APPENDIX C

PREVIOUS REPORTS

Peters Canyon 6MG Reservoir

Instrumentation Monitoring May 2025

Performed By:

Technical Services Inc.

June 10, 2025

Mr. Bobby Young
Engineering Manager
East Orange County Water District
185 N. McPherson Rd.
Orange, CA 92869

Subject: Instrumentation Monitoring Report
Peters Canyon 6 MG Reservoir
Orange County, CA

Dear Mr. Young:

AECOM Technical Services Inc. (AECOM) is pleased to submit this report for the Peters Canyon 6 million-gallon (MG) Reservoir (Project Site) Instrumentation Monitoring visit, conducted on May 22, 2025, as part of the FY 2024-2025 and 2025-2026 Instrumentation Monitoring Project. The report has been prepared in accordance with our proposal dated June 3, 2024, and your Notice to Proceed and Agreement for Professional Services, both dated October 21, 2024. Should you have any questions or require further clarification, please feel free to contact Sabah Fanaiyan at 818-857-2884.

Sincerely,

AECOM Technical Services, Inc.

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References

1. AECOM. 2021. Goetechnical Investigation Report, Peters Canyon 6MG Reservoir Investigation, Orange County, California: Prepared for East Orange County Water District. December 7, 2021.
2. AECOM 2022. Peters Canyon 6MG Reservoir Monthly Instrumentation Monitoring Final Report. Report dated April 2022.

1. Executive Summary

On May 22, 2025, AECOM conducted instrumentation monitoring of the existing monitors at the project site including inclinometers, crack monitors and groundwater monitoring wells. The recorded data indicate no significant changes on the readings compared to the previous readings or the baseline established in 2021. Therefore, there are no concerns regarding the structural integrity of the reservoir at this point.

2. Project Background

East Orange County Water district (EOCWD) owns and operates the 6 MG Peter Canyon Reservoir within a gated facility off Handy Creek Road in unincorporated Orange County east of the city of Orange, California. In December 2020 EOCWD contracted AECOM to evaluate the cause of the cracking/separation at the southwest corner of the existing 6 MG Peters Canyon Reservoir for EOCWD. In 2021 AECOM conducted a geotechnical field investigations and laboratory testing program and prepared a report (AECOM 2021) summarizing the findings of the investigations. The results of the evaluation at that point suggested that the existing separations do not adversely affect the structural integrity of the reservoir structure, nor affect its ability to store water. Improvement of the existing site drainage system was recommended by AECOM to mitigate the distress development of the observed separation. In 2022 EOCWD repaired the existing drainage system. Since then, EOCWD hired AECOM to perform instrumentation monitoring of the installed inclinometers and crack monitors and provide reports summarizing the readings and observations along the southwest corner of the reservoir.

3. Scope of work

The general scope of this contract is to perform up to 6 sets of readings of the existing instruments and prepare a report summarizing the readings. The instruments to be monitored consist of:

- 3 Inclinometers: A-21-1, A-21-2, and A-21-3
- 5 Crack monitors on the southwest corner of the reservoir: A", B', C", E", and F"
- 4 Crack monitors on the curb: G, H, I, and J
- 4 Existing Groundwater wells
- Conduct a walkthrough of the project site, photograph and note down visible issues of the existing drainage and pavement.
- Prepare this Monitoring Report

4. Inclinometers

Three 2.75-inch diameter 65 ft deep inclinometers (A-21-1, A-21-2, and A-21-3) were installed in the HSA borings during AECOM's geotechnical exploration in March 2021 to monitor long-term horizontal displacement of the Southwest corner slope of reservoir. The baseline readings were performed in April 2021. Since the initial readings, AECOM has performed 11 monthly readings (between May 2021 and April 2022) and 3 additional readings in 2023 (March, September and December of 2023). The inclinometers are tilt-sensing devices for monitoring deformation normal to the axis of a flexible PVC casing. Horizontal displacement is calculated by measuring the tilt along the casing at 2-foot increments starting from the bottom of borehole. As part of monitoring, the inclinometer probe is advanced down the casing using guide wheels that are inserted into the casing grooves. The probe sensors will measure tilt along the plane of the guide wheels referred to as the A-axis, while another sensor measures tilt perpendicular to the guide wheel, referred to as the B-axis. Measurements, and any changes in tilt are always relative to that baseline. Summary of 2023 and 2024 readings, along with our recent readings performed on May 22, 2025, are provided in Table 1 of this report. Plots of the latest readings, including the one from AECOM's most recent site visit, are provided in Appendix A. The location map of the inclinometers is provided in Figure 1.

5. Reservoir Wall Crack Monitors

A total of 5 crack monitors (A, B, C, E, and F) were installed by EOCWD sometime after December 2020 on the south and the west wall between the wall and the corresponding pilasters of the reservoir. Per EOCWD the installed crack monitors were originally set to zero movement (photographs available with EOCWD). Therefore, AECOM continued monitoring any changes from the residual movement starting on April 2021. During our site visit in March 2023, we noticed that all the 5 crack monitors were removed prior to our visit. Therefore, 5 new crack monitors (A', B', C', E', and F') were installed on March 3, 2023, and a baseline reading was taken during the same site visit. A total of three instrumentation monitoring visits were conducted and reported in 2023, and the readings are provided in Table 2. The location map of the reservoir wall crack monitors is provided in Figure 2.

As part of FY 2024-2025 monitoring project, AECOM's field engineer visited the site in December 2024. It was observed that since our last visit the reservoir crack monitors had experienced some detachment over time, possibly due to weather-related effects on the adhesive. Therefore, the existing crack monitors were removed, and new monitors (A", B", C", E", and F") were installed at the similar locations of the previous monitors. Prior to the removal, measurements were taken to document existing conditions (see Table 2). During the recent site visit, crack monitor A" had a portion of the monitoring device missing, and crack monitor F" had detached, possibly due to weather-related effects on the adhesive. Apparently, the adhesive provided by the manufacturer has not performed as expected. AECOM will research alternative adhesives that offer enhanced durability and reinstall all the monitors to ensure a lasting solution. The latest readings for the current ones are provided in Table 3. Field pictures showing the installed crack monitors with the corresponding readings are also provided in Appendix B-1.

6. Concrete Curb Crack Monitors

To monitor the existing cracks on the concrete curbs located at the southwest corner of the reservoir, a total of 4 crack monitors were installed on the curbs. The crack monitors were installed on the concrete curb at the southwest corner of the reservoir. The readings for the concrete curb crack monitors are provided in Table 4 of this report.

7. Groundwater Monitoring Wells

A total of 4 monitoring wells installed by others are located near the reservoir. The wells are generally monitored via a water level sounder to gauge for any presence of water. The location of these wells is shown in Figure 3. The latest readings are tabulated in Table 5.

8. Visual Site Observation

During the recent site visit, AECOM's field engineer preformed a follow up visual inspection of the asphalt around the site, specifically the southwest corner of the reservoir for any visible distresses around the reservoir. Besides some surficial cracks throughout the site and some low-level cracks (less than 1 inch) predominantly between the asphalt and the southwest portion of the concrete curb, no significant alligator or fatigue cracking of the asphalt was observed during this recent visit. The low-level cracks appear very uniform and are possibly a result of placing asphalt in separate sections. Photos of the existing asphalt surface from this site visit are presented in Appendix B-2.

Tables

Table 1: Inclinometer Readings

Inclinometer	Max Deviation from Baseline 4/27/2023						Max Deviation from Baseline 9/20/2023					
	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (in.)	Depth bgs (ft.)	Direction	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (in.)	Depth bgs (ft.)	Direction
A-21-1	-0.32	4	North East (Towards reservoir)	-0.26	40	South East	-0.22	4	North East (Towards reservoir)	-0.23	40	South East
A-21-2	0.13	18	West	-0.11	2	South	-0.06	2	East	-0.12	2	South
A-21-3	0.09	8	West	-0.14	4	East	-0.06	22	West	-0.11	4	East

Inclinometer	Max Deviation from Baseline 12/20/2023						Max Deviation from Baseline 12/23/2024					
	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (in.)	Depth bgs (ft.)	Direction	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (in.)	Depth bgs (ft.)	Direction
A-21-1	-0.21	4	North East (Towards Reservoir)	-0.23	40	South East	-0.36	4	North East (Towards Reservoir)	-0.26	40	South East
A-21-2	-0.13	2	East	-0.05	2	South	-0.20	2	East	-0.05	2	South
A-21-3	0.05	2	North	-0.11	4	East	0.11	22	North	-0.20	4	East

Inclinometer	Max Deviation from Baseline 5/22/2025						N/A					
	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (In.)	Depth bgs (ft.)	Direction	A-Axis (in.)	Depth bgs (ft.)	Direction	B-Axis (in.)	Depth bgs (ft.)	Direction
A-21-1	-0.24	4	North East (Towards Reservoir)	-0.24	40	South East						
A-21-2	-0.20	2	East	-0.05	2	South						
A-21-3	0.07	56	North	-0.16	4	East						

Notes:

1. Directions refer to project north direction relative to reservoir orientation.
2. Inclinometer probe accuracy +/- 0.01 in. per reading.
3. 2021-2022 readings not reported in this report for simplification. Refer to Peters Canyon 6MG Reservoir Monthly reports dated prior to 2023.
4. Bgs = Below ground surface

Table 2: Previous Reservoir Wall Crack Monitor Readings

Crack Monitor ID		Baseline 3/30/23	10/12/23	12/20/23	12/23/24
A’	Reading (mm)	0	-1.0 North	0.0	+0.5 South
	Change from Baseline (mm)	-	-1.0 North	0.0	+0.5 South
B’	Reading (mm)	0	0.0	0.0	0.0
	Change from Baseline (mm)	-	0.0	0.0	0.0
C’	Reading (mm)	0	0.0	0.0	+1.0
	Change from Baseline (mm)	-	0.0	0.0	+1.0
E’	Reading (mm)	0	-3.0 West	-3.0	-4.5 East and -4.5 North
	Change from Baseline (mm)	-	-3.0 West	-3.0	-4.5 East and -4.5 North
F’	Reading (mm)	0	-2.0 North	-1.0	-2.0 North
	Change from Baseline (mm)	-	-2.0 North	-1.0	-2.0 North

Notes:

5. Reservoir crack monitors in Table 2 had experienced some detachment over time, possibly due to weather-related effects on the adhesive. Therefore, the existing crack monitors were removed. Readings taken on the 12/23/2024 were the last readings prior to removal.

