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APPENDIX G
SEISMIC EVALUATION

This appendix presents our methodologies for and graphic results of preliminary seismic evaluation for the Dad's Point levee.

GENERAL METHODOLOGY

It should be noted there are no formal published guidelines for seismic evaluation of levees. California Department of Water Resources (DWR) has recently published a draft document entitled "Proposed Interim Levee Design Criteria for Urban and Urbanizing Area State-federal Project Levees" dated August 22, 2008 which indicates that for urban and urbanizing areas, 200-year ground motions are required for seismic assessments. However, there are no details about methodology as well as specific design criteria in terms of liquefaction and/or acceptable/unacceptable levee deformations under seismic conditions in this document. We also understand USACE is developing a guidance document on this topic; however, no specific information is available at this time. The following sections present the general methodology for the seismic evaluation.

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

- Site-specific probabilistic seismic hazard (PSHA) and deaggregation analyses are performed to estimate peak ground acceleration (PGA) and associated magnitude of an earthquake having return period of 200 years to be consistent with the return period of the design flood event.
- Design groundwater level for liquefaction analyses is taken as the normal water surface level.
- Liquefaction analyses are performed using the estimated PGA and magnitude and factors of safety are estimated based on the data from the CPTs containing potential liquefiable layers.

- If the factor of safety (FOS) against liquefaction is less than 1 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, 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 or pseudo-static $k_y \leq 0.5PGA$ indicates large deformations.

- If the FOS against liquefaction is greater than 1 for a soil layer, a pseudostatic slope stability analysis is conducted to estimate the yield acceleration. If the estimated yield acceleration is less than 0.5PGA, potential large deformation is warranted. Otherwise, it indicates only limited deformation

- Based on the results of 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) large deformation due to liquefaction-induced flow liquefaction or large deformations due to $K_y < 0.5PGA$; or (b) limited deformation where there is either no potential liquefaction soils or the yield acceleration is greater than 0.5PGA.

- These cases are further explained in the following section.

SUMMARY OF LIKELY SCENARIOS

Two likely scenarios are considered to summarize the results of the seismic evaluation of the Dad's Point levee. They are as follows:

Case A: Potential Flow Liquefaction or Large Deformations
This case indicates that the FOS against liquefaction is less than one. In addition, the FOS against post-earthquake slope stability is less than one and/or $k_y \leq 0.5PGA$ for a pseudo-static slope stability analysis or a post-earthquake slope stability analysis.

Case B: Limited Deformation

This case indicates that it does not meet the criteria for Case A. However, deformation due to earthquake-induced settlement in dry soils or soil strength softening may be possible.

SITE-SPECIFIC GROUND MOTION STUDY

We have developed PGA for the 200-year return period level earthquake using a probabilistic approach. The 200-year return period earthquake corresponds to the ground motion having 22.1 percent probability of exceedance in 50 years assuming a Poisson process for ground motion occurrences. A probabilistic modeling procedure using the software EZ-FRISK 7.37 (Risk Engineering, 2010) was used to estimate the PGA near the western end of the levee (Latitude: 37.9565N; Longitude: 121.3502W). A deaggregation analysis was performed at the PGA level to estimate the earthquake magnitude that may generate the design level PGA at the site. Deaggregation or disaggregation is a common practice in a PSHA to break the hazard back down into its contributions from different magnitude and distance pairs to provide insight into what events are the most important to the hazard at a given ground motion level. More discussions on deaggregation analysis can be found in Bazzurro and Cornell (1999). We have deaggregated the hazards into 3D Magnitude-Distance-epsilon bin.

Reliability of attenuation relationships beyond 200 km is questionable. In addition, seismic sources beyond 200 km from a site do not contribute to the seismic hazard for a site especially for an event having return period of about 200 years. Therefore, we have used faults within 200 km of the project site in our analyses and significant faults only within 100 kilometers of the site and corresponding fault parameters are shown in Table 6-1. Our seismic source model is based on the seismic source model used in developing probabilistic seismic hazard maps by the USGS/CGS for the State of California (Cao et al., 2003) and by the Working Group on California Earthquake Probabilities (2003) for the San Francisco Bay Area. The locations of the faults and associated parameters presented on Table 6-2 are based on data presented by Jennings (1994), Frankel et al. (1996, 2002), Cao et al. (2003), and Working Group on California Earthquake Probabilities (2007). The maximum earthquake magnitudes presented in this table are based on Wells and Coppersmith (1994) and are presented on the moment magnitude scale developed by Kanamori (1977) and Hanks and Kanamori (1979).

In addition to the individual seismogenic sources, we also allow for background seismicity that accounts for random earthquakes between M_w 5 and 7 based on the methodology described by Frankel et al. (1996). Some of the local faults have not been considered by USGS/CGS as independent seismogenic sources. Their seismicity has been included as background seismicity in the analyses.

