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GEOTECHNICAL REPORT
DAD'S POINT LEVEE EVALUATION
STOCKTON, CALIFORNIA

MARCH 11, 2010

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File No. 92459.G02
March 11, 2010 (revised)

Mr. David Peterson
Peterson Brustad, Inc.
1180 Iron Point Road, Suite 260
Folsom, CA 95630

Subject: **Geotechnical Report**
Dad's Point Levee Evaluation, Smith Canal
Conditional Letter of Map Revision
Stockton, California

Dear Mr. Peterson:

Kleinfelder is pleased to present the attached geotechnical report for inclusion in the submittal to the Federal Emergency Management Agency (FEMA) for certification of the Dad's Point Levee located in Stockton, California. The scope of this investigation included evaluating approximately 1,600 feet of levee along the east bank of the San Joaquin River at the confluence with Smith Canal in Stockton, California. The purpose of our investigation was to explore subsurface conditions along the levee and perform a geotechnical evaluation of existing levee conditions in accordance with FEMA's requirements in support of the Smith Canal Conditional Letter of Map Revision (CLOMR). The enclosed report contains a summary of our field explorations, laboratory testing results, and engineering analyses and our conclusions and recommendations for levee mitigation.

Based on available geotechnical data and the results of our field exploration, laboratory testing, and engineering analyses completed to date, it is our professional opinion that the subject levee currently meets FEMA geotechnical requirements for freeboard, seepage, and slope stability.

We appreciate the opportunity to provide our services for this project. If you have questions regarding this report or if we may be of further assistance, please contact us.

Respectfully submitted,

KLEINFELDER WEST, INC.

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1 INTRODUCTION

1.1. GENERAL

This report presents the results of our geotechnical investigation associated with the evaluation of the existing levee along the east bank of the San Joaquin River at the confluence with Smith Canal in Stockton, California (see Plate 1). Currently, the levee is a peninsula with water on both the landside and waterside at the same elevation because of the connection of the San Joaquin River to the west of Dad's Point and Smith Canal to the east of the levee. However, it is currently planned to construct a closure structure that would have the potential to separate the San Joaquin River from Smith Canal during a high water event in the San Joaquin River which would create the potential for a head differential on either side of the levee. This report will be used to assist the San Joaquin Area Flood Control Agency (SJAFCA) in documenting that this section, with a length of approximately 1,600 feet or about 0.30 miles, meets the design criteria described in Title 44 Code of Federal Regulations 65.10 (44 CFR Section 65.10).

Conclusions and recommendations presented in this report are based on the subsurface conditions encountered at the locations of our explorations from our current and previous investigations and the provisions and requirements outlined in the LIMITATIONS section of this report. Recommendations presented herein should not be extrapolated to other areas or used for other projects without Kleinfelder's prior review.

1.2. PROJECT TEAM

The evaluation of this levee section is being performed by a team of consultants working under contract with SJAFCA. The consultant team is primed by Peterson Brustad, Inc. and consists of the following firms with their associated services provided to the District:

Peterson Brustad Inc. (PBI) – Civil Engineering
Kleinfelder – Geologic and Geotechnical Engineering
Kjeldsen, Sinnock, and Neudeck, Inc. (KSN) – Surveying

This report provides the results of the geotechnical components of the CFR 65.10 requirements for the Smith Canal CLOMR.

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2 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation was to explore subsurface conditions and perform an evaluation of the levee and subsurface geotechnical conditions in accordance with FEMA requirements for seepage, stability, and settlement as per 44 CFR Section 65.10.

Kleinfelder's scope of services was originally outlined in our contract and included the following:

- A review of available subsurface information;
- Geotechnical reconnaissance of the site;
- A geomorphic evaluation;
- Engineering analyses, including settlement, seepage, static slope stability, seismic evaluation; and
- Preparation of this report.

This report primarily addresses the geotechnical evaluation required for the Smith Canal CLOMR process.

The results of engineering analyses of the items not covered in this report will be presented by others in separate documents.

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3 BACKGROUND INFORMATION

3.1. GENERAL

This investigation was undertaken to assess the existing condition and variations in subsurface profile along the Dad's Point levee and to evaluate the performance of the levee during a design flood event (head differential). It should be noted that the San Joaquin River side of the levee is the waterside and the Smith Canal side is the landside. This characterization and analysis is a required component of FEMA's CLOMR levee certification compliance process, as governed by 44 CFR Section 65.10 of the National Flood Insurance Program (NFIP) regulations.

3.2. PREVIOUS INVESTIGATIONS, REPORTS, AND CONSTRUCTION DRAWINGS

As part of our levee evaluation, we reviewed other pertinent information that was available regarding the project site. This information was obtained in connection with our DWR Contract No. 4600008102 and included boring and CPT data, laboratory data, and historical aerial photographs of the area from the 1930's onward.

3.3. PROJECT DATUMS AND COORDINATE SYSTEM

Elevation references in this report are in feet and are based on the North American Vertical Datum of 1988 (NAVD88). Northing and easting coordinates are based on the California Coordinate System Zone III and the 1983 North American Datum (NAD83).

3.4. LEVEE TOPOGRAPHIC DRAWINGS AND LEVEE MILE REFERENCE

Transverse levee cross sections were supplied by KSN and use NAVD88 vertical datum. These cross sections along with a cross section map are presented on Plates D-1 through D-4 in Appendix D.

3.5. WATER SURFACE PROFILES

The 1-in-100 annual exceedance probability (1/100 AEP) water surface elevation for the subject levee was provided to us by KSN and is shown on the levee cross sections in

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Appendix D. We understand the 1/100 AEP is the appropriate design water surface profile for this project. In general, the 1/100 AEP water surface for the Dad's Point levee evaluated is Elevation +9.9 feet. However, in order to be conservative, for the purpose of our evaluation we have used a water elevation of +10.0 feet.

In this report where we describe a condition that either meets or does not meet the 1/100 AEP water surface condition, we are referring to the minimum acceptable geotechnical requirements for levees as contained in the U.S. Army Corps of Engineers (USACE) Levee Design Manual (EM 1110-2-1913) and Technical Letter Design Guidance for Levee Underseepage (ETL 1110-2-569). Our analyses considered steady-state seepage and slope stability for end-of-construction, rapid drawdown, 100-year flooding steady-state, and pseudostatic cases, as described in detail in subsequent sections. In this report the term "1/100 AEP" water level is used interchangeably with "100-year water surface elevation," or "100-year WSE."

Additionally, in our analysis an "inside" low water elevation was also used. The low water elevation used in our analysis was Elevation +2.0 feet. The low water elevation was based on discussions with PBI and KSN which indicated that an 8-foot head differential should be used for design which corresponds to a water elevation of +2.0 feet. This is the maximum head differential that could occur if the Smith Canal closure device was closed at low tide (2.0) and a 10.0 foot tide occurred in the San Joaquin River.

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4 FIELD INVESTIGATION

4.1. GENERAL

Our field investigation for the evaluation of the subject levee consisted of the following activities:

- Review of Existing Information
- Site Reconnaissance
- Geomorphology Study
- Field Exploration
- Geotechnical Laboratory Testing

These activities are described in greater detail below.

4.2. REVIEW OF EXISTING INFORMATION

As described in Section 3.2, we were able to obtain historical aerial photographs of the area since 1937 to assess site history over time. Specifically, we reviewed black and white historical aerial photographs dated 1937, 1940, 1957, and 1963.

4.2.1. Levee Construction History

Very little construction information regarding the Dad's Point levee is available. Based on review of the historical aerial photographs, it appears that the subject portion of the levee was constructed sometime prior to 1937.

4.2.2. Levee Past Performance

To our knowledge, no flooding, seepage, or instability at or near this levee section has ever been reported. This is not necessarily unexpected since the water level on either side of the levee has always been the same (no head differential).

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4.3. SITE RECONNAISSANCE

We performed a site reconnaissance of the Dad's Point levee on September 24, October 6, December 23, December 29, and December 30, 2009. To assist in our assessment of the existing conditions, we noted the following elements, if present, during our reconnaissance:

- Surface geology
- Evidence of levee settlement
- Evidence of erosion
- Evidence of excavation, tilling, piping, placement of utility poles, and other surface features
- Evidence of inadequate or poor levee maintenance
- Evidence of breaching, cracking, ruts, or depression
- Evidence of seepage or sand boils at landslide levee toe
- Accumulation of debris in the channel that may deflect floodwaters toward the channel bank
- Evidence of burrowing animals
- Vegetation on the levee and creek bottom

We photographed and tabulated those areas of the levee where these items were noted. A representative selection of items photographed is presented on Plates 5A through 5E.