Table 3: Reservoir Wall Crack Monitor Readings (2024-2025)

Crack Monitor ID		Baseline 12/23/24	5/22/25		
A”	Reading (mm)	0	N/A		
	Change from Baseline (mm)	-	-		
B”	Reading (mm)	0	0.0		
	Change from Baseline (mm)	-	0.0		
C”	Reading (mm)	0	0.0		
	Change from Baseline (mm)	-	0.0		
E”	Reading (mm)	0	0.0		
	Change from Baseline (mm)	-	0.0		
F”	Reading (mm)	0	N/A		
	Change from Baseline (mm)	-	-		

Notes:

6. Crack monitors in Table 2 were replaced with new devices in the same location, and a new baseline reading was taken, as shown in Table 3 above.
7. Crack monitor A’ and F’ were found detach at the time of the site visit. Adhesive for F’ appeared to have worn out. A portion for crack monitor A’ was missing. All wall crack monitors will be reinstalled and reinforced with significant adhesive.

Table 4: Concrete Curb Crack Monitor Readings

Crack Monitor ID		Baseline 3/30/2023	10/12/2023	12/20/2023	12/23/2024	5/22/2025
G’	Reading (mm)	0.0	-4.0 West and +1.75 North	-5.0 West and +1.75 North	-9.0 West and +3.0 North *	> -10.0 West and +3.0 North**
	Change from Baseline (mm)	-	-4.0 West and +1.75 North	-5.0 West and +1.75 North	-9.0 West and +3.0 North *	> -10.0 West and +3.0 North**
H’	Reading (mm)	0.0	0.0	0.0	+2.0 west	+3.0 West and +1.0 North
	Change from Baseline (mm)	-	0.0	0.0	+2.0 west	+3.0 West and +1.0 North
I’	Reading (mm)	0.0	N/A	N/A	N/A	N/A
	Change from Baseline (mm)	-	N/A	N/A	N/A	N/A
J’	Reading (mm)	0	0.0	0.0	0.0	0.0
	Change from Baseline (mm)	-	0.0	0.0	0.0	0.0

Notes:

* Crack Monitor G’: based on our field observation, the greater deviation between 12/20/2023 and 12/23/204 readings are due to the movement of the curb pieces (separated pieces from day first). Therefore, there is no concern of any exceeded crack or slope movement. Crack monitor I’ needs replacement.

** Crack Monitor G’: based on our field observation, the minor movement between 12/23/2024 and 5/22/2025 readings are possibly due to the movement of the curb pieces. Inclinometer data does not show significant changes, therefore, there is no concern of any exceeded crack or slope movement. Crack monitor I’ needs replacement.

Table 5: Groundwater Well Readings

GW Well ID	Groundwater Depth				
	Reading Date: 4/28/23	Reading Date: 9/20/23	Reading Date: 12/20/23	Reading Date: 12/23/24	Reading Date: 5/22/25
G3MW-1	Dry	Dry	Dry	Dry	Dry
G3MW-2	Dry	Dry	Dry	Dry	Dry
G3MW-3	Dry	Dry	Dry	Dry	Dry
G3MW-4	Dry	Dry	Dry	Dry	Dry

Figures

Legend

2021 Exploration

A-21-1 ◎ AECOM Inclinometer

A-Axis

B-Axis

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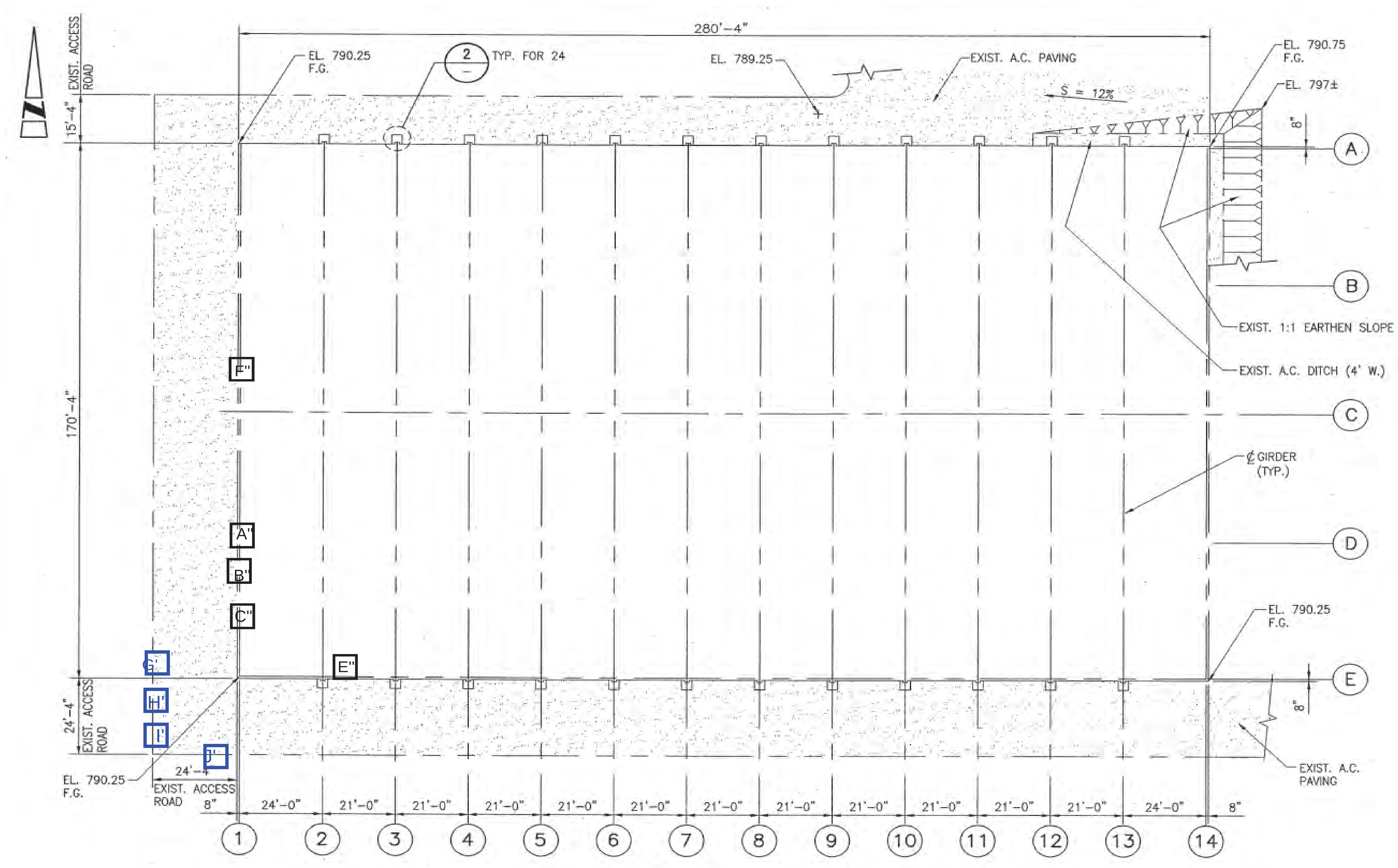
INCLINOMETER PLAN

Project No. 60653060	Date: March 2025
Project Name Peters Canyon 6MG Reservoir Geotechnical Investigation	Figure 1

Crack Monitoring Plan



NTS

LEGEND

A' Reservoir Wall Crack Monitor Location.
Removed in December 2024*

G' Curb Crack Monitor Location

A" Reservoir Wall Crack Monitor Location.
Installed in December 2024

Reference: Plan from 2000
Retrofit by Boyle Engineering

Note: The direction of the movement indicated is referring to the building orientation as shown in this plan.

* Refer to previous reports for crack monitors installed in 2023 and nomenclature

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Crack Monitoring Plan

Project No.	March 2025
Project Name: Peters Canyon 6MG Reservoir Investigation	Figure 2

Legend

G3 Soil Works Monitoring Wells 2016

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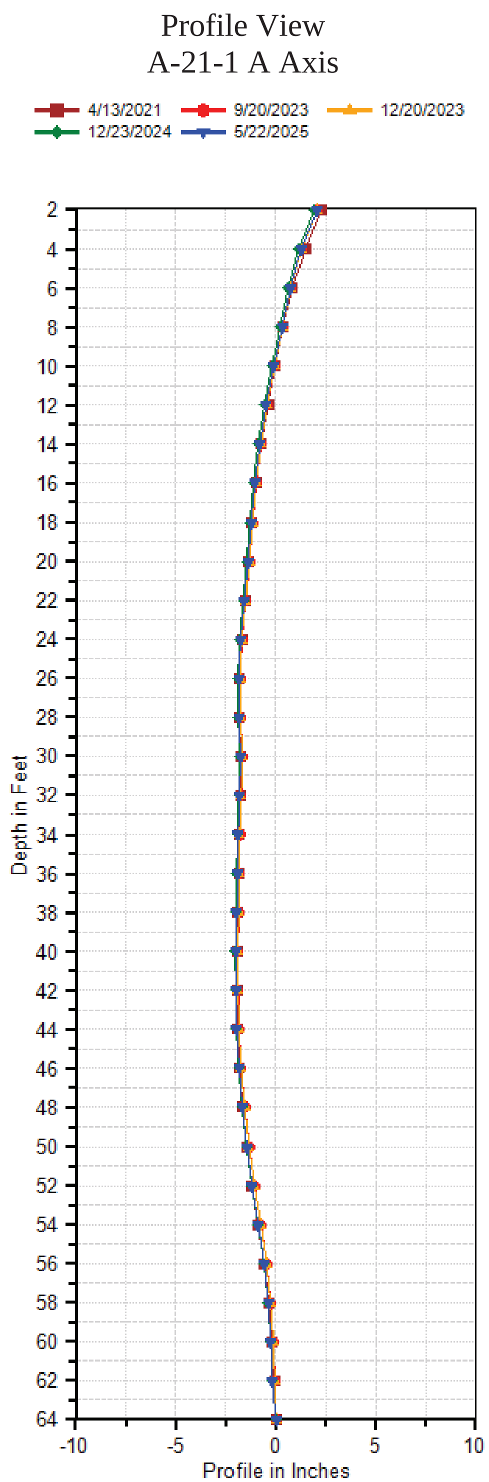
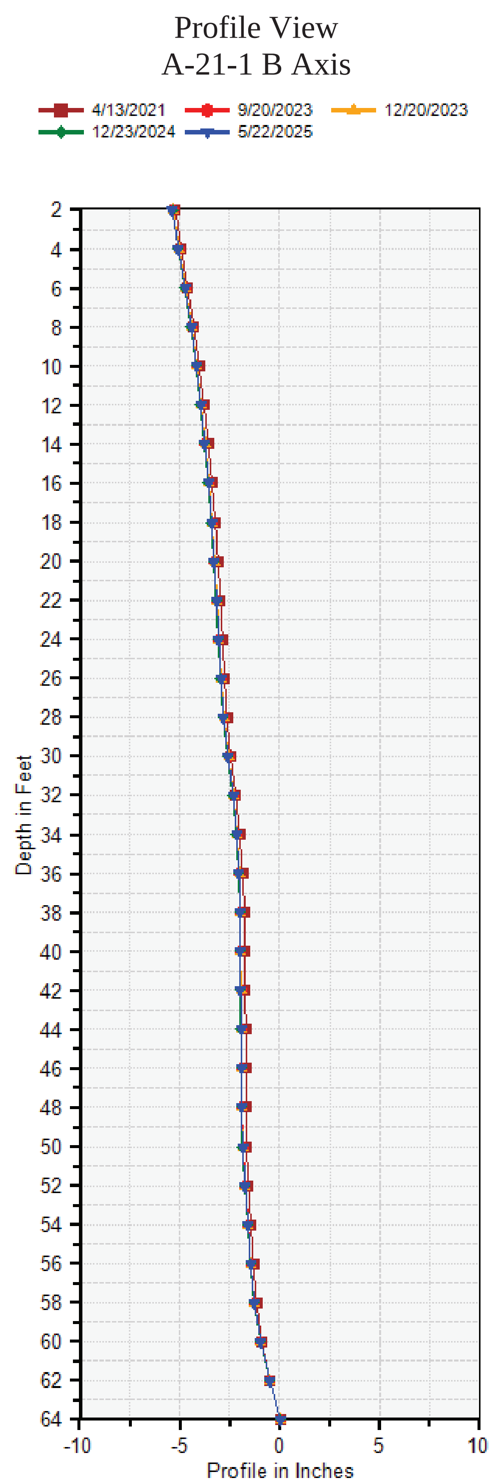
Existing Monitoring Wells