Site-specific ground motions can be influenced by the styles of faulting, magnitudes of the earthquakes, and local soil conditions. The attenuation relationships used to estimate ground motion from an earthquake source need to consider these effects. Many attenuation relationships have been developed to estimate the variation of peak ground surface acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Under a Pacific Earthquake Engineering Research (PEER) Center project entitled "Next Generation of Attenuation (NGA)," five teams have developed and presented attenuation relationships for shallow crustal earthquakes in Western North America. These relationships are Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008). Prior to these, four of the most used relationships and widely accepted by seismologists for shallow crustal earthquakes in Western North America were the ones presented by Boore et al. (1997), Abrahamson and Silva (1997), Campbell and Bozorgnia (2003), and Sadigh et al. (1997).

The NGA relationships are more robust and preferred by the development teams instead of the 1997 relationships. We also understand that the USGS used three of these NGA relationships for shallow crustal earthquakes in developing the 2008 National Seismic Hazard Maps published this year. Therefore, we have decided to use the same three NGA relationships (Boore and Atkinson, Campbell and Bozorgnia, and Chiou and Youngs) as used by the USGS. All of these relationships require an estimate of V_{S30} (average shear wave velocity in the top 30 m) as an input. Based on the results of our field and laboratory investigations and using available correlations between soil strength parameters and shear wave velocity (V_S), we have estimated a V_{S30} of about 250 m/s (512 feet/sec) for the soils along the levee. In addition to the V_{S30} and in order to account for the effects of deep soil deposits and basin effects, some of these relationships use two additional parameters; $Z_{1.0}$ and $Z_{2.5}$. $Z_{1.0}$, defined as depth in meters to the location where V_S is about 1,000 m/s, is used by the Chiou and Youngs (2008) relationship. $Z_{2.5}$, defined as depth in km to the location where V_S is about 2,500 m/s, is used by the Campbell and Bozorgnia (2008) relationship. In absence of actual measurements for these parameters, empirical equations based on the V_{S30} value have been provided by the authors to estimate and then use these parameters into their equations. Using the equations provided by these authors, we have used 331.9 m for $Z_{1.0}$ and 2.5 km for $Z_{2.5}$ in our analyses.

A summary of the results of the site specific ground motion study is presented in Table G-1:

Table G-1. Summary of Site Specific Probabilistic Seismic Hazard Analysis

Reach	Return Period of Earthquake (years)	Probability of Exceedance	Deaggregated Moment Magnitude (M_w)	Peak Ground Acceleration (g)
1	200	22 percent in 50 Years	6.65	0.21

LIQUEFACTION ANALYSIS

Liquefaction describes a phenomenon in which saturated soil loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking during an earthquake. Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which cause settlement of the levee. Shear strength reduction combined with inertial forces from the ground motion may result in lateral migration (lateral spreading), extensional ground cracking of liquefied material, and slope failure. Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose sandy soils and low plasticity clay and silt.

In the past decade, several concentrated efforts have been made to come up with a uniform guideline for liquefaction analyses. Youd et al. (2001) published general guidelines for liquefaction analyses which presented consensus of a task committee comprising of more than 20 members from all over the country. In more recent years, however, Youd et al. procedures have been questioned by some researchers due primarily on the following findings/researches that:

- Post-earthquake reconnaissance found ample evidence of liquefaction and ground softening at sites where critical soil layers contained more than 15% particles finer than 5 mm. (1999 Mw 7.4 Kocaeli EQ and 1999 Mw 7.6 Chichi EQ).

- Improved evaluation of maximum acceleration (a_{max}) at each field case history site due to the improved understanding and treatment of:

  a. Directivity effects
  b. Effects of site conditions on response
  c. Improved attenuation relationships
  d. Availability of strong-motion records.

- Database of field performance case histories used by Youd et al. is particularly lacking in data from cases wherein peak ground shaking levels were high (CSR > 0.25).

- The CPT-based correction of Robetson and Wride is slightly unconservative for clean sands, especially at high CSR, and that it is very unconservative for soils of increasing fines content and plasticity.

These findings/researches resulted in significant modifications to the Youd et al. procedures. Some new approaches based on these findings/researches have been presented by Seed et al. (2003), Idriss and Boulanger (2004), Boulanger and Idriss (2006), and Bray and Sancio (2006).

Kleinfelder recently developed in-house spreadsheets that can compute three liquefaction evaluation approaches simultaneously including Youd et al. (2001), Seed et al. (2003), and Idriss and Boulanger (2008). However, our liquefaction evaluation for the Dad's Point Levee generally followed methods proposed by Youd et al. (2001). An earthquake magnitude of 6.65 and a PGA of 0.21g were used in liquefaction analyses that were estimated using EZ-Frisk program.

The liquefaction potential of subsurface soils can vary significantly depending on the selection of the design water level. Therefore, it is imperative that a proper water level elevation be selected for the liquefaction analysis. Due to the low probability of liquefaction occurring during a high water event, we have selected to use the normal water level in the San Joaquin River and Smith Canal as the design groundwater level. We believe this is an appropriate design water level for this study. A groundwater elevation of +2.0 ft (NAVD88) was used in the liquefaction analyses. A FOS of 1.0 was used as criteria to identify potential liquefiable layers.