The surface of the levee crest is landscaped with lawn, various trees, park benches, and a concrete walkway that meanders along the length of the levee. According to information provided by KSN, the levee height ranges from approximately 40 to 50 feet on the waterside and about 10 to 18 feet on the landside.

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A steel pipe was observed coming out of the waterside slope of the levee between Cross Sections D(1)-D(2) and E(1)-E(2) and extending into the San Joaquin River. The pipe could only be seen during low tides because the high tide elevations were greater than the elevation of the top of the pipe where it daylights out of the levee slope. According to City of Stockton records, this is an abandoned sanitary sewer pipe.

Other conditions observed included:

- Vegetation (trees, brush, and grass) and scattered riprap and concrete debris are present on the landside and waterside levee slope throughout the length of the levee.
- Levee crown is about 60 to 70 feet wide along the length of the alignment with the northern tip being about 30 feet wide.

4.4. GEOMORPHOLOGY REVIEW

We reviewed and compared historic topographic maps with current topographic maps, aerial photographs, and geologic and soil maps in order to identify landscape features that could impact the stability of the existing levee system and that could be used to assist us in locating future exploration points described in the subsequent field exploration section of this report. Such features include meandering stream channels, natural and artificial levees, borrow pits, marsh areas, and springs.

For this project, selected historical and current topographic maps and aerial photographs covering the project area were reviewed to identify such features. Documents reviewed included black and white historical aerial photographs dated 1937, 1940, 1952, 1957, and 1963 and USGS topographic maps dated 1913 and 1987 of the Stockton 7.5-minute and Stockton West 7.5-minute quadrangles, respectively. We also reviewed regional and local geologic maps (see Plate 3). The results of our review are discussed in Section 5 of this report.

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4.5. FIELD EXPLORATION

As part of our State of California Department of Water Resources (DWR) levee evaluation contract (No. 4600008102), we performed a field exploration program to evaluate subsurface conditions along the subject levees. Our field exploration program consisted of two CPTs and two borings.

Following our DWR protocol, our field program consisted of performing two exploration points along the 1,600 feet of levee. Each CPT was located adjacent a boring to confirm and calibrate the results obtained. All explorations were located in the middle of the levee alignment.

To supplement the information gathered as part of our DWR project, three additional borings were drilled through the levee crown. These borings were drilled at locations between the previous exploration locations for the primary purpose of performing vane shear testing.

Prior to performing or drilling the CPTs and borings, permits were obtained from the San Joaquin County Environmental Heath Department (SJCEHD). Also, Underground Service Alert (USA) was notified at least two working days prior to performing the explorations.

CPT and boring locations were surveyed by KSN or Kleinfelder. Latitude and longitude coordinates of our boring and CPT locations were converted to northing and easting coordinates based on the California Coordinate System Zone III (1983 North American Datum). The elevations were based on the NAVD88 Vertical Datum. The approximate ground surface elevation for the borings drilled as part of our DWR work is shown on the boring logs included in Appendix A. Plan locations of CPTs and borings are shown on Plate 2. Details of the CPTs and borings performed are discussed below.

4.5.1. Cone Penetration Tests (CPTs)

We explored the site by CPT at two locations along the subject levee alignment. CPTs WR0828_010C and WR0828_011C were pushed on January 21, 2009 on the levee crest to depths of about 49.4 and 79.3 feet, respectively. The CPTs were performed by

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Nadatek and Direct Sensing, Inc. (NDS) of Richmond, California. CPT logs are included in Appendix B and the locations are shown on Plate 2.

NDS used a Vertek Digital 1.75 CPTU cone with seismic capacity along with a 30-ton push capacity, truck-mounted platform. The Vertek equipment meets the ASTM D 5778 Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils. The cones have 10 square centimeters (cm^2) tips and 150 cm^2 friction sleeves and include a porous filter and pressure sensor. The cone and porous filter are saturated under vacuum with glycerin to promote rapid equilibration with in situ pore pressures. Cones are advanced at the ASTM standard rate of two cm/second. Baseline readings are performed both before and after each push to determine temperature and load cell drift. The cone measures bearing, friction sleeve, and dynamic pore pressure at two cm intervals and this data is plotted in real-time and recorded on a laptop computer adjacent to the push platform. Holes were grouted upon completion of each push with cement grout in general accordance with SJCEHD and DWR criteria.

Soil behavior type (SBT), SPT N60 energy ratio, undrained shear strength, and unit weights are calculated and/or are interpretations generated by the CPT-Pro software based on algorithms presented in Robertson et al. (1986), Robertson (1990) and Lunne et al. (1997).

4.5.2. Exploratory Borings

We further explored the site by drilling two soil borings (WR0828_001B and WR0828_010B) along the levee alignment. The borings were drilled on March 25 and 26, 2009 on the levee crest to a depth of 100 feet. Borings WR0828_001B and WR0828_002B were drilled by Neil O. Anderson & Associates of Lodi, California. Neil O. Anderson drilled borings WR0828_001B and WR0828_002B using a truck-mounted CME-75 drill rig and both 10-inch diameter hollow-stem auger and rotary wash equipment.

Three additional borings (B-1, B-2, and B-3) were also drilled on the levee alignment through the crest. The borings were drilled on December 22, 2009 to depths ranging

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from 12 to 21½ feet. Borings B-1, B-2, and B-3 were drilled by Precision Drilling of Stockton, California. Precision drilled the borings using a truck-mounted CME 850 drill rig with 8-inch diameter hollow-stem auger equipment.

An engineer with Kleinfelder maintained a log of the borings, visually identified and classified soils encountered in general accordance with ASTM Standard Practice D 2488 (see Plate A-1 in Appendix A), and obtained representative samples of the subsurface materials.

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- Standard Penetration Split Spoon Sampler (2.0-inch O.D., 1.4-inch I.D.)
- Punch Core Sampler (2.5-inch O.D.)
- Thin walled Shelby Sampler (2.5-inch O.D.)
- Bag Samples (1-gallon plastic bag)

The Standard Penetration Test (SPT) sampler was driven 18 inches (unless otherwise noted) into undisturbed soil using a 30-inch drop of a 140 pound calibrated trip-hammer. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported as uncorrected blow counts on the logs. The punch-core sampler consisted of a wireline system with a 5-foot long core barrel that was used to core through the soil. The sampler was advanced ahead of the cutting head when coring.

Down-hole field vane shear tests were also performed in borings B-1, B-2, and B-3 in accordance with ASTM D 2573. The vane shear tests were performed using 2½ and 3.625 inch vanes in order to determine the undrained shear strength of the subsurface soils.

Upon completion, the 100-foot borings were backfilled with cement grout in general accordance with SJCEHD and DWR criteria, and the excess cuttings and drilling fluid were retained in drums and disposed of off site. Borings B-1, B-2, and B-3 were backfilled to approximately 3 to 5 feet below the ground surface with bentonite hole plug

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and the holes capped with soil cuttings in accordance with SJCEHD criteria. The remaining soil cuttings were spread onsite.

Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and brought to our Kleinfelder Stockton office for laboratory testing.

Upon completion of laboratory testing (See Section 4.6.1), soil classifications were evaluated in general accordance with ASTM Standard Practice D 2487 and are presented on the Log of Borings. A key to the Log of Borings is presented on Plate A-2 in Appendix A. The Logs of Borings are presented on Plates A-3 through A-7 in Appendix A.

4.6. GEOTECHNICAL AND CHEMICAL LABORATORY TESTING

4.6.1. Geotechnical Laboratory Testing

Representative samples obtained from the exploration boring program were tested at our Kleinfelder Laboratory in Stockton, California. The following tests with their respective ASTM designations were performed for the subject site.