Project No.	60705526	Date: March 2025
Project Name	Peters Canyon 6MG Reservoir Monitoring	Figure 3

Appendices

Appendix A: Inclinometer Reading Graphs





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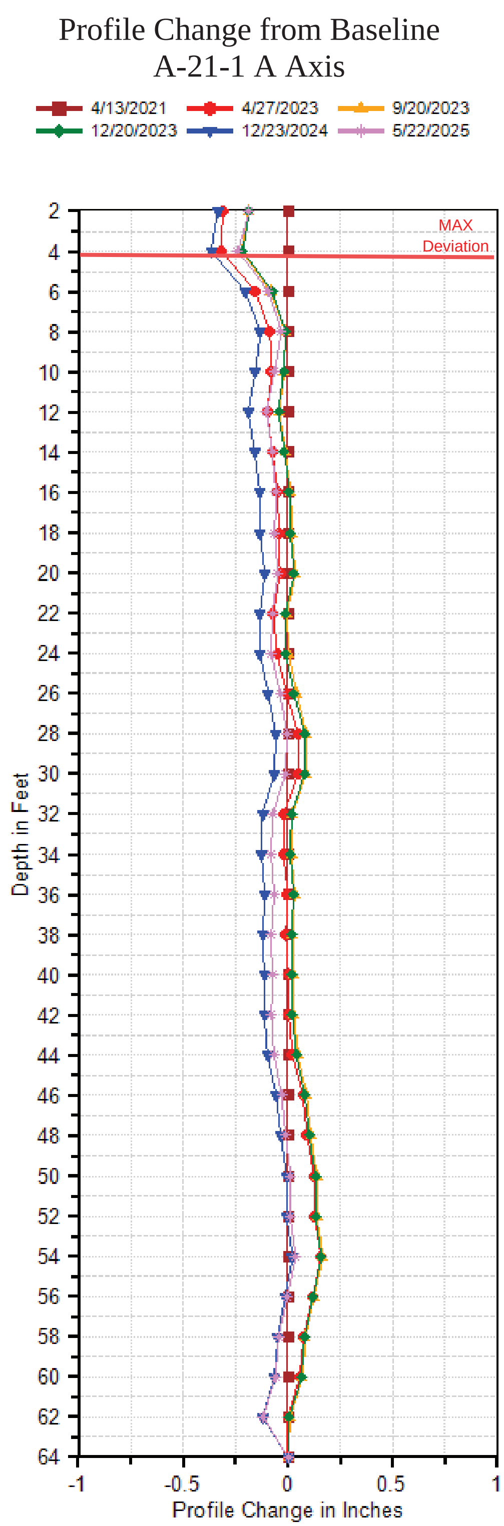
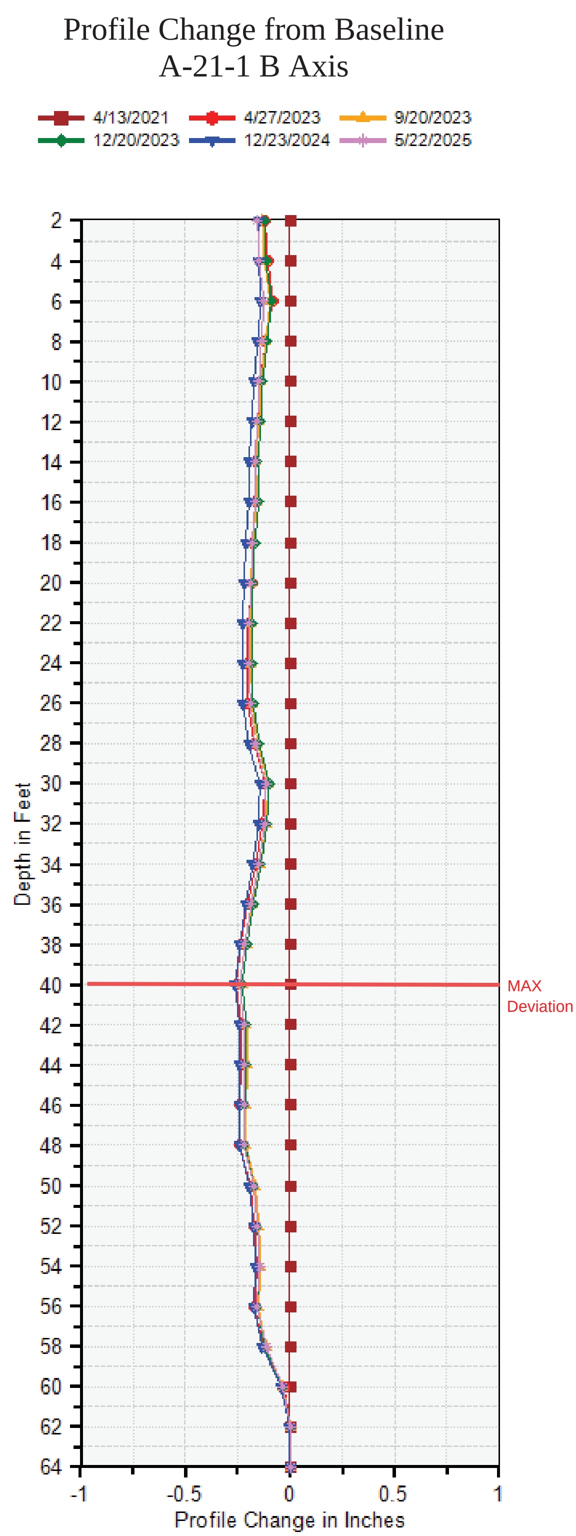
Inclinometer Reading May 2025