Table G-2 summarizes the results of our liquefaction analyses for the CPT's along Reach 1. The results of our liquefaction analyses also indicate that only post-earthquake and pseudo-static stability analyses are required.

Table G-2. Summary 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

POST-EARTHQUAKE SLOPE STABILITY ANALYSIS

Post earthquake static slope stability analyses were 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. A section was 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 potential liquefiable layers were estimated based on lower third value of Seed and Harder (1990) and used in the analyses as presented in Table G-3. Therefore, the post-earthquake static slope stability analysis was performed for Section C within Reach 1 using the slope stability software SLOPE/W. The post-earthquake static slope stability analysis was performed on both landside and riverside slopes of the section. If the post-earthquake FOS is larger than 1.0, then a pseudo-static analysis with a seismic coefficient of 0.11g (i.e., 0. 5PGA) is performed for another post-earthquake stability analysis to see if the FOS is still larger than 1.0. Results of post-earthquake static and pseudo-static slope stability analyses are presented on Plates G-4 through G-7. Material properties for each layer used in the analyses are also presented on these plates. Results of the post-earthquake analyses in terms of FOS against flow failure for the section analyzed are presented in Table G-3. Results of the pseudo-static analyses in terms of FOS against slope failure for the section modeled are presented in Table G-4.

Results of the pseudo-static slope stability analyses show that k_y for the waterside and landside slopes at the selected cross sections are greater than 0.5PGA (i.e., 0.11g), thus indicating limited lateral deformation which is Case B.

Table G-3 – Factors of Safety for Post-Earthquake Static Slope Stability

Reach	Section	$(N_1)_{60-cs}$ 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)

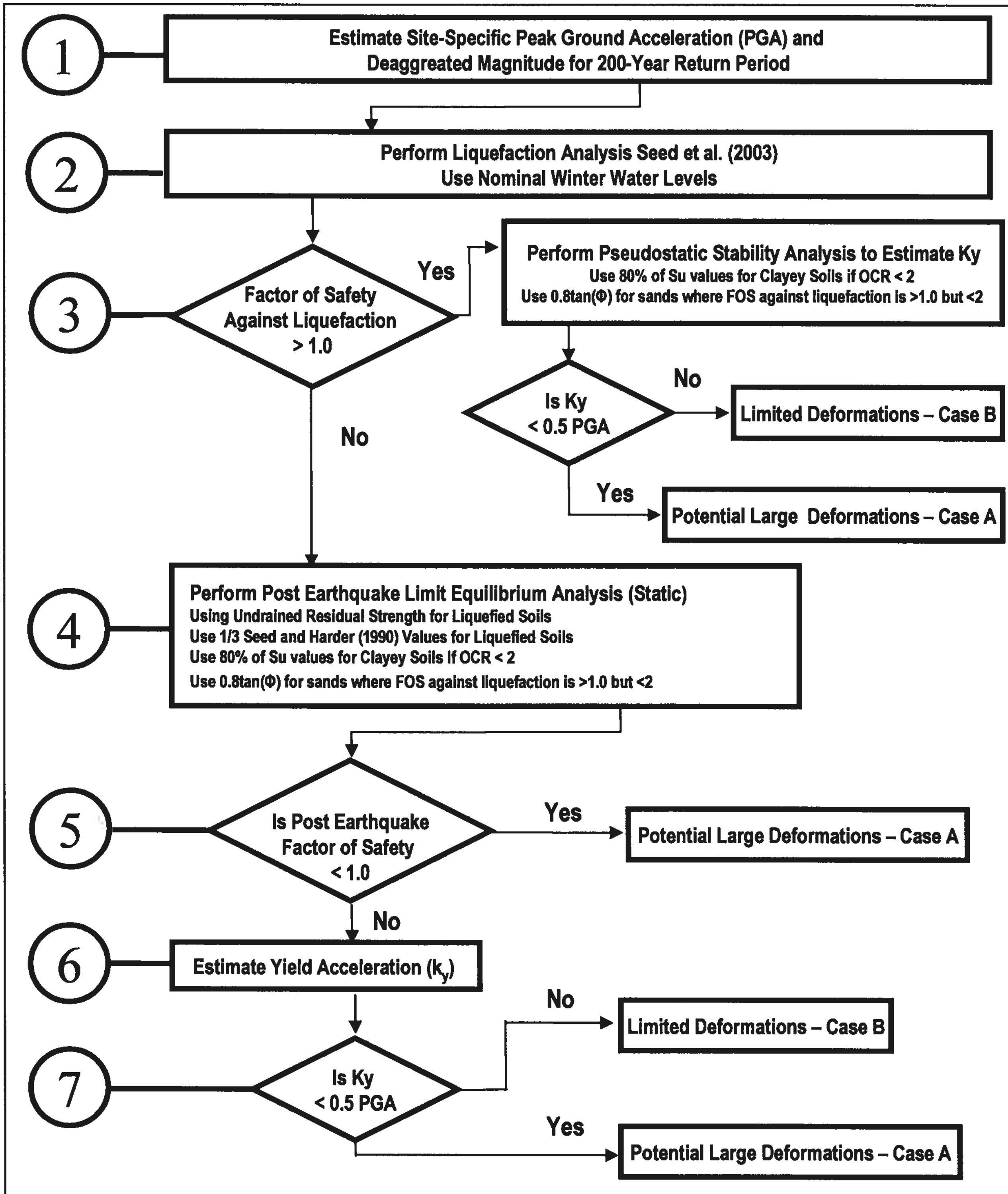
Table G-4. Results of Pseudo-Static Slope Stability