- Moisture (ASTM D 2216)
- Organic Content (ASTM D 2974)
- Atterberg Limits (ASTM D 4318)
- Sieve (ASTM D 422)
- Material Finer Than a 0.75-mm (No. 200) Sieve (ASTM D 1140)
- Specific Gravity (ASTM D 854)
- Unconfined Compression (ASTM D 2166)
- One-Dimensional Consolidation (ASTM D 2435)

The results of the geotechnical laboratory tests are shown on the boring log in Appendix A. Detailed laboratory results of the tests are presented in Appendix C. We note that the DWR sampling protocol, while mostly continuous, relies primarily on SPT and punch core techniques which reduce the number of undisturbed samples available for strength testing.

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5 GEOLOGY AND GEOMORPHOLOGY

5.1. GEOLOGIC SETTING

The study area lies within the central portion of the Great Valley geomorphic province. The province is bordered to the north by the Cascade Range and Klamath Mountains, to the west by the structurally complex sedimentary and volcanic rock units of the Coast Ranges, to the east by the granitic and metamorphic basement rocks which form the gently sloping western foothills of the Sierra Nevada Mountains, and to the south by the east-west trending Transverse Ranges. About 645 km long and 80 km wide, the Great Valley is an asymmetrical, synclinal trough formed by tilting of the Sierran block during the late Tertiary and Quaternary periods with the western side dropping to form the valley and the eastern side uplifting to form the Sierra Nevada Mountains.

The Great Valley is subdivided into the Sacramento Valley to the north and the San Joaquin Valley to the south; the Sacramento Valley is drained by the south-flowing Sacramento River and the San Joaquin Valley is drained by the generally north-flowing San Joaquin River. The two rivers meet at the Sacramento-San Joaquin Delta, which empties into San Francisco Bay, ultimately connecting with the Pacific Ocean via the Golden Gate.

Within the project area, erosion of the adjacent Sierra Nevada Mountains and Coast Ranges has in-filled the Great Valley with a thick sequence of unconsolidated to semi-consolidated Quaternary (Pleistocene and Holocene) age alluvial, basin, and delta plain sediments deposited by the Sacramento and San Joaquin rivers and their tributaries. The thickness of the valley sediments varies from a thin veneer at the edges of the valley to thousands of meters in the western portion. The bedrock complex is likely composed of metamorphosed marine sediments similar to those found in the foothills of the western Sierra Nevada Mountains and the core of the Coast Ranges.

The geology of this area has been mapped by several geologists including Atwater (1982), Wagner et al. (1991 and 1987), and Knudsen and Lettis (1997). A few of these maps are regional maps containing generalized compilations of mapping efforts by other geologists. The general geologic conditions of this area are depicted on Plate 3, which is a portion of the geologic map by Knudsen and Lettis (1997). According to the

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Knudsen and Lettis map, the subject levee system lies atop artificial fill/dredge spoils, undivided (afds). This material consists largely of soils dredged from the San Joaquin River. However, Dad's Point appears to be a remnant of a portion of Rough and Ready Island that was cut off in the early 1900's when the Stockton Deep Water Channel was excavated. Prior to the dredge spoils, this location was likely underlain by Late Holocene alluvial floodplain deposits, undivided (Qhfp) which is described by Knudsen and Lettis as deposits of abandoned oxbows, channels and interdistributary basins, flood basins and basin rims, distal alluvial fans and low natural levees.

5.2. REGIONAL GROUNDWATER

The California Department of Water Resources (2003) identifies this area as the Eastern San Joaquin Groundwater Subbasin, which is one of several groundwater subbasins within the San Joaquin Valley Groundwater Basin. The Eastern San Joaquin Subbasin is defined by the aerial extent of unconsolidated to semiconsolidated sedimentary deposits that are bounded by the Mokelumne River on the north and northwest; the San Joaquin River on the west; the Stanislaus River on the south; and the consolidated bedrock on the east. This basin is underlain by more than 1,000 feet of unconsolidated Quaternary age sediments (Department of Water Resources, 2003).

Groundwater depths in the area surrounding the subject levee vary from 3 feet to around 17 feet based on historical groundwater data compiled by the Department of Water Resources (2009). During our field explorations, groundwater was encountered at a depth of approximately 17 feet below ground surface.

5.3. HISTORIC GEOMORPHOLOGY

Dad's Point is a man-made peninsula on the San Joaquin River at the mouth of Smith Canal. It was created in the early 1900's when the San Joaquin River was realigned during the excavation of the Stockton Deep Water Channel. Dad's Point appears to be a remnant of a portion of Rough and Ready Island that was cut off when the Stockton Deep Water Channel was excavated. The peninsula is now part of Louis Park. Smith Canal first appears on a Corps of Engineers map dated 1898. It appears to be a man-made channel that likely collected natural and irrigation drainage, and diverted it to the San Joaquin River.

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Review and comparison of historic topographic maps with current topographic maps, aerial photographs, and geologic and soil maps can be used to identify landscape features that could impact the stability of the existing levee system. Such features include meandering stream channels, natural and artificial levees, borrow pits, marsh areas, and springs. For this project, selected historical and current topographic maps and aerial photographs covering the project area were reviewed to identify such features. Documents reviewed included black and white historical aerial photographs dated 1937, 1940, 1952, 1957, and 1963, and USGS topographic maps dated 1913, and 1987 of the Stockton 7.5-minute and Stockton West 7.5-minute quadrangles, respectively. The following are the observations made during our review:

- Based on review of the historical aerial photographs and topographic maps, it appears that Dad's Point was constructed between 1913 and 1937. The channel does not appear on the 1913 USGS topographic map, but is on the 1937 aerial photos.

- The subject levee was constructed on hydraulic dredge spoils within alluvial flood plain deposits, near the distal edge of the alluvial fan associated with the Modesto Formation. This area was relatively flat prior to construction.

- This portion of levee was originally an inland portion of Rough and Ready Island, before the realignment of the San Joaquin River and excavation of the Stockton Deep Water Channel.

- The documents reviewed did not indicate the presence of a paleochannel or marsh underlying the subject levee.

- Louis Park appears to have occupied the site since at least 1937.

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6 SEISMICITY

6.1. SEISMIC SETTING

The site is located in a region traditionally characterized by minimal seismic activity and few active faults. Therefore, during the project design life[1], the project site will likely experience minor earthquakes and possibly a major earthquake (Moment magnitude greater than 7.0) from one or more of the adjacent active faults. The nearest mapped fault is the seismically inactive Stockton fault. The south-westward trending Stockton fault has been mapped to pass approximately 6½ kilometers to the southeast of the levee, and is shown as a concealed, Pleistocene fault on the 1994 California State Fault map (Jennings, 1994). For major active faults, the distance from the site and the estimated maximum moment magnitude are summarized in Table 6-1.

Table 6-1. Regional Faults and Seismicity

Fault Name	Distance from Site (km)	Direction from Site	Maximum Moment Magnitude
Great Valley (segment 7)	30	South	6.7
Great Valley (segment 5)	38	Northwest	6.5
Greenville (GN)	40	West	6.6
Greenville (GN + GS)	40	West	6.9
Greenville (GS)	41	West	6.6
Mount Diablo	42	West	6.6
Foothills Fault System (Bear Mountain Fault)	55	East	6.5
Concord-Green Valley (CON)	56	West	6.2
Concord-Green Valley (CON + GVN + GVS)	56	West	6.7
Concord-Green Valley (CON + GVS)	56	West	6.6
Calaveras (CC + CN)	58	Southwest	6.9
Calaveras (CN)	58	Southwest	6.8
Calaveras (CN + CC + CS)	58	Southwest	7.0
Great Valley (segment 8)	61	South	6.6

[1] *Design life is assumed at 50 years.*

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Fault Name	Distance from Site (km)	Direction from Site	Maximum Moment Magnitude
Great Valley (segment 4)	61	Northwest	6.6
Concord-Green Valley (GVS)	64	West	6.2
Concord-Green Valley (GVN + GVS)	64	West	6.5
Foothills Fault System (Melones Fault Zone)	64	East	6.5
Hayward-Rodgers Creek (SH)	69	West	6.7
Hayward-Rodgers Creek (SH + NH)	69	West	6.9
Hayward-Rodgers Creek (SH + NH + RC)	69	West	7.2
Calaveras (CC)	70	Southwest	6.2
Calaveras (CS + CC)	70	Southwest	6.4
Ortigalita	77	South	7.1
Hayward-Rodgers Creek (NH)	77	West	6.4
Hayward-Rodgers Creek (NH + RC)	77	West	7.1
Concord-Green Valley (GVN)	77	West	6.2
West Napa	82	Northwest	6.5
Hunting Creek-Berryessa	92	Northwest	7.1
Monet Vista-Shannon	93	Southwest	6.7
Hayward-Rodgers Creek (RC)	96	West	7.0
Great Valley (segment 3)	98	Northwest	6.9
San Andreas (SAP)	100	West	7.1
San Andreas (SAP + SAN + SAO)	100	West	7.8
San Andreas (SAS + SAP)	100	West	7.4
San Andreas (SAS + SAP + SAN)	100	West	7.7
San Andreas (SAS + SAP + SAN + SAO)	100	West	7.9
Great Valley (segment 9)	100	South	6.6