Boring ID: A-21-1

*4/13/21 Baseline Reading

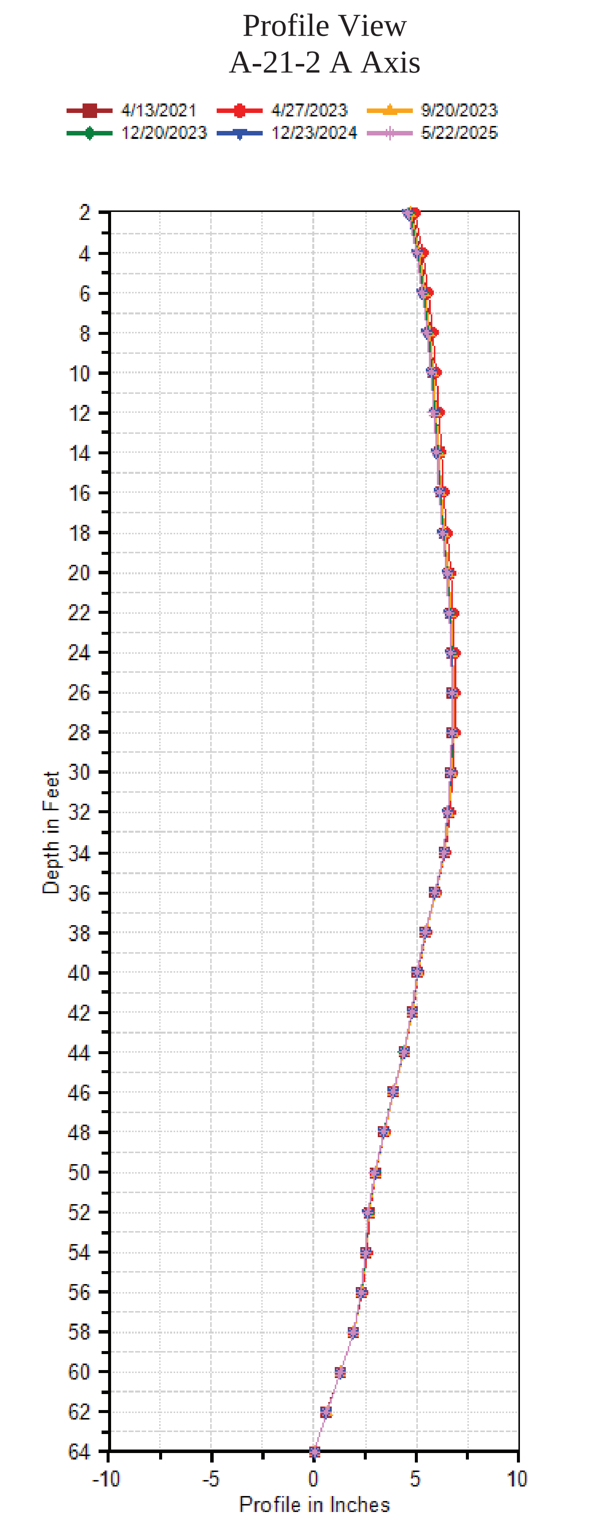


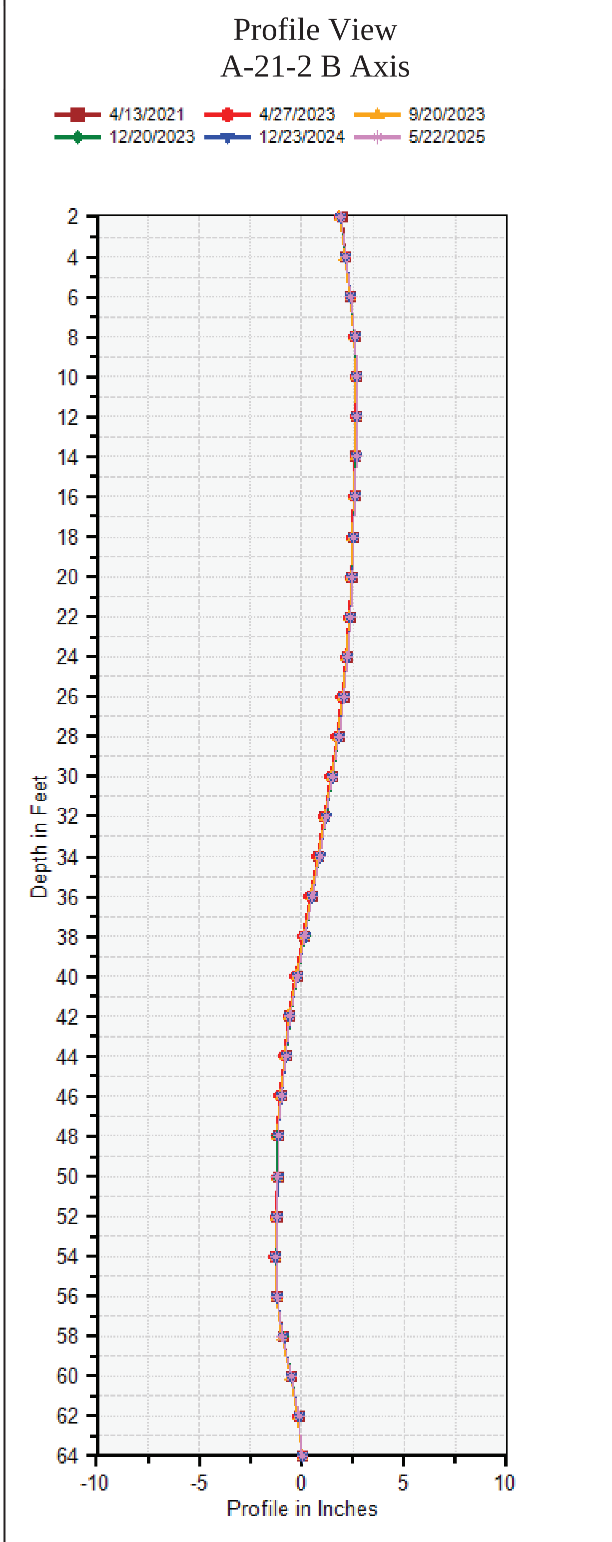


Inclinometer Reading May 2025

Boring ID: A-21-1

*4/13/21 Baseline Reading

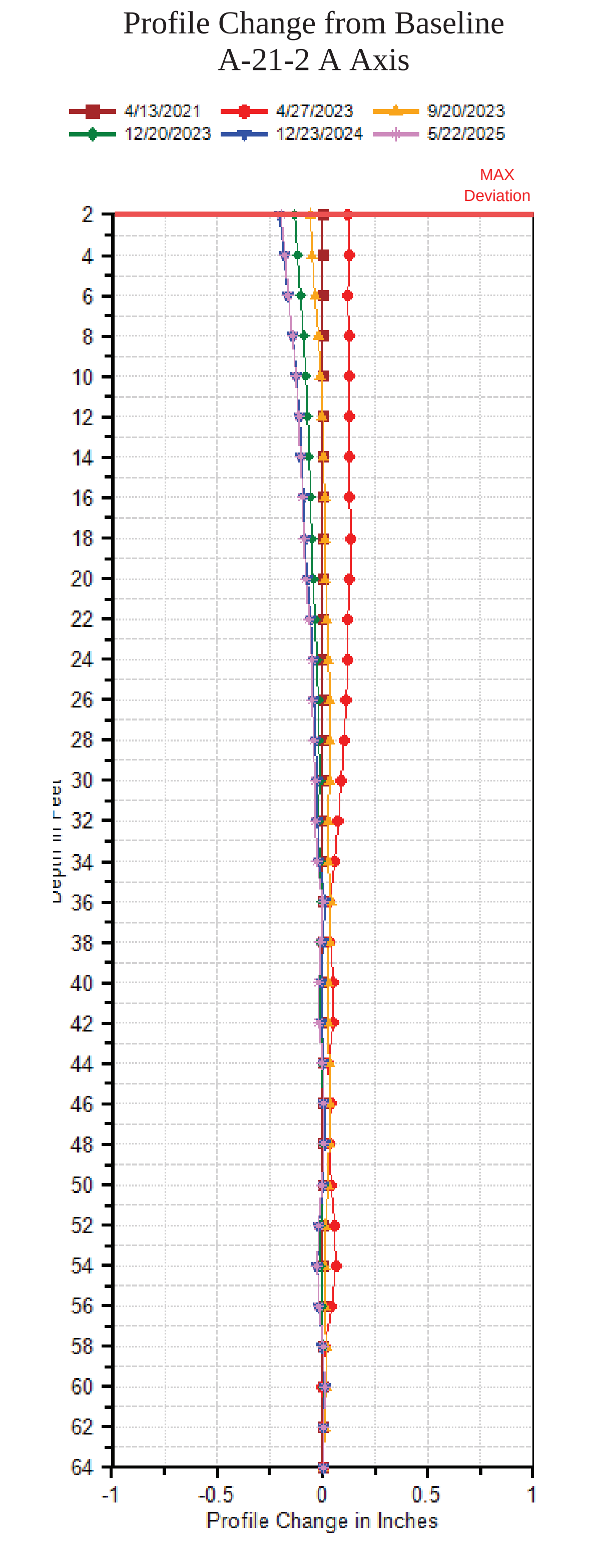


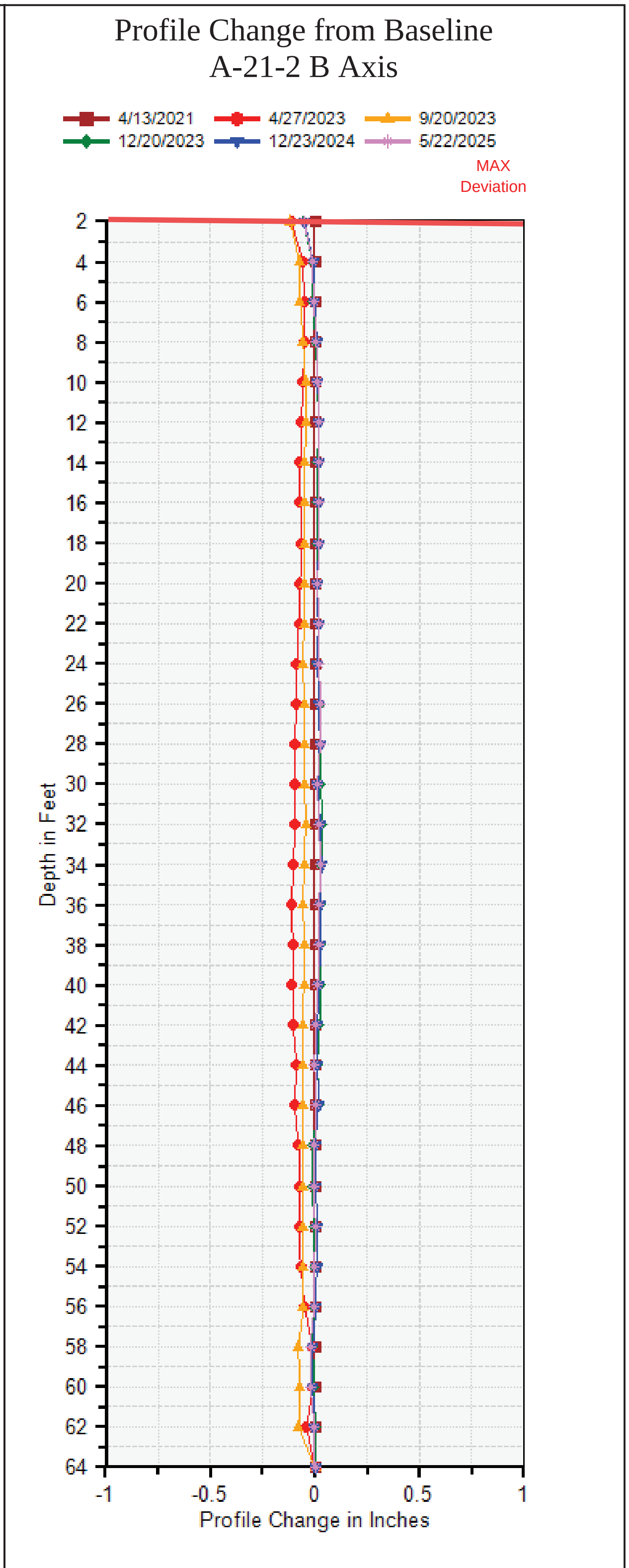


Inclinometer Reading May 2025

Boring ID: A-21-2

*4/13/21 Baseline Reading



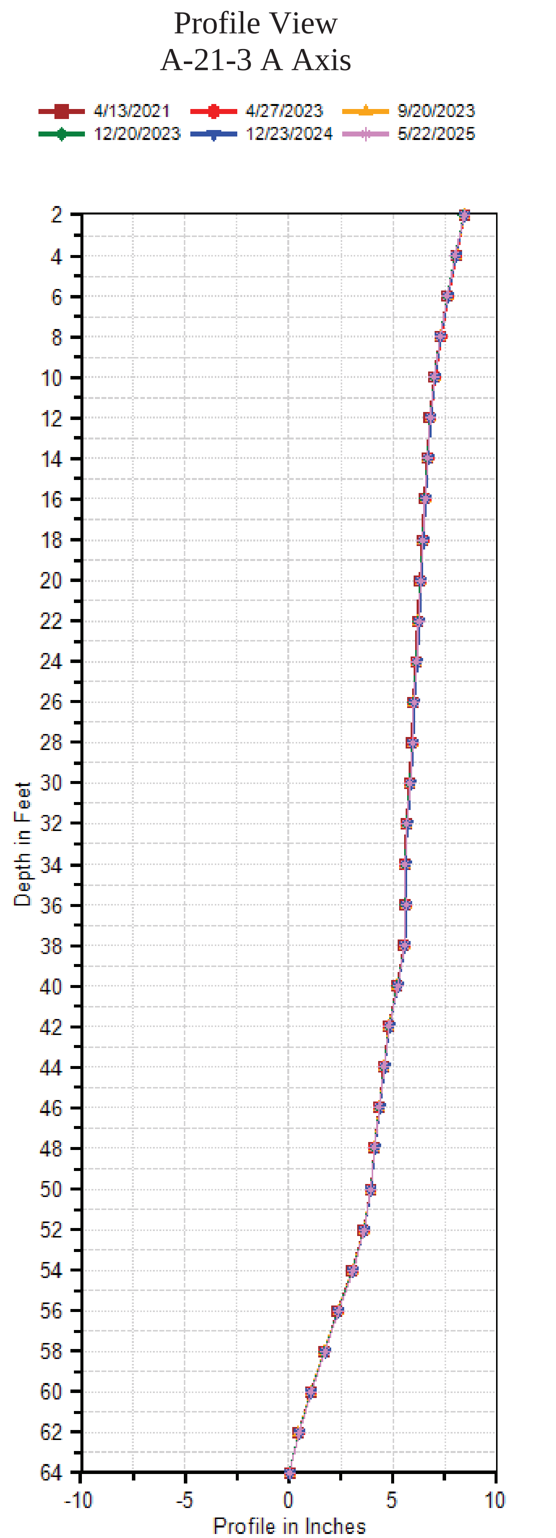


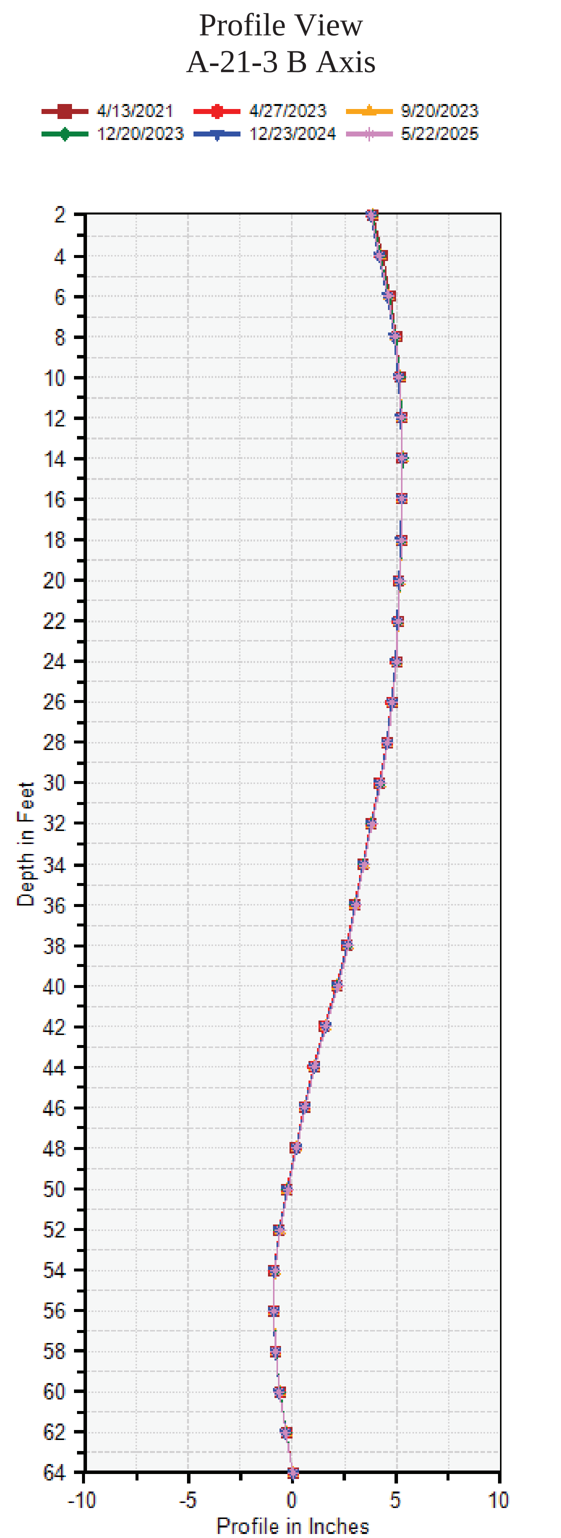
AECOM

Inclinometer Reading May 2025