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

LIST OF ATTACHMENTS

The following plates are attached as part of this appendix and include graphical presentations of the seismic stability analyses:

Plate G-1	Flow chart for Seismic Evaluation
Plate G-2	CPT WR0828_010C Liquefaction Evaluation
Plate G-3	CPT WR0828_011C Liquefaction Evaluation
Plates G-4	Post-Earthquake Landside Stability Analysis
Plates G-5	Post-Earthquake Waterside Stability Analysis
Plates G-6	Pseudo-Static Landside Analysis with k_y = 0.5PGA = 0.11
Plate G-7	Pseudo-Static Waterside Analysis with k_y = 0.5PGA = 0.11



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FLOW CHART FOR SEISMIC EVALUATION

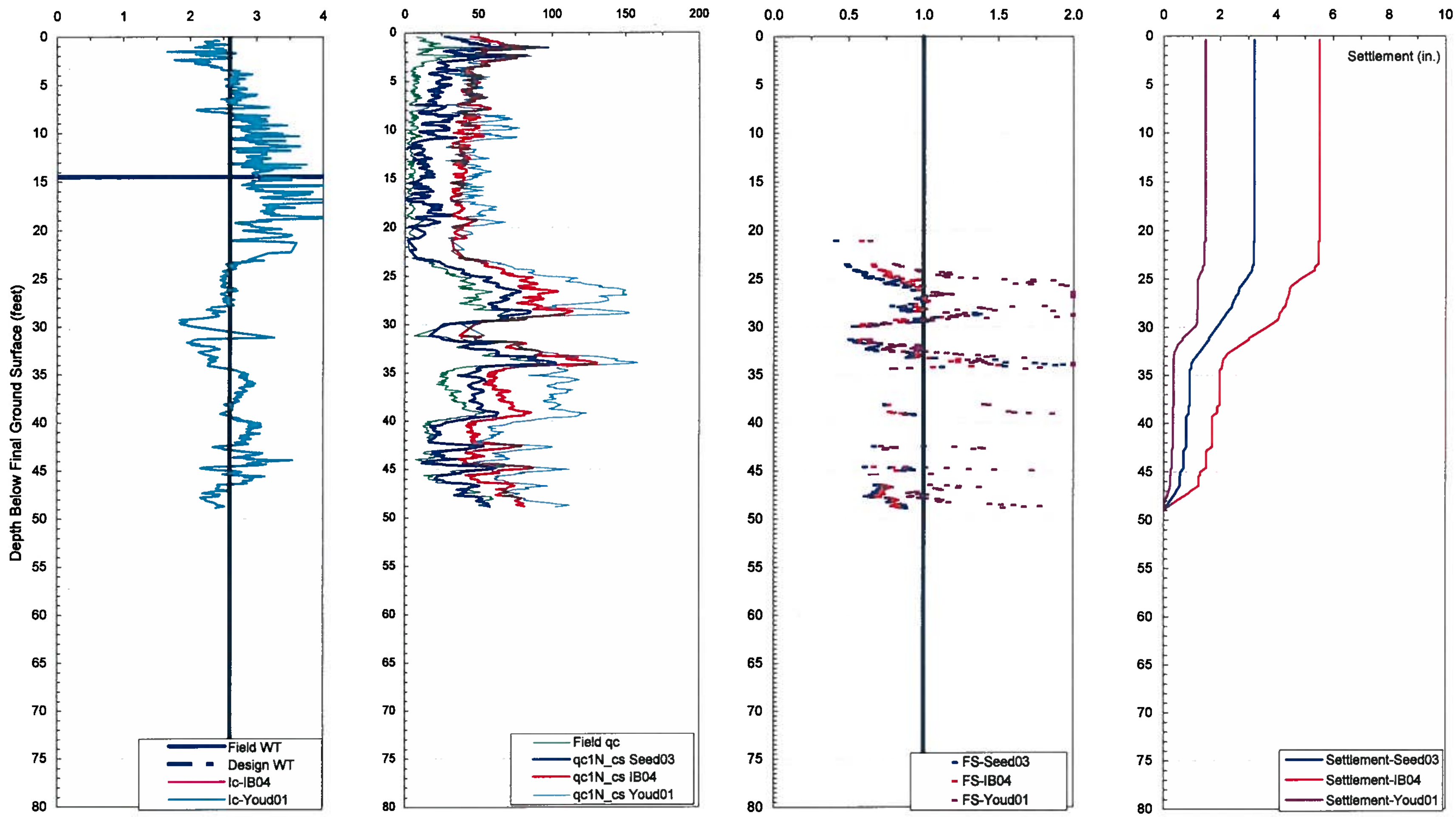
DAD'S POINT LEVEE EVALUATION
SAN JOAQUIN AREA FLOOD CONTROL AGENCY
STOCKTON, CALIFORNIA