6.2. HISTORICAL SEISMICITY

The project site and vicinity are located in an area characterized by low to moderate seismicity. A number of earthquakes have occurred within the site vicinity during historic time (since 1800). Some of the significant regional earthquake events include the 1866

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(M6.0) West San Joaquin Valley earthquake, located approximately 51 km to the south of the site; the 1868 (M7.0) Hayward earthquake, located about 72 km to the west; the 1881 (M6.0) West San Joaquin Valley earthquake, located about 62 km to the south; and the 1980 (M5.8) Livermore earthquake, located approximately 40 km to the west. Other significant regional earthquakes include the 1858 (M6.3) San Jose earthquake, the 1889 (M6.3) Antioch earthquake, and the 1911 (M6.6) Calaveras fault earthquake.

The database used in our historical earthquake search contains in excess of 5,500 seismic events and covers the period from 1800 through January 2010. The earthquake database is primarily comprised of an earthquake catalog for the State of California prepared by the California Geological Survey. The catalog contains earthquake records from January 1, 1900 through December 31, 1974. Updates prepared by the CGS in 1979 and 1982 extend the coverage through 1982. In addition to the CGS updates, the data for earthquakes that occurred during the period between 1910 through January 2010 has been obtained from a composite catalog by the Advanced National Seismic System (ANSS). The ANSS catalog is a worldwide earthquake catalog which is created by merging the master earthquake catalogs from contributing ANSS member networks and then removing duplicate events, or non-unique solutions from the same event. The ANSS network includes the Northern and Southern California Seismic Networks, the Pacific Northwest Seismic Network, the University of Nevada, Reno Seismic Network, the University of Utah Seismographic Stations, and the United States National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900 from Seeburger and Bolt (1976) and Toppozada et al. (1978, 1981). In addition, we have also used the data from DMG Map Sheet 49 (Toppozada et al., 2000).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through January 2010), and magnitudes ($M \geq 4$). A summary of the results of the historical search is presented below.

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Table 6-2. Summary of Historical Earthquake Search

Time Period (1800 to January 2010)	210+ years
Maximum Magnitude*	7.0
Approximate distance to nearest historical M ≥ 4 earthquake	40 km
Number of events exceeding magnitude 4 within search area	126

*Moment magnitude

6.3. FUTURE EARTHQUAKE PREDICTION

In 2007, the Working Group on California Earthquake Probabilities (WGCEP 2007) at the U.S. Geologic Survey (USGS) predicted a 63 percent or larger probability of a magnitude 6.7 or greater earthquake occurring in the adjacent San Francisco Bay Area in the next 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 6-3.

Table 6-3. 30-Year Probability of a Magnitude 6.7 or Greater Earthquake by WGCEP (2007)

Fault	Mean Probability (Percent)	Min – Max Probability (Percent)
Hayward – Rodgers Creek	31	12 – 67
N. San Andreas	21	6 - 39
Calaveras	7	1 – 22
San Gregorio	6	4 – 9
Concord – Green Valley	3	1 – 6
Greenville	3	2 – 4
Mount Diablo Thrust	1	0 – 1

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7 SURFACE AND SUBSURFACE CONDITIONS

7.1. GENERAL

The surface and subsurface conditions described below have been lumped into one reach for the entire length of the subject levee. The single reach was determined primarily based on:

1) Little variation noted in geomorphology
2) Geology consistent in the study area
3) Adequate levee geometry and topographic features
4) Geotechnical interpretation of subsurface conditions.

The reach evaluated was from the southern end of the Dad's Point Levee where a sheet pile wall is planned to be installed due to freeboard concerns, to the northern end where another sheet pile wall is planned to be installed as part of the proposed closure structure. The northern sheet pile wall should extend through the levee from the closure structure to where the levee has at least 60 feet of crown width. The distance between the two sheet pile walls is unknown at this time but is less than the 1,600 feet of total levee length.

7.2. REACH 1

Based on the perpendicular cross sections provided by KSN, the levee height in Reach 1 varies from approximately 40 to 50 feet on the waterside (San Joaquin River side) and about 10 to 18 feet on the landside (Smith Canal side). The levee crown elevation is at approximate elevation +10 to +17 feet. Landside levee toe elevations range between approximate elevations -2 and +5 feet. The BFE shown on the plates prepared by KSN is +9.9 feet so the maximum head above the landside toe is approximately 11.9 feet. However, for our elevation we used a design water elevation of +10.0 so the maximum head above the landside toe in our analysis is 12 feet. The levee crest width is approximately 60 to 70 feet with the northern tip being about 30 feet wide. The landside and waterside slopes are typically between 1(horizontal):1(vertical) and 2:1.

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Based on the soils encountered in Kleinfelder's subsurface exploration programs, the subsurface conditions along the levee alignment in Reach 1 generally consisted of lean to fat clay and silt to depths of about 11 to 20 feet below the levee crown underlain by organic silt to silt to depths of about 20 to 25 feet and lean clay to a depth of about 35 feet. The near-surface, fine-grained soils were underlain by sandy silt, silty sand, and poorly-graded sand with interbedded lean clay layers to the depths explored.

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8 GEOTECHNICAL EVALUATION

8.1. GENERAL

Our geotechnical evaluation consisted of assessing the following, as per 44 CFR Section 65.10:

- Settlement
- Through-Seepage
- Underseepage
- Static Slope Stability
- Seismic Slope Stability, including liquefaction potential

The acceptance criteria we based our analyses on and the results of our analyses are described in the following sections.

8.2. ACCEPTANCE CRITERIA

The following section summarizes acceptance criteria for geotechnical evaluation based on the following references:

- USACE EM 1110-2-1913 "Design and Construction of Levees," dated April 30, 2000.

- USACE ETL 1110-2-569 "Design Guidance for Levee Underseepage," dated May 1, 2005.

- USACE SOP SPK EDG-03 "Geotechnical Levee Practice," Revision 2, dated April 11, 2008.

- Title 44 Code of Federal Regulations 65.10: Mapping of Areas Protected by Levee Systems.

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8.2.1. Levee Cross Section Geometry

Per SPK EDG-03 (USACE Sacramento District's reference), the minimum cross section for new levees should have a 3:1 waterside slope and a 3:1 landside slope. Existing levees with landside slopes as steep as 2:1 may be used in rehabilitation projects if past landside slope performance has been acceptable. Increasing the height of an existing levee to meet the 100-year plus freeboard water surface would be considered a rehabilitation project. Easements are necessary for maintenance, inspection, and floodfight access.

Per USACE EM 1110-2-1913, a minimum 2:1 slope is required for both landside and waterside slopes. Both SPK EDG-03 criteria and USACE EM 1110-2-1913 are provided in Table 8-1.

Table 8-1. Minimum Levee Cross Section Geometry

Dimension	Criteria per SPK EDG - 03		Per EM 1110-2-1913
	Existing Levee	New Levee	
Levee Crown Width (feet)	20* / 12**	20* / 12**	10
Landside Levee Slope	2:1 minimum	3:1 minimum	2:1 minimum
Waterside Levee Slope	3:1 minimum	3:1 minimum	2:1 minimum

* A minimum 20-foot wide crown for main line levees, major tributary levees, and bypass levees.
** A minimum 12-foot wide crown for minor tributary levees.

8.2.2. Design Water Surface Elevation

For the purpose of our analyses, a water elevation equal to the 100-year WSE was selected as the design water surface elevation WSE for both the steady-state seepage and slope stability analyses.

The design WSE for geotechnical evaluation was provided by KSN to Kleinfelder and is shown on the cross sections presented in Appendix D. The design WSE is labeled on the cross sections as being +9.9 feet. However, as discussed in Section 3.5, the

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design WSE used in our evaluation was +10.0 feet in order to provide a slightly more conservative evaluation of the subject levee.