Boring ID: A-21-2

*4/13/21 Baseline Reading



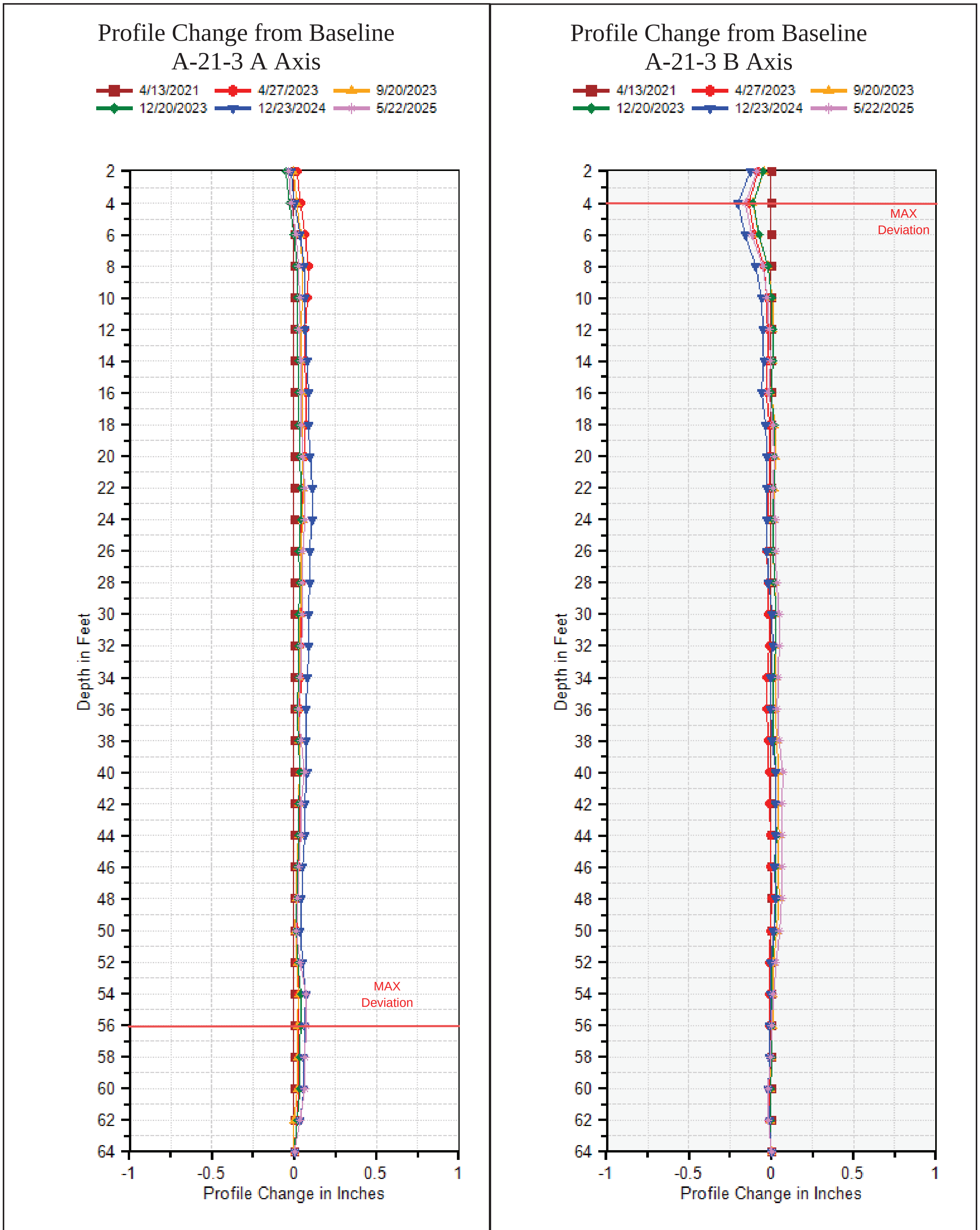


AECOM

Inclinometer Reading May 2025

Boring ID: A-21-3

*4/13/21 Baseline Reading



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Inclinometer Reading May 2025

Boring ID: A-21-3

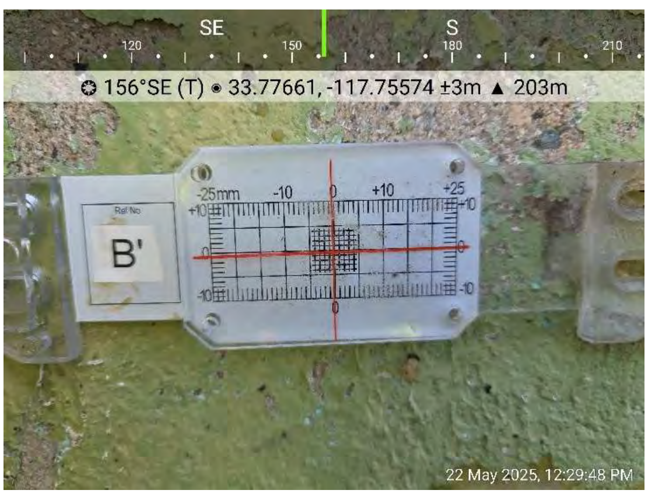
*4/13/21 Baseline Reading

Appendix B: Site visit Photographs

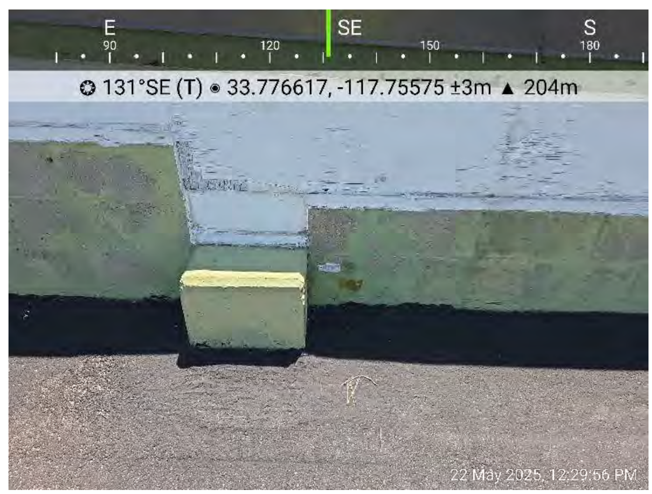
Appendix B-1: Crack Monitoring Photographs