DRAWN BY: Z. Zafir
REVISED BY: J. Liao
CHECKED BY: Z. Zafir

PLATE
G-1

DRAWN: 4/30/2009 | APPROVED BY________ | PROJECT NO. 92699

CPT ID: **WR0828_010C**	M_w= **6.7** PGA= **0.21g**	Field Ground Water Depth (ft)= **14.5 ft** Design Ground Water Depth (ft)= **14.5 ft**	Existing Ground Elevation= **16.5 ft** Final Ground Elevation= **16.5 ft**
Ana. by: SJW	Checked by:		



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Dad's Point Levee Evaluation
Project No.: 92459
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 1/13/2009 **PLATE G-2**

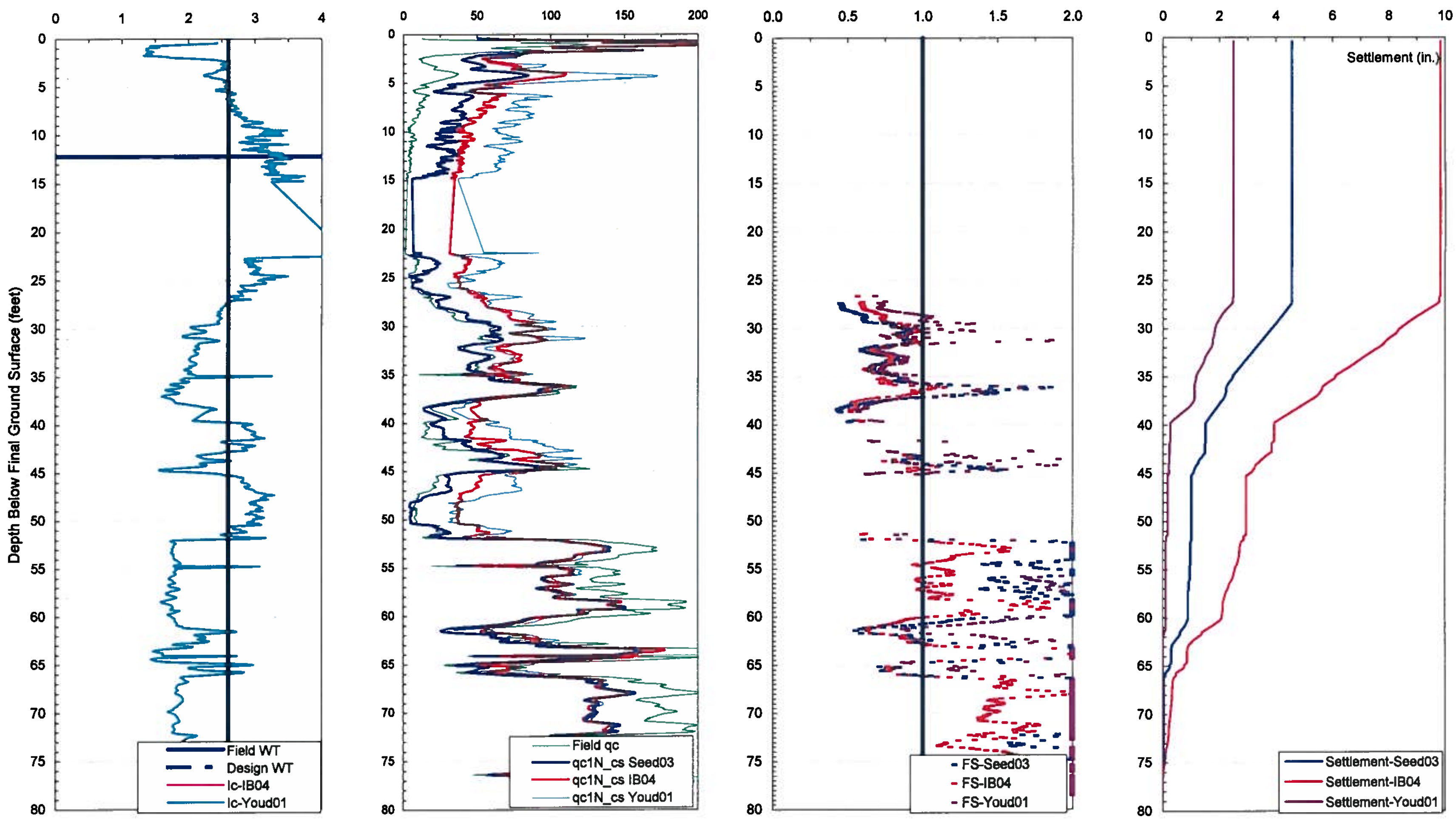
CPT ID: **WR0828_011C**

M_w= 6.7
PGA= 0.21g

Field Ground Water Depth (ft)= 12.2 ft
Design Ground Water Depth (ft)= 12.2 ft

Existing Ground Elevation= 14.2 ft
Final Ground Elevation= 14.2 ft

Ana. by: SJW
Checked by:



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Dad's Point Levee Evaluation
Project No.: 92459
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 1/13/2009 **PLATE G-3**

Dad's Point Levee — Section C, Post Earthquake Landside Stability Analysis

File Name: Section C_Stability_Post Earthquake_Landside.gsz
Last Saved Date: 1/12/2010
Analysis Type: SLOPE/W
Analysis View: 2D

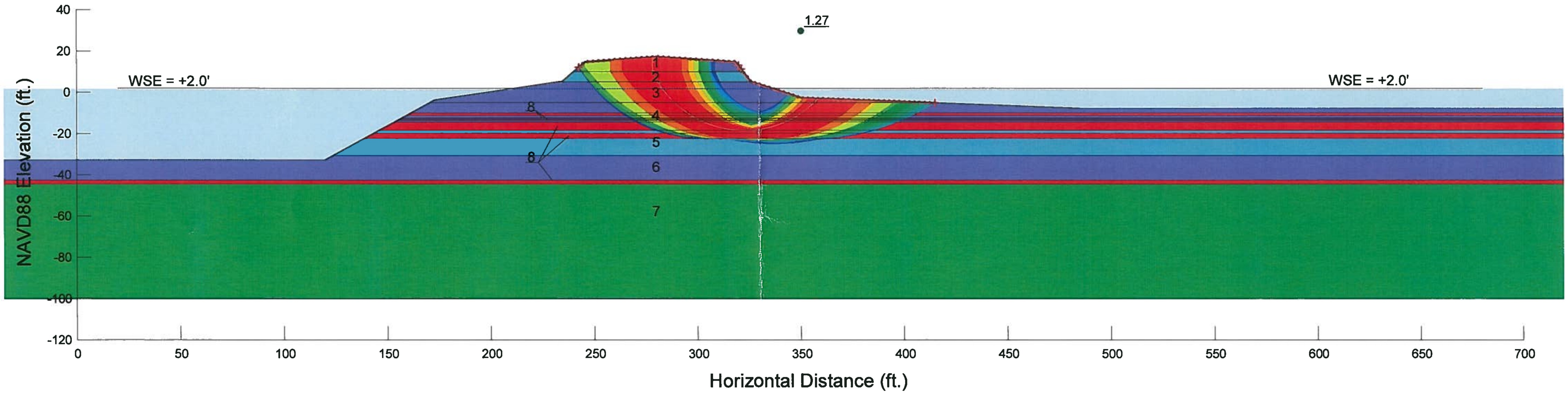
Material Number, Description, Unit Weight, Cohesion, Friction Angle

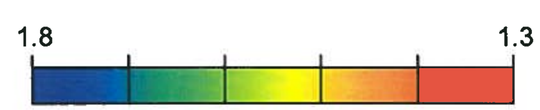
Material # 1: Fat Clay Unit Weight: 115 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 2: Silt Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 25 degrees
Material # 3: Organic Silt/Fat Clay Unit Weight: 100 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 4: Lean Clay Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 24 degrees
Material # 5: Sandy Silt Unit Weight: 115 pcf; Cohesion: 100 psf; Friction Angle: 26 degrees
Material # 6: Sandy Lean Clay Unit Weight: 115 pcf; Cohesion: 150 psf; Friction Angle: 24 degrees
Material # 7: Silty Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: 30 degrees
Material # 8: Liquefied Soil Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 0 degrees

Top of Levee Elevation: +17.4 Feet (NAVD88)
Landside Toe Elevation: -2.6 Feet (NAVD88)
Landside Elevation 150 Feet From Toe: -7.6 Feet (NAVD88)

Waterside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the waterside of the model was equal to the normal water elevation.

Landside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the landside of the model was equal to the normal water elevation.





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PROJECT NO.	92459.G02
DATE:	9.28.09
DRAWN BY:	S. Wiesner
CHECKED BY:	R. Heinzen

Dad's Point Levee
Post Earthquake Stability Analysis

LEVEE EVALUATION
DAD'S POINT LEVEE CERTIFICATION
STOCKTON, CALIFORNIA

PLATE
G-4

Dad's Point Levee — Section C, Post Earthquake Waterside Stability Analysis

File Name: Section C_Stability_Post Earthquake_Waterside.gsz
Last Saved Date: 1/13/2010
Analysis Type: SLOPE/W
Analysis View: 2D