8.2.3. Freeboard Criteria

Per 44 CFR Section 65.10, the minimum freeboard requirements are presented as the following:

- For riverine levees,
    - A minimum freeboard of 3 feet above the base flood must be provided.
    - An additional 1 foot above the minimum is required within 100 feet on either side of structures riverward of the levee or wherever the flow is constricted.
    - An additional ½ foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.
    - Under no circumstances will freeboard of less than 2 feet be accepted.
- For coastal levees,
    - The freeboard must be established at 1 foot above the height of the 1-percent-annual-change wave or the maximum wave runup (whichever is greater) associated with the 1-percent-annual-change stillwater surge elevation at the site.
    - Under no circumstances will a freeboard of less than 2 feet above the 1-percent-annual-change stillwater surge elevation be accepted.

For this project located within a riverine environment, the criteria have been selected as 100-year WSE plus 3 feet of freeboard.

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8.2.4. Underseepage Criteria

Levees constructed on low permeability foundation soil (silt and clay) underlain by a higher permeability layer (sand and gravel) are susceptible to piping and failure due to underseepage during periods of high river stage. Under these conditions, seepage travels horizontally under the levee through the pervious layer with relatively little piezometric head loss. At the landside levee toe, seepage is driven vertically upward through the low permeability foundation layer (blanket) due to the high total head at the bottom of the blanket. Failure of the blanket can occur by either uplift where the blanket materials are nearly impervious and do not have enough weight to resist the pressure on the bottom of the blanket or by piping through a blanket consisting of low to non-plastic erodible soils. This condition can exist with as little as one order of magnitude difference between the permeabilities of the blanket layer and the underlying more pervious layer.

The risk of uncontrolled underseepage that could lead to failure of a levee increases as the vertical seepage gradient through the landside confining blanket layer increases. It is customary to calculate the exit gradient (average vertical gradient) through the confining blanket layer as the head loss through the blanket divided by the blanket layer thickness. Calculations can be performed in accordance with blanket theory as presented in the USACE Levee Design Manual (EM-1110-2-1913) or using total head contours from finite element (FE) analysis programs[2].

Based on extensive evaluations of past levee performance, the USACE has developed criteria for acceptable exit gradients. In general, levees performed well when exit gradients (average vertical gradients through a confining blanket) are less than the prescribed criteria. A summary of USACE exit gradient criteria used in this geotechnical evaluation is presented in the following table.

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[2] *Additional underseepage references include CESPK-ED-G SOP EDG-03 (Geotechnical Levee Practice, June 28, 2004) and ETL 1110-2-569 (Design Guidance for Levee Underseepage, May 1, 2005).*

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Table 8-2. Allowable Exit Gradients

Location	Allowable Exit Gradients**
Landside toe of levee	<0.5
Bottom of empty ditch at landside toe*	<0.5
Bottom of empty ditch 150 feet landward of toe*	<0.8
Ditch between landside toe and 150 feet landward of toe*	Interpolate

* Reference USACE EM 1110-2-1913, Section 8-16, 'Ditches Landside of Levee'
** Allowable exit gradients are applicable for 100-year WSE in this report. Assumes a minimum saturated soil unit weight = 112 pcf.

8.2.5. Through-Seepage Criteria

Seepage through a levee embankment can occur during periods of high river stage. Depending upon the duration of high water and the permeability of embankment soil, seepage may exit at the face of the levee by passing directly through pervious layers in the levee. Under these conditions the stability of the landside levee slope may be reduced.

In general, any time a seepage model indicates seepage on the face of the levee a steady state slope stability analysis should be performed to determine the influence of through-seepage on slope stability. The acceptance criteria for through-seepage are either no seepage on the face of the levee or a factor of safety (FOS) of at least 1.4 against slope failure under steady state seepage conditions (for levees built of clay). However, it is unacceptable if the phreatic surface exits through the landslide slope of the levee if it contains erodible materials like silts and sandy silts. For sand levees, a 5:1 landside slope is considered flat enough to prevent damage from seepage on the landside slope.

8.2.6. Factor of Safety Criteria for Slope Stability Evaluation

The USACE Engineering Manual EM 1110-2-1913 identifies four types of loading conditions to be evaluated for slope stability analyses:

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Case I: End of Construction: This case addresses slope stability at the end of construction of a new levee or the mitigation for an existing levee requiring a minimum FOS of 1.3. This case represents undrained conditions for impervious levee embankments and foundation soils. Excess pore pressures are present because the low permeability soil has not had time to drain since being loaded. This loading condition assumes the construction will be completed in one stage.

Case II: Rapid Drawdown: This case represents the condition where the water level (100-year WSE used in this study) saturates a majority of the waterside portion of the embankment, and then falls faster than the soil can drain. The FOS must be greater than 1.0 (short duration flood stage) to 1.2 (long duration flood stage). For locations where levee fill is comprised of sand and silty sand, we believe pore pressures will dissipate and a rapid drawdown analysis is not required, per EM 1110-2-1902.

Case III: Steady-State Seepage from Full Flood Stage: This condition occurs when the water level remains at or near flood stage (100-year WSE used in this study), thus fully saturating the levee embankment soils, and a steady-state seepage phreatic surface develops. The FOS for this case must be greater than 1.4.

Case IV: Earthquake: This case represents a preliminary screening analysis not addressed in detail in EM 1110-2-1913. Similar to the current DWR interim levee design criteria for urban and urbanizing levees, we have based our study on a 200-year return period earthquake event/motion. A typical winter water level was used in our analyses to conservatively assess if the levee embankment might be susceptible to large deformations during the design seismic event. The typical winter water level was assumed to be +2.0 (NAVD 88) which has been used in the past in the site area and was also estimated during our site visits. The minimum required FOS for these loading conditions are summarized in Table 8-3.

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Table 8-3. Minimum Required Slope Stability Factor of Safety

Case	Minimum Factor of Safety
Case I – End of Construction*	1.3
Case II – Rapid Drawdown*	1.0 to 1.2
Case III – Steady-State Seepage*	1.4
Case IV – Earthquake	Check for liquefaction & deformation

* FOS criteria from USACE EM 1110-2-1913.

8.3. FREEBOARD AND SETTLEMENT ANALYSIS

8.3.1. General

According to freeboard criteria (as detailed in Section 8.2.2), the levee in this study requires 3 feet of freeboard above the 100-year WSE. Based on the cross section information provided by KSN, the existing Dad's Point levee along the San Joaquin River and Smith Canal meets this requirement with the exception of the eastern area near Cross Section A(1)-A(2). It is our understanding that a sheet pile wall is currently planned to be installed in the low area with a top elevation at least equal to the design freeboard elevation in order to meet the required freeboard criteria. The cross sections in Appendix D present the existing topography of levee and 100-year WSE for visual comparison.

Given that the existing levee either meets the freeboard requirement or will have a sheet pile wall installed and no new fill will be placed, settlement due to loading from a levee raise was not considered in this evaluation. Weight of new levee fill would increase the stress on the underlying soils, causing settlement. The magnitude of settlement at the levee crown is a common settlement analysis scenario for levee raises. If required in the future, settlement should be evaluated based on the components identified by classical geo-mechanics: elastic and consolidation settlements. Elastic settlement, commonly called immediate settlement, typically applies to compression of cohesionless soils (sands and gravels) experiencing negligible pore pressure change with stress change. Consolidation settlement typically

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applies to cohesive soils (silts and clays) that compress in response to the dissipation of pore pressures over a relatively long period of time.

8.3.2. Analysis Results

As previously mentioned, the existing levee either meets the freeboard requirement or will have a sheet pile wall installed within the Reach 1 area. No new fill is planned. Accordingly, we did not perform a settlement analysis because there are no planned levee modifications at this time that would cause any consolidation or elastic settlement of the subsurface soils. The remaining analyses (seepage, slope stability, and seismic evaluation) were all conducted based on the existing levee crown elevation.

8.4. SEEPAGE ANALYSES

8.4.1. Study Cases

A generalized cross section representative of the subsurface conditions and levee section geometry along Reach 1 of Dad's Point was developed and evaluated:

- Reach 1– Cross section at Section C(1)-C(2) for seepage and stability analyses.