Close up view: Portion of corner tale Crack Monitor A" missing

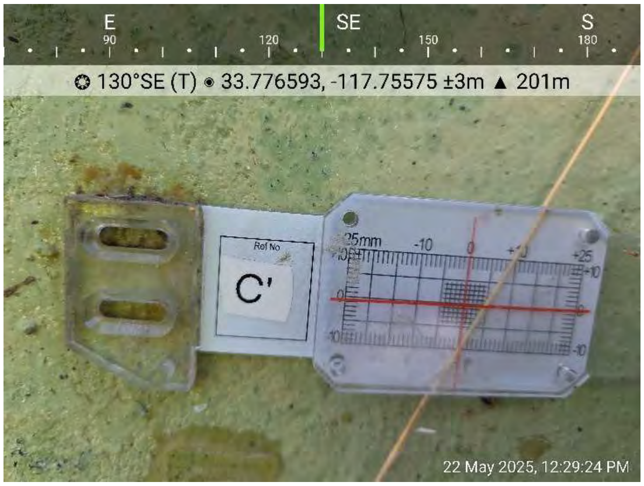
Far view: Portion of corner tale Crack Monitor A" missing



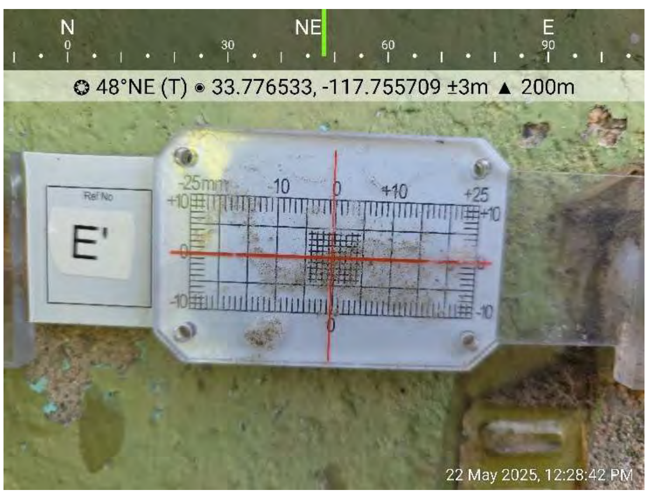
Close up view: Corner tale crack monitor B"

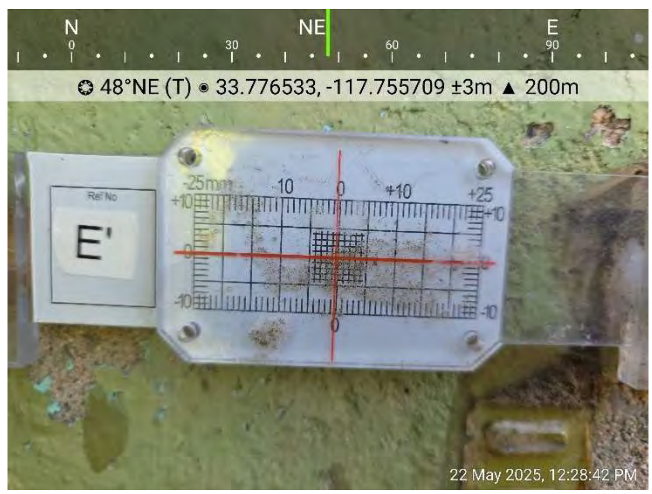


Far view: Corner tale crack monitor B" relative to pilaster



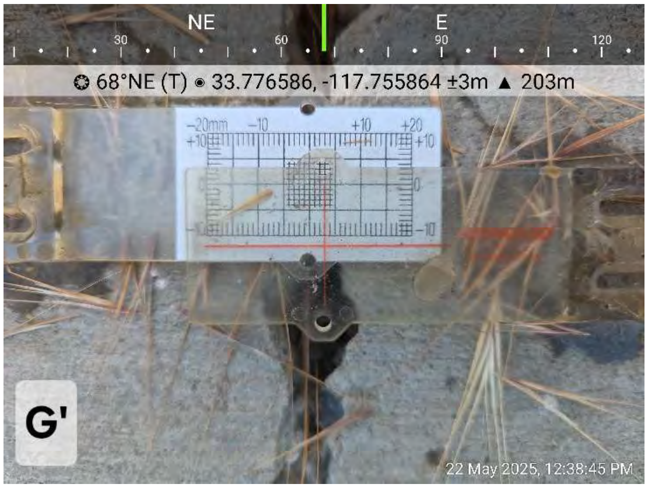
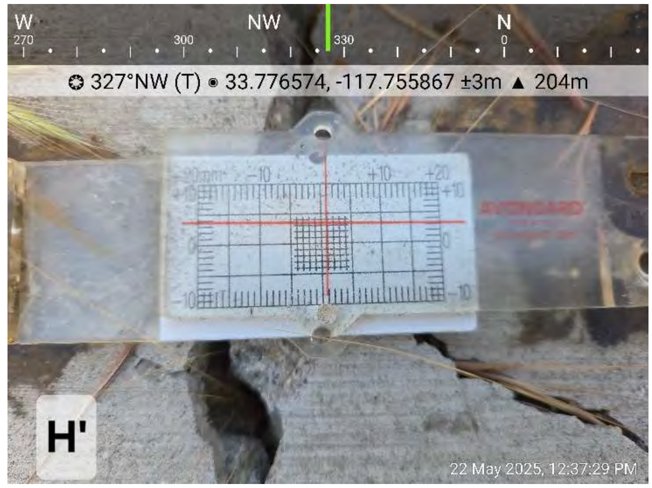
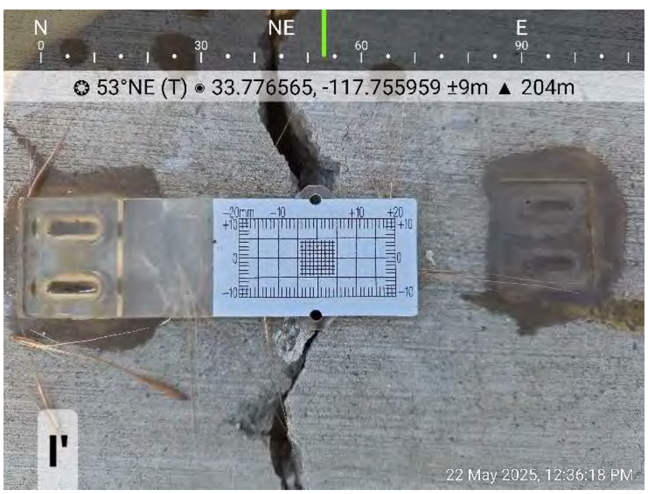
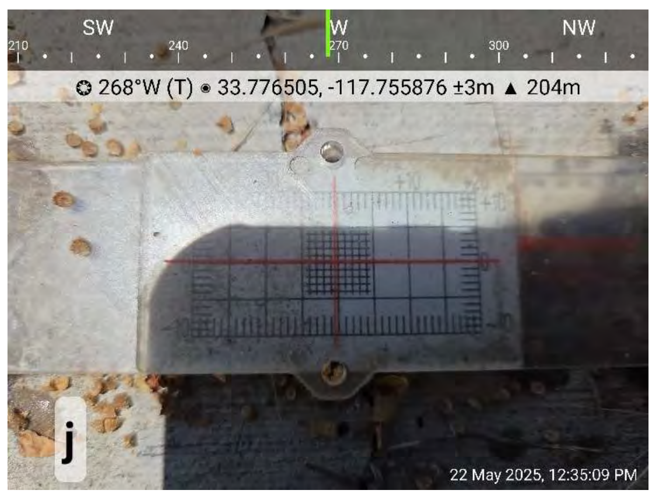
Close up view: Corner tale crack monitor C”	**Far view**: Corner tale crack monitor C” relative to pilaster





Close up view: Corner tale crack monitor E”	**Far view**: Corner tale crack monitor E” relative to pilaster

	
Close up view: Corner tale crack monitor F" with one side detached	**Far view**: view of F" relative to pilaster with one piece on top of pilaster

	
Close up view: observation of existing curb crack monitor	**Close up view**: observation of existing curb crack monitor
	
Close up view: observation of existing curb crack monitor	**Close up view**: observation of existing curb crack monitor

Appendix B-2: Asphalt Pavement Assessment Photographs

view: South end of site.

view: South end of site. Low severity longitudinal and transverse cracking

Close up view: observed low severity cracking near inclinometer boring A-21-1

View: low severity edge cracking along Southwest Curb

Close up view: minor edge cracking observed

Close up view: minor edge cracking measured. Less than 1”

view: minor cracking observed around drainage system

view: longitudinal cracking and raveling observed throughout asphalt surface.

	
view: Asphalt surface along the west end of the site	**view:** Asphalt surface along the North end of the site. Low severity cracking observed
	
view: Asphalt surface along the North end of the site. Low severity cracking observed	**view**: low severity cracks near north end concrete curb

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J.G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

EAST ORANGE COUNTY WATER DISTRICT

PETERS CANYON AND VISTA PANORAMA RESERVOIR

CONDITION ASSESSMENT

FINAL
April 2019

3150 BRISTOL STREET, SUITE 500 • COSTA MESA, CALIFORNIA 92626 • P.714.593.5100 • F.714.593.5101

EAST ORANGE COUNTY WATER DISTRICT

PETERS CANYON AND VISTA PANORAMA RESERVOIR

CONDITION ASSESSMENT

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PETERS CANYON AND VISTA PANORAMA RESERVOIR CONDITION ASSESSMENT

1.0 INTRODUCTION

1.1 Scope

East Orange County Water District (EOCWD) has tasked Carollo Engineers (Carollo) to provide a seismic evaluation and condition assessment of two of its reservoirs: the 6 million gallons (MG) Peters Canyon Reservoir and the 150,000 gallon Vista Panorama Reservoir. The scope of work is based on the proposal letter issued by Carollo Engineers, dated November 2014.

As outlined in our scope of work the following tasks were performed:

1.1.1 Task 1 – Peters Canyon Reservoir

- Review existing reports and drawings
- Perform additional seismic calculations
- Perform an on-site external condition assessment and a slope stability analysis
- Develop and analyze retrofit/replacement alternatives
- Prepare a technical memorandum summarizing our findings

1.1.2 Task 2 – Vista Panorama Reservoir

- Review existing reports and drawings
- Perform additional seismic calculations
- Perform an on-site external assessment
- Develop and analyze retrofit alternatives
- Prepare a technical memorandum summarizing our findings

1.2 Background

1.2.1 Peters Canyon Reservoir

The Peters Canyon Reservoir was originally designed and constructed in 1963. The reservoir, situated on a hillside in east Orange County, is located just east of Jamboree Road. The site can be reached by driving east on Handy Creek Road, a private access road which intersects with Jamboree Road. An aerial view of the site is provided on Figure 1.