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Fat Clay Unit Weight: 115 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 2: Silt Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 25 degrees
Material # 3: Organic Silt/Fat Clay Unit Weight: 100 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 4: Lean Clay Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 24 degrees
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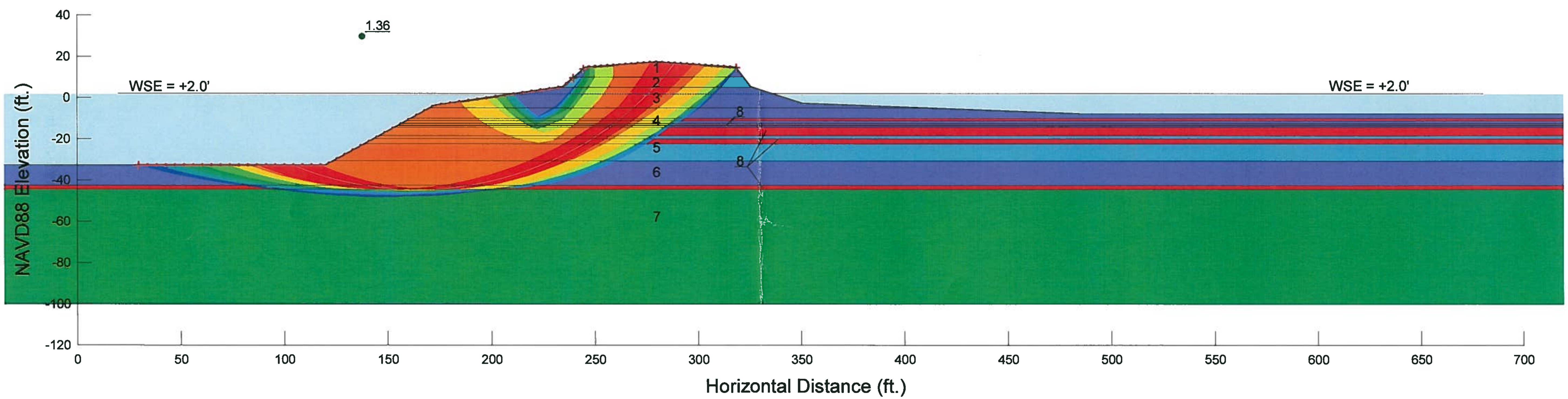
Material # 6: Sandy Lean Clay Unit Weight: 115 pcf; Cohesion: 150 psf; Friction Angle: 24 degrees
Material # 7: Silty Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: 30 degrees
Material # 8: Liquefied Soil Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 0 degrees

Top of Levee Elevation: +17.4 Feet (NAVD88)
Landside Toe Elevation: -2.6 Feet (NAVD88)
Landside Elevation 150 Feet From Toe: -7.6 Feet (NAVD88)

Waterside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the waterside of the model was equal to the normal water elevation.

Landside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the landside of the model was equal to the normal water elevation.



Factor of Safety Spectrum

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CHECKED BY:	R. Heinzen

Dad's Point Levee
Post Earthquake Stability Analysis

LEVEE EVALUATION
DAD'S POINT LEVEE CERTIFICATION
STOCKTON, CALIFORNIA

PLATE

G-5

Dad's Point Levee — Section C, Pseudostatic Landside Stability Analysis

File Name: Section C_Stability_PseudoStatic_Landside.gsz
Last Saved Date: 1/12/2010
Analysis Type: SLOPE/W
Analysis View: 2D

Material Number, Description, Unit Weight, Cohesion, Friction Angle

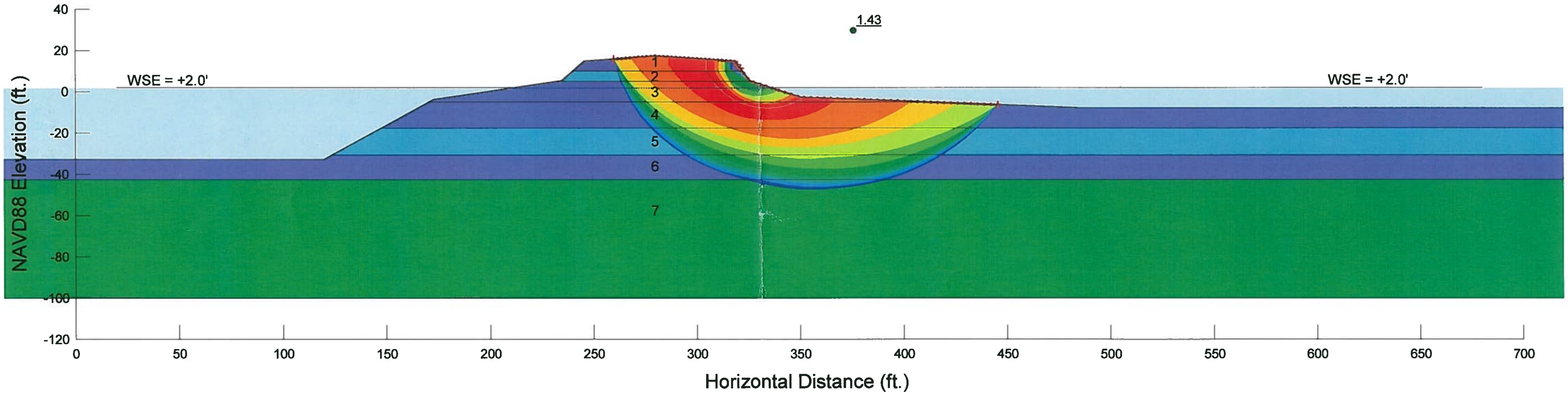
Material # 1: Fat Clay Unit Weight: 115 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 2: Silt Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 25 degrees
Material # 3: Organic Silt/Fat Clay Unit Weight: 100 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 4: Lean Clay Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 24 degrees
Material # 5: Sandy Silt Unit Weight: 115 pcf; Cohesion: 100 psf; Friction Angle: 26 degrees
Material # 6: Sandy Lean Clay Unit Weight: 115 pcf; Cohesion: 150 psf; Friction Angle: 24 degrees
Material # 7: Silty Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: 30 degrees