The cross section at Section C(1)-C(2) was selected because of the six cross sections prepared by KSN, Section C(1)-C(2) had the tallest landside slope and the toe at the lowest elevation. The cross section also has slopes on both the landside and waterside which are comparable to the rest of the reach.

8.4.2. Seepage Analysis Details

Levee profiles have been used to develop cross sections with generalized subsurface soil layering and material properties representative of each reach. The generalized cross section may not represent specific borings at the cross section location but represent critical features such as bathymetry, thin blanket layer, elevation of landside toe and ground surface 100 feet or more landward, or thick layers of clean sand within the specific reach.

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A summary of the permeability and anisotropy values used in the seepage analyses are presented in Table 8-4. Further discussion of the selection of these permeability values is discussed in Appendix E.

The coefficients of permeability given in Table 8-4 represent the anticipated properties of the various materials under saturated conditions. These values are selected based upon:

- Unified Soil Classification System and published empirical relationship between soil type and the hydraulic conductivity, such as those presented by Terzaghi and Peck (1967), and
- Laboratory gradation analysis results (sieve analysis) and Atterberg Limits testing.

Anisotropy values selected for our seepage analyses are shown in Table 8-4.

Table 8-4. Summary of Seepage Parameters

Material No. In Seep/W Model	Soil Layer		Horizontal Permeability (K_h)		Anisotropy Ratio (K_v/K_h)
			cm/sec	ft/day	
1	Fat Clay	CH	$1x10^{-5}$	0.028	1:4
2	Silt	ML	$1x10^{-4}$	0.28	1:4
3	Organic Silt/Fat Clay	OH/CH	$1x10^{-5}$	0.028	1:4
4	Lean Clay	CL	$1x10^{-5}$	0.028	1:4
5	Sandy Silt	ML	$1x10^{-4}$	0.28	1:4
6	Sandy Lean Clay	CL	$1x10^{-5}$	0.028	1:4
7	Silty Sand	SM	$4x10^{-4}$	1.12	1:4

The permeability of materials typically decreases as the degree of saturation decreases. Hydraulic functions that relate unsaturated permeability to soil moisture suction and percent saturation were used to model the impact of saturation on permeability.

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Steady-state seepage analyses were completed using SEEP/W finite element program SEEP/W (version 7.12). This software was developed by GEO-SLOPE International, Ltd (2007). Average vertical gradients through the blanket layers (exit gradient) at the landside toe were estimated for a 100-year flood event used in this evaluation.

8.4.3. Seepage Model Boundary Conditions

Fixed-head boundary conditions set to the 100-year WSE were applied to the vertical waterside edge, the boundary nodes of the waterside river bottom, and waterside slope of the levee. This assumes potential blanket layers beneath the river may be penetrated by scour. Nodes along the bottom of the model were modeled as a no flow boundary (zero total flux boundary condition). A fixed-head boundary condition set to the normal water surface elevation was used along the vertical landside edge, the boundary nodes on the landside river bottom, and landside slope of the levee. To reduce the impact of the assumed boundary conditions on the seepage analysis results, the models were extended approximately 2,000 feet landward from the centerline of the levee and about 700 feet from the centerline on the waterside (to represent the center of the adjacent San Joaquin River). The elements on the top of the model extending from the landside levee hinge point to the normal water elevation on the landside slope are modeled as a potential seepage surface. These nodes are assigned a zero total flux boundary condition that is automatically adjusted by the computer program to a constant head boundary based on the iterative results of successive finite element runs. The calculated pressure head at each node is compared to the elevation head for each iteration. If the pressure head is positive at the node, the node becomes a constant head node with head equal to the ground surface elevation, thus allowing water to seep from the surface.

8.4.4. Seepage Analysis Results

The results of the seepage analysis for the 100-year WSE are shown graphically in Appendix E. These plates show the model geometry, material properties, estimated steady-state phreatic surface, and computed total head and vertical gradient contours. Exit gradients calculated based on the computed total head contours are also shown. If

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near surface soils were moderately permeable but were underlain by a low permeability layer, a FOS against uplift would have been calculated in lieu of an exit gradient.

Table 8-5 summarizes the results of the seepage analyses for the selected cross section. Based on the results of our seepage analyses, the proposed levee meets the criteria for underseepage (i.e., $i < 0.5$). Because the levee fill is composed of clay and a head differential on the levee will only last the duration of high water events, through seepage is not considered a concern.

Table 8-5. Summary of Seepage Analysis

Reach	Section	Exit Gradient (i)
1	C	0.21

8.5. STATIC SLOPE STABILITY ANALYSIS

8.5.1. Slope Stability Analysis Details

The USACE Manual EM 1110-2-1913 indicates four types of loading conditions for slope stability analysis: Case I - End of Construction, Case II - Rapid Drawdown, Case III - Steady-State Seepage at Full Flood Stage, and Case IV - Earthquake. This section discusses Cases II through III. Case I was not considered as part of this evaluation because no levee modifications are currently planned other than a sheet pile wall. Stability analyses for Case IV loading conditions will be presented and discussed later in Section 8.6, Seismic Evaluation.

Slope stability analyses were performed to evaluate the global stability of the levee embankments. Design parameters input into the slope stability models included the existing levee geometry, seepage analysis phreatic surfaces, the approximate soil unit weight, and shear strength properties for the native and levee fill soils. As discussed further in Appendix F, drained and undrained strength parameters were developed using correlations from SPT $(N_1)_{60}$ blow counts, CPT results, field vane shear test results, and laboratory testing results.

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For the purpose of levee safety, shallow failure surfaces within the landside levee slope that do not impact the levee crest are judged to be levee maintenance concerns and do not affect levee safety. For the purposes of this analysis, a depth of 5 feet was selected as the limiting depth for maintenance concerns. Shallow failures less than 5 feet in height are not addressed in this geotechnical evaluation.

Material properties of the levee and underlying soil were assessed based on the results of our field and laboratory testing program. A summary of material properties used in the stability analyses is presented in Table F-1 of Appendix F.

Slope stability analyses were conducted using the limit equilibrium software program SLOPE/W (version 7.12), a component of the GeoStudio 2007 suite. The FOS against landside slope failure was calculated using Spencer's method ("entry and exit" search routine) and pore water pressures computed by SEEP/W. Spencer's method is a two-dimensional, limit-equilibrium method that satisfies force equilibrium of slices and overall moment equilibrium of the potential sliding mass. The inclination of side forces between vertical slices is assumed to be the same for all slices and is calculated along with the FOS.

This method uses the levee slope configuration, unit weight and shear strength properties of the levee and foundation materials, and boundary and internal forces due to water pressures. After a potential failure surface has been assumed, the soil mass located above the failure surface is divided into a series of vertical slices. Forces acting on each slice include the slice weight, the pore pressure, the effective normal force on the base, the mobilized shear force (including both cohesion and friction), and the horizontal side forces due to earth pressures.

The FOS is calculated by determining the ratio of the resisting forces (cohesion and friction along the failure surface) to the driving forces about the center of the assumed failure surface. The computer program was used to perform automatic searches of different potential failure surfaces that included tension cracks filled with water and to compute the lowest FOS corresponding to a critical failure surface for a particular condition.

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Pore water pressure distribution under steady-state seepage conditions as computed by the SEEP/W seepage analysis was used in the steady-state seepage slope stability analysis. For rapid drawdown slope stability analysis, two water tables were required: high and low water tables. The high water table was assumed to be the 100-year flood event (100-year WSE) and the low water table was assumed to be the normal water level.

8.5.2. Stability Analysis Results

The results of our analyses for the stability cases (Cases I through III) are provided graphically in Appendix F and summarized in Table 8-6. These plates also show the model geometry, material properties, and the estimated phreatic surface. The plots show a FOS map in which red indicates the lowest (most critical) FOS and blue indicates the highest FOS. The range of computed FOS is shown on each plate. A band of the same color indicates a zone of equal FOS.

Table 8-6. Summary of Slope Stability Analysis

Reach	Section	Analysis Condition	Factor of Safety	
			Waterside Rapid Drawdown (Case II)	Landside Steady-State Seepage (Case III)
1	C	Existing condition	1.40	1.33
1	C	Assuming Slope Fails and Scarp Is Flattened		1.29
1	C	Assuming Slope Is Flattened To 2:1 After Initial Failure		1.48

As previously mentioned, there are no plans to modify the levee which would require an End-of-Construction analysis (Case I). Accordingly, a Case I analysis was not included as part of this evaluation. As shown in Table 8-6, the levee meets the minimum FOS requirement for the Rapid Drawdown case (Case II).