The reservoir is a rectangular hopper-bottom shaped partially buried reservoir. The reservoir bottom and sideliner walls are constructed of 4-inch (in) thick reinforced concrete slabs with thickened edges at the perimeters. The reservoir plan dimensions are approximately 280-ft-4-in in the east-west direction and 170-ft 4-in in the north-south direction. The reservoir above ground consists of 4 ft tall, 8 ft thick concrete masonry unit (CMU) perimeter retaining walls with eccentric footings. These CMU walls have cast-in-place concrete pilasters at the main glulam beam and purlin support locations along the perimeter wall. The roof structure is comprised of sawn lumber purlins and glulam beams. At the interior of reservoir, the beams are supported on precast concrete columns, which in-turn are supported on concrete spread footings. The purlins span in the east-west direction and the glulam beams span in the north-south direction. All the wood members were preservative-treated (Wolmanized). In the east-west direction, additional bracing is provided by precast concrete diagonal braced frames. The roofing membrane consists of a Zip-Rib style aluminum decking, which is attached to the wood-framing members using sliding connections that limit the ability of the roof the decking to perform as a functional structural diaphragm. At the center of the roof in the east-west direction, the roof framing is raised to accommodate venting. The roof framing slopes to the north and to the south for drainage.

The concrete liner walls and concrete columns are overlain by a Hypalon liner installed in 1995. The bottom elevation of the reservoir slopes from 767.25 feet (ft) on the east side to 765.25 ft on the west side. The high operating water level is at elevation 790 ft, which is a minimum depth of 22.75 ft. The top of the perimeter CMU wall is at elevation 792 ft. A wood pony wall is built-up along the east and west walls of the reservoir. The pony wall is framed with wood studs and preservative treated plywood sheathing. The structural roof slopes up to the middle of the reservoir. The average height of the roof above the bottom of the reservoir is about 28 ft.

In 1996, portions of roofing and structural wood framing were significantly damaged from high winds resulting in roughly one-third of the roofing and a significant amount of wood framing requiring replacement. A seismic assessment was conducted in 1999, which resulted in a seismic retrofit in 2000. The retrofit consisted of adding steel rod rock anchors to the base of each of the concrete piers along the north and south perimeter walls only. The seismic assessment report was not available for review, but it appears that these rock anchors were added to increase the sliding friction capacity of the pier footing to resist seismic loads imposed by the glulam beams in the direction parallel with those beams.

Additional structural and seismic evaluations were performed over the last few years in response to observations made during prior dive inspections.

Figure 1
Aerial View of Peters Canyon Reservoir

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1.2.2 Vista Panorama Reservoir

The Vista Panorama Reservoir is a 46.5-ft diameter circular, conventional (non-prestressed) reinforced concrete tank with a wood-framed roof. The tank was built in the 1930's. The tank is above ground and the wall height is approximately 12 ft. The wood roof beams are supported on a total of 7 interior steel columns. The storage capacity of the tank is approximately 150,000 gallons. An aerial view of the reservoir is provided in Figure 2.

1.3 Cost Estimates

To assist EOCWD with their planning efforts in this evaluation report, major scope items were identified and rough-order of magnitude costs were estimated for each. Cost estimates provided in this evaluation/study are considered to be a Class 5 estimate as defined in "Recommended Practice 18R-97 Cost Estimate Classification System for the Process Industries," published by the Association for the Advancement of Cost Engineering International (AACE International). These costs are anticipated to have an accuracy range of plus 50 percent to minus 30 percent and are for intended for planning purposes. Cost estimates do not include soft costs, such as engineering consulting fees and permitting. Costs were estimated using the following resources:

- Our proprietary cost database
- Cost summaries from previous projects
- RS Means Heavy Construction Cost Data
- RS Means Building Construction Cost Data
- Manufacturer and contractor quotes

1.4 Data Collection and Review

Data and information necessary for use in the evaluation of Peters Canyon and Vista Panorama reservoirs, was obtained from the following documents provided in electronic format by from EOCWD:

- Construction drawings of 6 MG Reservoir and Filtration Plant, sheets 17 through 26, dated 1963.
- Construction drawings 6 MG Reservoir Liner, sheets 1 through 5, dated 1995.
- Construction drawings of 6 MG Reservoir Remedial Roof Construction, sheets S-1 and S-2, dated 1997.

Figure 2
Aerial View of Vista Panorama Reservoir

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- Construction drawings of Reservoir Improvements, sheets 1 to 6, dated 2000.
- Tank Inspection Report for Peters Canyon Reservoir, by Dive/Corr, Inc., dated January 2010.
- East Orange County Water District – Field Report, by Potable Divers Inc., dated July 2011.
- East Orange County Water District - Engineering Evaluation of the Peters Canyon Six Million Gallon Reservoir report, by Richard Brady & Associates, dated October 2012.
- Peters Canyon 6 MG Reservoir Structural Peer Review, by AARK Engineering, dated September 2013.
- Beam Inspection – East Orange County Reservoir; Orange, California report prepared by Western Wood Structures, Inc., dated November 2013.
- East Orange County Water District – Results and Recommendations for Peters Canyon Reservoir Roof report prepared by Richard Brady & Associates, dated September 9, 2014.
- Geotechnical and Seismic Hazard Assessment Report, Prepared by American Geotechnical, Inc., dated April 2, 2014.
- Retrofit Design of Vista Panorama Reservoir Report, prepared by Simon Wong Engineering, dated February 15, 2012.

1.5 Site Visits

Carollo conducted site visits for Peters Canyon Reservoir and Vista Panorama reservoir on Wednesday December 3, 2014, and Thursday December 4, 2014, to gather as-built information and structural configurations. At the time, both the reservoirs were in service and observations of the interior structure were limited. Conditions observed during these site visits are described in the following sections. Photographs taken during these site visits are included in Appendix B of this report.

2.0 PETERS CANYON RESERVOIR

2.1 Review of Existing Drawings and Reports

Based on the review of the existing structural drawings and the reports prepared by Richard Brady & Associates (Brady), the following detailed conclusions about the condition of the various structural elements were made.

We generally concur with the conclusions presented by Brady in their 2012 and 2014 reports with the exceptions and additional issues set forth in this report. We have also reviewed supplemental reports that were prepared by AARK Engineering and Western Wood Structures and we have also noted exceptions and potential issues upon which we have provided expanded discussions and recommendations.

2.1.1 Hinge Connector and Glulam Beam Condition

Brady identified that the glulam beams at the glulam hinge connectors are damaged and in need of immediate repair to prevent a roof collapse. The investigation made by Western Wood Structures in November 2013 indicated general deterioration of the top lamination due to rot and some limited lamination damage at the hinge connector. The investigation also noted that there appeared to be some rotation of the hinge connector and indicated that the rotation was likely due to the softening of the top bearing lamination of the cantilevered beam end. This rotation in turn is driving the hinge connector into the bottom lamination of the cantilevered end causing it to split.

It is not clear why the glulam beams are apparently vulnerable to rot. The original construction drawings specify the “framing lumber” to be “Wolmanized treated,” which is a wood preservation process that adds chromate copper arsenate (CCA) as a pesticide to protect the lumber from rot and decay. It is possible that the glulam beams are not treated with this preservative. A simple visual observation can confirm whether these beams were treated or not. Lumber that has been treated will have small uniformly spaced incising marks (narrow thin penetrations) on all of the wood surfaces. Smaller lumber sections may appear to have a green tint to them as well.

If the glulam beams were not treated with any wood preservative, the glue used to structurally bond the individual beam laminates together might not be of a sufficiently durable grade for the environment within the reservoir. Degradation of the laminate glue can lead to structural failure of the beam. Delamination has not been identified in any surveys thus far.

Entry into the reservoir to observe framing conditions inside was not within our scope of work. If further information regarding the glulam beam conditions is desired, we recommend an interior survey be made of the roof framing. Since the laminations may continue to deteriorate, the damaged hinge connectors should be corrected as soon as practicable.

2.1.2 Roof Framing Hardware Corrosion

During our site visit, we viewed the interior of the reservoir immediately adjacent to the access door located at the west end of the reservoir. Corroded nails were observed at a few of the joist hangers, but the nails were in place. No other evidence of severe corrosion was observed. EOCWD staff provided a photograph of a collection of metallic debris that included stainless steel screws and heavily corroded nails that were retrieved from the bottom of the reservoir during the last cleaning, which occurred in 2013. It is not known if this debris is indicative of an on-going problem with connection hardware corrosion or if the debris was simply construction litter.

Given the age of the reservoir and the high relative humidity within the reservoir, it is likely that the roof framing fasteners that are embedded within the wood have corroded to some extent. Our visual observations are limited and we can only assume that corrosion is active. Corrosion of metals in wood will continue unabated inside of the reservoir.

Metals that are embedded in wood can experience accelerated corrosion rates compared to that of typical atmospheric corrosion, since the wood can retain moisture and also serve as an electrolyte, allowing the chemical reactions to occur. The following conditions are considered to accelerate corrosion of embedded metals in wood:

- A wood moisture content that exceeds 15 percent
- Relative humidity that exceeds 70 percent
- Use of wood preservatives that contain copper ions
- Use of dissimilar metals (hangers and fasteners of different materials)

The investigation made by Western Wood Structures indicated that the wood moisture content, the greatest single driving factor in corrosion of embedded metals in wood, was measured to be 6.5 percent at the time of the investigation. These conditions can change. The exposed ends of the glulam beams, if permitted to get wet, can wick large amounts of moisture into the beam along its full length. We recommend waterproofing the glulam beam-ends by providing a protective covering or mastic coating that will prevent water from entering in through the beam ends. The existing beam-ends are currently exposed and appear to be painted. Some appeared to be wet during our site visit.

It should also be pointed out that the aluminum Zip-rib roof system may include aluminum fasteners. Product literature for the wood preservative (CCA) used in the roof framing members, indicates that aluminum should not be used in direct contact with wood that has been treated with CCA. We understand that the aluminum can deteriorate when placed in contact with CCA.

We recommend that a corrosion survey be made of the interior framing connections within the interior of the reservoir. The conditions within the reservoir are conducive to corrosion and some evidence has been visually observed suggesting that fasteners are corroding and failing. Since the framed lumber members are supported on the glulam beams using face-nailed hangers, nail failure can result in collapse of the joist.

