

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

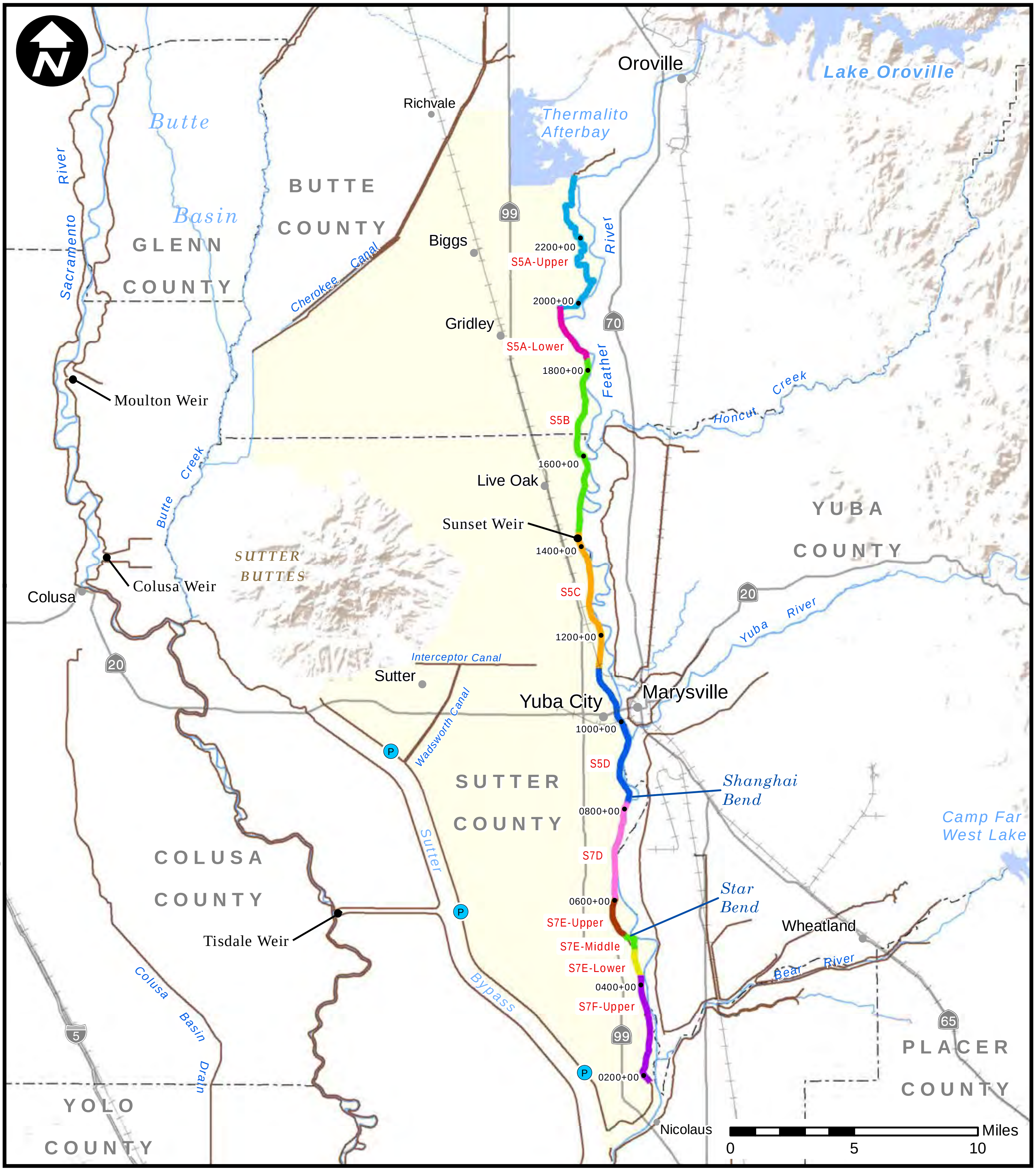
ALTERNATIVE SB-7: FIX-IN-PLACE FEATHER RIVER LEVEES: SUNSET WEIR TO LAUREL AVE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by Jim Mars

APR 2013

PLATE 1-7



Legend

Study Area Extent	Lake or Reservoir
S5B Project Reach	River or Stream
Federal Levee	Highway
Stationing Point	Railroad
Pump Station	County Boundary
City or Town	

SUTTER BASIN PILOT FEASIBILITY STUDY
ENGINEERING APPENDIX

ALTERNATIVE SB-8:
FIX-IN-PLACE
FEATHER RIVER LEVEES:
THERMALITO TO LAUREL AVE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

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APR 2013