Top of Levee Elevation: +17.4 Feet (NAVD88)
Landside Toe Elevation: -2.6 Feet (NAVD88)
Landside Elevation 150 Feet From Toe: -7.6 Feet (NAVD88)

Waterside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the waterside of the model was equal to the normal water elevation.

Landside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the landside of the model was equal to the normal water elevation.

Seismic Coefficient = 0.5 x PGA = 0.11



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Dad's Point Levee
Pseudostatic Stability Analysis

LEVEE EVALUATION
DAD'S POINT LEVEE CERTIFICATION
STOCKTON, CALIFORNIA

PLATE
G-6

Dad's Point Levee — Section C, Pseudostatic Waterside Stability Analysis

File Name: Section C_Stability_PseudoStatic_Waterside.gsz
Last Saved Date: 1/12/2010
Analysis Type: SLOPE/W
Analysis View: 2D

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Fat Clay Unit Weight: 115 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
Material # 2: Silt Unit Weight: 115 pcf; Cohesion: 200 psf; Friction Angle: 25 degrees
Material # 3: Organic Silt/Fat Clay Unit Weight: 100 pcf; Cohesion: 300 psf; Friction Angle: 22 degrees
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Material # 5: Sandy Silt Unit Weight: 115 pcf; Cohesion: 100 psf; Friction Angle: 26 degrees
Material # 6: Sandy Lean Clay Unit Weight: 115 pcf; Cohesion: 150 psf; Friction Angle: 24 degrees
Material # 7: Silty Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: 30 degrees

Top of Levee Elevation: +17.4 Feet (NAVD88)
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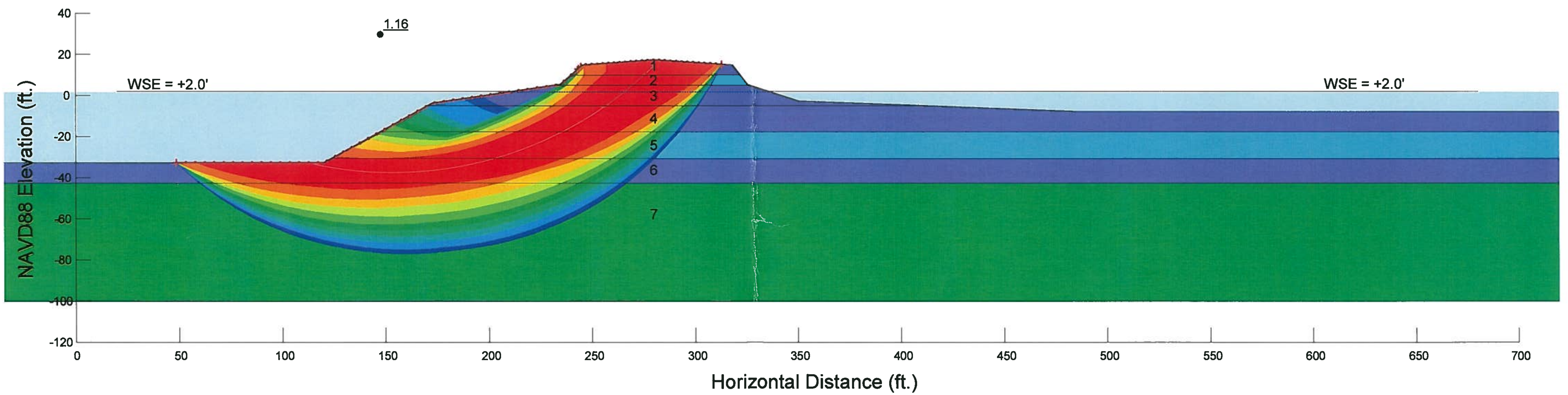
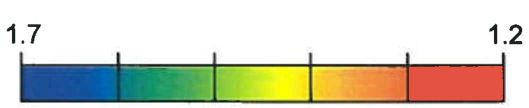
Landside Water Surface Elevation: +2.0 Feet (NAVD88)
*Note: The water elevation used on the landside of the model was equal to the normal water elevation.

Seismic Coefficient = 0.5 x PGA = 0.11





Factor of Safety Spectrum

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Pseudostatic Stability Analysis

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PLATE
G-7