The levee does not meet the requirement of the Steady-State Seepage case (Case III) with the existing conditions. Since the levee is 60 to 70 feet wide and the most likely

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slip surface will only take out about 10 to 20 feet of the levee crown, leaving 40 to 60 feet of crown, an additional steady-state stability analysis was performed assuming the most likely failure slope were to slough off. This model assumed not only that the most likely slip surface fails but also that the material, which will slough off, will buttress the bottom of the slope and the steep scarp is flattened to a 2:1 slope as part of the levee maintenance. This model still did not meet the minimum FOS criteria, so another steady-state stability analysis was performed which assumed that the whole slope, from the landside water surface elevation to the crown, was flattened to a 2:1 slope and the sloughed off material still buttressed the bottom of the slope. As shown in the table above, this analysis meets the minimum FOS criteria.

It should be mentioned that the failure of the landside slope is unlikely as shown by a calculated FOS of 1.3 and that, even if the slope were to fail, there would be no loss in flood protection because the design levee template would still be maintained within the remaining levee section. Accordingly, we are not suggesting any modifications to the existing slope. It is our opinion that levee maintenance should be performed to "dress up" the slope if localized failures occur.

8.6. SEISMIC EVALUATION

8.6.1. General

This section presents the results of our seismic evaluation of the Dad's Point levee including liquefaction potential and potential deformation of the levee under an earthquake event with a 200-year return period (A 22.1% probability of exceedance in 50 years). It should be noted that there are no formal published guidelines for seismic evaluation of levees. DWR has recently published a draft document entitled "Proposed Interim Levee Design Criteria for Urban and Urbanizing Area State-federal Project Levees" dated May 15, 2009 which indicates that for urban and urbanizing areas, 200-year ground motions are required for seismic assessments. However, DWR's document contains no details about methodology or specific design criteria in terms of liquefaction and/or acceptable/unacceptable levee deformations under seismic conditions. We also understand that USACE is developing a guidance document on this topic; however, no specific information is available at the time of this report.

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A summary of the methodology used for our seismic evaluation is presented in detail in Appendix G and is briefly described below.

8.6.2. General Methodology for Seismic Evaluation

An outline of the proposed general methodology for preliminary seismic evaluation of the levees is presented on Plate G-1 in Appendix G. The proposed methodology can be described as follows:

- Site-specific probabilistic seismic hazard (PSHA) and deaggregation analyses are performed to estimate PGA and associated magnitude of an earthquake having return period of 200 years per the recently published draft DWR document.
- Design groundwater level for liquefaction analyses is taken as the normal water surface level.
- Liquefaction analyses are performed using the estimated PGA and magnitude.
- If the FOS against liquefaction is less than 1.0 for a soil layer, a post-earthquake static slope stability analysis is performed using undrained residual shear strength for the potentially liquefiable layer on a representative cross section. If the post-earthquake static slope stability analysis yields a FOS greater than 1.0, then a pseudo-static slope stability analysis is performed to estimate the yield acceleration (k_y). A post-earthquake static FOS of less than 1.0 or pseudo-static k_y less than or equal to 0.5 times the PGA (i.e. $k_y \leq 0.5PGA$) indicates significant deformations.
- Based on the results of the liquefaction and post-earthquake static and/or pseudo-static slope stability analyses, a cross section is classified into one of the two likely scenarios; (a) significant deformation due to liquefaction-

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induced flow liquefaction or large deformations due to $k_y \leq 0.5PGA$; or (b) limited deformation.

8.6.3. Liquefaction Potential

We performed liquefaction analyses using a PGA of 0.21 and a magnitude of 6.65. Details regarding this analysis are presented in Appendix G. Table 8-7 summarizes the results of our liquefaction analyses using the CPT data obtained at two locations along the levee.

Table 8-7. Results of Liquefaction Analyses

CPT No.	Depths of Potentially Liquefiable Layers Below Levee Crown (ft.)	Thickness (ft)	USCS Classification
WR0828_010C	21.1 to 24.2, 29.3 to 32.5, 44.6 to 44.7, 47.2 to 47.9	3.1, 3.2, 0.1, 0.7	ML, SM, SM, & SP
WR0828_011C	27.4 to 28.5, 29.8 to 30.0, 30.6 to 30.9, 31.7 to 35.8, 37.2 to 39.7, 61.0 to 61.8	1.1, 0.2, 0.3, 4.1, 2.5, 0.8	ML, SM, SM, & SP

8.6.4. Post-Earthquake Slope Stability Analysis

A post earthquake static slope stability analysis was performed to estimate factors of safety against flow failure. "Flow failure" is a liquefaction-related phenomenon which occurs when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state. Sections are considered susceptible to flow failure if FOS was less than 1.0. Once triggered, the flow liquefaction may produce large deformations in slopes. Residual undrained shear strengths (S_r) of the potentially liquefiable layers were estimated based on the lower third value of Seed and Harder (1990) and used in the analyses as presented in Table 8-8. Therefore, the post-earthquake static slope stability analysis was performed on one section within Reach 1 using the slope stability software SLOPE/W. The post-earthquake static slope stability analysis was performed on both landside and waterside slopes of each section. Results of the analysis in terms of FOS against flow failure for each section are presented in Table 8-8.

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Table 8-8 – Factors of Safety for Post-Earthquake Static Slope Stability

Reach	Section	$(N_1)_{60\text{-}cs}$ for Potentially Liquefiable Layers	Residual Undrained Shear Strength (psf)*	Factors of Safety	
				Landside	Waterside
1	C	9 - 11	200	1.27	1.36

* Residual undrained shear strength is estimated based on lower third value of Seed and Harder (1990)

Results of the post-earthquake static slope stability analyses are presented on Plates G-4 and G-5. Material properties for each layer used in the analyses are also presented on these plates.

8.6.5. Pseudo-Static Slope Stability Analysis

A pseudo-static slope stability analysis was performed to evaluate the seismic performance of the levee slopes. This evaluation was performed by calculating the yield acceleration (k_y)[3] during the design seismic event. The pseudo-static slope stability analysis was performed on the landside and riverside slopes of the section at Section C(1)-C(2) using the slope stability software SLOPE/W (version 7.12). Results of pseudo-static slope stability analyses are presented on Plates G-6 through G-7 of Appendix G. Results of our analyses in terms of FOS are presented in Table 8-9.

Table 8-9. Results of Pseudo-Static Slope Stability

Reach	Section	Pseudo-Static Coefficient, k	Factor of Safety	
			Landside	Riverside
1	C	0. 5PGA = 0.11	1.43	1.16

Results of the pseudo-static slope stability analyses show that k_y for the riverside slope at Section C(1)-C(2) is greater than 0.11 for the landside slope. It should be noted that k_y is greater than 0.5PGA and the PGA associated with an earthquake having a return period of 200 years. Accordingly, the levee should have limited deformation (Case B).

3 *Yield acceleration is defined as the horizontal acceleration that results in a factor of safety of 1.0 in pseudo-static slope stability analysis.*

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8.7. EMBANKMENT PROTECTION

As described previously, the slopes of this portion of the Dad's Point levee are covered with vegetation and scattered riprap and concrete debris and are performing satisfactorily. Flood flow velocities are unknown for this project. However, it would be reasonable to assume that flood flow velocities against the levee would be low to moderate (in the range of 6 feet per second or less). For reference, information provided in the USACE Design Manual EM 1110-2-1601 "Hydraulic Design of Flood Control Channels," indicates that mean channel velocities in excess of 2 feet per second would be sufficient to initiate scour or erosion of exposed fine sand channel material. Where flow velocity range from about 2 to 6 feet per second, vegetation may be adequate to protect the slopes. The same manual indicates that exposed clay embankments should be able resistant to initial erosion and scour for flow velocities up to 6 feet per second. The boring and CPT information suggests that portions of the upper levee fill materials consist of clays. Additionally, no erosion was observed during our site inspection, and the waterside slope, as shown on Plates 5A to 5E, is generally covered with extensive vegetation. If it is determined that the flood flow velocities should be greater than 6 feet per second, then the embankment should be protected with stone riprap.

8.8. SUMMARY OF ANALYSES

For our evaluation of the Dad's Point levee, we have performed settlement, steady-state seepage, static and seismic slope stability, including assessment of the liquefaction potential of the site based on the existing levee section geometries. Table 8-10 summarizes the results of our analyses based on this new levee elevation. It should be noted that the FOS shown in Table 8-10 for Case III assumes that the landside slope is flattened to at least a 2:1 slope if a failure were to occur.

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Table 8-10. Summary of Analyses

Reach	Section	Settlement* (in)	Exit Gradient (i)	Factor of Safety					
				End-of-Construction** (Case I)		Rapid Drawdown (Case II)	100-yr WSE - Static (Case III)	Pseudostatic*** (Case IV)	
				Landside	Waterside	Waterside	Landside	Landside	Waterside
1	C	N/A	0.21	N/A	N/A	1.40	1.48	1.27	1.16

* Settlement analysis was not performed because no levee modifications are planned
** End-of-Construction analysis was not performed because no levee modifications are planned
*** Factor of safety was estimated based on a seismic coefficient of 0.11 (= 0.5PGA)

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9 MITIGATION DESIGN AND GRADING RECOMMENDATIONS

9.1. GENERAL

As summarized in Section 8, based on the results of our engineering analyses, it is our opinion that the Dad's Point Levee meets the acceptance criteria of EM 110-2-1913 for stability and seepage. It should also be noted that all construction recommendations contained herein are dependent upon existing ground surface conditions remaining unchanged. Should there be future modifications to the levee, excavations, or lowering of the ground, Kleinfelder should be contacted to evaluate the potential impacts to the area. The following design recommendations have not taken into consideration future changes to the existing topography and/or land uses as no modifications are currently planned. It should be noted that the following design recommendations are for general use only and not for final design.

9.2. FILL CRITERIA

9.2.1. Levee Fill Criteria

If the levee is modified in the future, we recommend that materials used for levee embankment fill meet the requirements in Table 9-1.

Table 9-1. Levee Embankment Fill Requirements

Fill Requirements		**Sieve Size (percent passing)**		**ASTM* Test Procedure**
Levee Embankment Fill				
Liquid Limit	Plasticity Index	#200	2 inch	D422/D4318
≤ 55	8 ≤ PI ≤ 40	≥ 30	100	
Organic Content				
Less than 4 percent				D2974

* American Society for Testing and Materials Standards (latest edition).

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9.3. SITE PREPARATION

9.3.1. Stripping and Grubbing

Prior to general site grading, existing vegetation, organic topsoil, and any debris should be stripped and disposed of outside the construction limits. Stripping depths should be on the order of 3 to 6 inches over a majority of the site, or as approved onsite by the geotechnical engineer. The stripped material can be spread on the surface of the levee landside slope after completion of construction. Deeper stripping or grubbing may be required where concentrations of organic soils or tree roots are encountered during site grading. Topsoil or any other organic laden materials should not be incorporated into any levee embankment.

9.3.2. Existing Utilities, Wells, and/or Foundations

Except for the steel pipe observed coming out of the waterside slope of the levee between Cross Sections D(1)-D(2) and E(1)-E(2) and extending into the San Joaquin River, our investigation did not encounter any active or abandoned existing utility lines, wells, and/or foundations. However, if any of these are encountered during grading, they should be removed and disposed of off site as per the Project Civil Engineer. Existing wells should be abandoned in accordance with applicable regulatory requirements. All excavations resulting from removal activities should be cleaned of loose or disturbed material, including all previously placed backfill. The excavation should be shaped with side slopes of 2:1 or flatter, to permit access for compaction equipment.

Per the City of Stockton, the steel pipe is a sewer pipe that extends from Atherton Island to the Dad's Point levee and outfalls into the San Joaquin River. During construction operations of the Smith Canal closure structure, the pipe should be examined and plugged to prevent water from the river seeping into Smith Canal during periods of head differentials between the two waterways.

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9.3.3. Subgrade Preparation and Compaction

Following stripping and grubbing, and/or over excavation of any soft/unsuitable materials, we recommend all areas to receive engineered fill be scarified to a depth of 8 inches, uniformly moisture conditioned to a range between one percent and three percent above optimum moisture content, and compacted to at least 90% of the maximum dry density as determined by ASTM Test Method D 1557 (Modified Proctor).

All engineered fill placed and compacted as part of any future levee embankment modification should meet the following compaction criteria.

- Embankment Fill: minimum 90% of the maximum dry density and between one percent and three percent above optimum moisture content as determined by ASTM Test Method D1557 (Modified Proctor).

If site grading is performed during or subsequent to wet weather, the near surface soils may be significantly above the optimum moisture content. This condition will hamper equipment maneuverability and impede adequate compaction. Where these conditions occur, disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile (fabric or grid), or other methods may be required to facilitate proper compaction and assist earthwork operations.

9.3.4. Construction Considerations

We do not anticipate excavation to be required for this project. Hence recommendations for temporary excavation including shoring, bracing, underpinning, or construction considerations related as such are not provided in this report.

9.4. PERMANENT SLOPES

9.4.1. General

If any levee improvements are planned, we recommend that slopes be constructed of engineered fill at a gradient no steeper than 2:1 and 3:1 on the landside and waterside

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slopes, respectively. We note that current USACE criteria for landside slopes is 3:1. However, considering that our conservative model of a flattened landside 2:1 meets the minimum FOS criteria, it is our opinion that the original guidelines of EM 1110-2-1913 can apply. The slopes should be constructed by overfilling and trimming back or by track walking with a sheeps foot compactor to provide a firm, well compacted slope face.

9.4.2. Key and Bench Requirements

It is unlikely that fills will be allowed on either slope due to environmental concerns. If fills are allowed, relatively thin "sliver fills" may be used to modify the existing levee slopes. Accordingly, new embankment fill placed on the existing levee slopes will not require a key into the existing levee slope or foundation. On other SJAFCA levees, "sliver fills" were allowed on the landside slope provided the exposed slope was first rolled with a wheel compactor to create "dents" in the exposed surface.

9.4.3. Erosion Control

To reduce the potential for surface erosion, all exposed cut and fill slopes should be vegetated with deep-rooted perennial grasses or similar plantings as soon as practical. In areas where relatively weak or erodible soils are encountered on slope faces, it may be necessary to use some type of erosion control matting, such as jute netting, straw mulch, and/or waddles, to help stabilize the slope surfaces until vegetation is well established. The project Civil Engineer should develop an erosion control plan to address both short-term and long-term erosion concerns.

Erosion control measures discussed above are for protection of the graded surfaces only and are not necessarily sufficient for flow and/or wind/wave action that may occur.

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10 LIMITATIONS

The conclusions and recommendations of this report are for evaluation and design purposes for the Dad's Point Levee evaluation project as described in the text of this report. The conclusions and recommendations in this report are invalid if:

- The assumed structural or grading details change
- The report is used for adjacent or other property
- Any other change is implemented which materially alters the project from that proposed at the time this report was prepared

The scope of services was limited to the three borings and a review of existing boring's and CPT's performed through the subject levee. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on the subsurface explorations performed along the subject levee and slope stability analysis, seepage modeling, laboratory testing, and visual site observations of the Dad's Point Levee.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more-detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involve greater expense, our clients participate in determining levels of service which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for future performance and maintenance.

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Recommendations and conclusions contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of this levee segment. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the conclusions of this report.

Should there be any new construction planned for this levee segment, as the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design and construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum, Kleinfelder should be retained to provide the following continuing services for any new site grading work.

- Review the project plans and specifications, including any revisions or modifications

- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for placement of engineered fill

- Confirm engineered fill is placed and compacted per the project specifications

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

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Should any new construction be planned, this report, and any future addenda or reports regarding this site, **may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report.** Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork operations. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

This report was prepared in accordance with the generally accepted standard of practice that existed in San Joaquin County at the time the report was written. No warranty, expressed or implied, is made.

It is the CLIENT'S responsibility to see that all parties to the project, including the designer, contractor, subcontractor, etc., are made aware of this report in its entirety.

This report may be used only by the client and only for the purposes stated within a reasonable time from its issuance, but in no event later than three years from the date of the report. Land use, site conditions (both on- and off-site), or other factors may change over time, and additional work may be required. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else, unless specifically agreed to in advance by Kleinfelder in writing, will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

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