2.1.3 Reservoir Roof Structure Seismic Load Path and Structural Diaphragm

Wood-framed roofs are typically encountered in buildings, where plywood sheathing is provided and serves to transfer seismic loads acting on the roof structure to the lateral load resisting system by diaphragm action. This is the most common framing method to transfer seismic loads for a wood-framed roof. However, while a diaphragm can efficiently transfer these loads, not all wood-framed roofs require a diaphragm to transfer seismic loads. Often, wood roofs over reservoirs are designed such that seismic loads are transferred from the roof framing members through weak-axis bending (bending sideways). This can be feasible when the roof framing weight is relatively light and spans are not excessive. These loads are then transferred into beam lines that are braced by perimeter walls, drilled piers, large pilasters, or other lateral load resisting elements. This system can be present in both orthogonal directions.

Based on our work on existing reservoirs of similar design and construction, the seismic load paths provided for the roof structure usually consists of using the roof wood joists and beams as the main seismic load transferring members. For the Peters Canyon Reservoir, in the north-south direction, the original seismic load path to support the roof is provided by transferring the roof seismic loads generated in each bay, along each of the column lines, by imposing axial loads into the wood members. This will create tension and compression axial loads in the wood members based on the direction of seismic loads. The wood beams transfer these axial seismic loads to the perimeter concrete piers as an out-of-plane lateral load. The piers and the footing resist the loads and transfer them into the soil. For the wood beams to act as axial tension and compression members, the seismic load has to be transferred to these main column lines. This can be achieved by providing a structural diaphragm to transfer the seismic mass to these column lines within each bay. Alternatively, if no structural diaphragm is present, the seismic roof mass loads can be transferred by out-of-plane bending and/or through axial loading of individual purlins. If the wood members have sufficient capacity to resist the out-of-plane bending and axial loads, then a structural diaphragm for seismic loads may not be required to complete the seismic load path.

The seismic load path in the east-west direction is similar except that the seismic loads are transferred through the roof joists in axially to the roof beams, which then bend about their weak axis and transfer the loads to “collector” lines spaced at about 40 ft on center. These collector lines (total of 3 at the interior) are comprised of glulam beams that transfer loads axially into chevron-style braced frames (inverted V’s) that are comprised of precast concrete bracing members. Seismic loads are then transferred into the soil via isolated pad footings. The north and south walls are CMU walls, which resist east-west seismic loads in shear.

While the transfer of seismic loads using weak-axis bending and axial load transfer (as opposed to using a diaphragm) appears to be the intent of the original structural engineer, alternatively, the tension compression in wood members can be ignored and a structural diaphragm can be provided with sufficient capacity to transfer the seismic lateral loads to the perimeter 8-in thick CMU walls for the loads acting in each direction.

The seismic load values have increased over the decades based on advances in knowledge and information about new faults and intensity of earthquakes and the response of buildings to these earthquakes. The reservoir roof structure will most likely be found to be deficient if the seismic loads calculated per the 2013 CBC are considered. A more detailed analysis and capacity checks have to be performed if the existing structural roof system is to be left in place and a retrofit strengthening option is chosen as the ideal solution to this reservoir.

In a typical roof structure, structural diaphragms perform two main functions. The first being that, in high seismicity areas a structural diaphragm acts as the main horizontal lateral load transfer element to transfer the seismic load from the roof structure mass to vertical lateral load resisting elements. The second function of a structural diaphragm is provide lateral bracing to wood purlins, beams and girders to prevent compression edge lateral buckling when these members are resisting bending in their strong axis due to gravity, wind, and seismic loads. The National Design Specification for the design of wood members sets forth prescriptive recommendations for providing stability of bending members. Accordingly, 2 by 12 and 2 by 14 purlins will require a structural diaphragm on the compression edge and full depth blocking or diagonal cross bracing at every 8 ft on center to prevent lateral buckling under design loads. For the 3 by 14 and 5.25 by 21 glulam beams, no compression bracing is needed due to their smaller depth to width ratios which makes them more stable against lateral buckling. Note that under gravity dead and live loads the compression edge is on the top of the wood members. Under uplifting wind loads there could be a net tension uplift loads on the wood members causing compression on the bottom edge of the members. To prevent lateral buckling under wind loads additional full depth blocking may be needed at the 2 by 12 and 2 by 14 purlins as providing a structural diaphragm on the bottom edge of these members will not be practical. This lateral buckling might have been one of the reasons why roof purlins failed in 1996 under high wind loads. Use of full-depth blocking is standard practice and this reservoir appears to lack this provision.

Additionally, a structural diaphragm on the top edge will prevent weak axis buckling of 2 by 12 and 2 by 14, 3 by 12 and 3 by 14 purlins if axial loads are imposed on these members. The Zip-Rib aluminum roofing is not considered to be a structural diaphragm as it is not rigidly connected to the purlins and has sliding connections to let it expand and contract due to temperature changes.

There are two options for providing a structural diaphragm: a galvanized corrugated metal decking or plywood sheathing. The plywood sheathing if used has to be preservative treated to prevent decay due to the presence of moisture. Wood preservatives are pesticides that protect wood against attack by fungi, bacteria, or insects. The active ingredients found in wood preservatives may include pentachlorophenol (pent or PCP), creosote, copper, zinc, chromium, arsenic and other compounds. Use of wood preservatives is now restricted and regulated by the Environmental Protection Agency (EPA). The EPA has published recommendations regarding the use of various types of preservatives and has concluded that they should not be used where they may come into direct or indirect contact with public drinking water. Since water condensation often accumulates under the roof deck and the roof deck may leak when it rains, wood preservatives in the wood framing can potentially leach into the water, albeit in small amounts at a time. If the roof structure were to collapse into the reservoir due to an earthquake, the wood preservatives can leach into the water at potentially higher rates than in the supported position out of the water. Marine-grade plywood is a durable product, but manufacturer's recommend that this plywood be preservative-treated when its use will be subject to long-term exposure to moisture.

Due to the above-mentioned reasons, preservative treated plywood sheathing is not recommended to be used as a structural diaphragm for the Peters Canyon Reservoir. Instead, a galvanized metal decking should be used as a structural diaphragm.

Note that based on the existing drawings, the original wood members and plywood sheathing used around the perimeter were treated with a wood preservative called Wolman CCA (chromated copper arsenate). As mentioned above the EPA does not allow the use of preservative treated wood near public drinking water. Furthermore, the EPA has established guidelines for the safe handling and disposal of preservative treated products. Mitigation strategies that involve the demolition of the existing roof framing may need to specify disposal requirements and limitations regarding recycling of the wood framing members.

We contacted Western Wood Preservers Institute (WWPI) and we were informed that, given the age of the structure, any leaching of the wood preservative is likely to have diminished significantly since the time of construction. However, it is not known if leaching would occur and at what rate, if the wood framing were to collapse into the reservoir. It should also be noted that the repairs made in 1997 to the wind-damaged roof sections involved the introduction of new replacement wood framing that was treated with a wood preservative, which is assumed to be CCA.

It is advisable to conduct testing to verify the content of CCA in the existing wood-framed members. The American Wood Protection Association (AWPA) has established wood testing procedures that can be used to help identify the content of preservatives in wood. Testing for chemical content in water that has been exposed to CCA treated lumber may be an alternative means to verify if this is a valid concern or not.

In lieu of using wood preservatives, newer reservoir roof covers that have been constructed of wood framing have used sawn lumber and glue-laminated beams that have a natural resistance to decay. Redwood and Alaska Yellow Cedar are two wood species that have a natural resistance to decay. The Van Norman reservoir, owned and operated by the Los Angeles Department of Water and Power (LADWP), was constructed in August of 1992. LADWP engineers did not want to introduce additional chemicals to the water and chose to use Alaska Yellow Cedar, a naturally durable species of wood that is commercially available for use in structural wood framing applications.

2.2 Additional Seismic Calculations

2.2.1 Sloshing Wave Height and Freeboard

The sloshing action of the water within the reservoir during an earthquake can generate a maximum wave height at the perimeter of the structure. When insufficient freeboard is provided, the water can slosh and surcharge the bottom side of the roof framing at or near the perimeter of the structure. The surcharge force will be directly proportional to the amount of freeboard deficit. The associated loading to the underside of a roof structure can be substantial and cause significant damage or collapse, especially when the roof is framed with a lightweight material.

Since most reservoirs operate at their high water level for substantial amounts of time, the wave height was estimated assuming the reservoir is full. This wave height was determined using the equations and procedures set forth in American Society of Civil Engineers (ASCE) 7-10 and American Concrete Institute (ACI) 350.3. The higher of the two values determined from these two different codes was used conservatively for this evaluation. The sloshing wave height and the required freeboard were estimated to be 37 in. Currently the freeboard available to the underside of the aluminum roofing is 34 in. Given this small amount of surcharge, damage to the roof framing is not expected to be substantial at the sloshing levels estimated.

2.2.2 Soil Rod Anchor Retrofit at North-South Wall Piers

In 1999, steel rod rock anchors were installed at 24 pier footings along the north and south walls. The pier footings were also extended to accommodate the anchors. It appears that these rock anchors were installed to increase the frictional lateral load carrying capacity of the pier footing to resist lateral loads imposed by the glulam beams in a seismic event. The rock anchor provides a net vertically downward acting force on the pier footing which will result in a net increase in frictional capacity of the pier footing. We estimated that the lateral load imposed by the glulam beams on each of the pier footings is approximately 8.6 kips at service load level. The rock anchor with 30 kips of net axial design load, as shown on the structural drawings, is estimated to provide up to 13.8 kips (service level) of frictional capacity to resist the lateral load, assuming a soil friction coefficient of 0.3.

The reaction at the base of the pier footing can be applied in a direction toward the reservoir or away from the reservoir. For loads applied away from the reservoir, the full frictional resistance is expected to be developed. However, in the direction towards the reservoir, the bottom of the footing butts up against the reservoir sloped liner. Development of the full lateral load capacity of the footing is questionable and loading of the piers during an earthquake can potentially result in direct loading of the sloped liner, which can cause cracking or more significant damage to the liner.

The selection of rock anchor retrofit confirms the tension-compression axial load seismic load path described in Section 2.1.1 was used by the structural engineer to resist seismic loads in-lieu of providing a structural diaphragm.

2.2.3 Seismicity due to Recently Studied faults

Carollo Engineers contracted with Converse Consultants to evaluate the seismic design parameters for the Peters Canyon Reservoir site taking into account recent fault data for the nearby Peralta Hills Fault. The results of this study are provided in Appendix C to this report. The ground accelerations determined from this study are slightly higher (about 10 percent) than those values obtained through the current building code and the United States Geological Survey data.

Any future seismic evaluations, retrofit designs, and/or new designs at the site should use the values determined by this study.

2.3 Onsite External Assessment and Slope Stability Analysis

The exterior assessment of the reservoir was limited to a visual assessment of the reservoir from what could be seen from a walk around the perimeter of the reservoir. In general, the roofing appears to be in good condition with evidence of localized sealant applications. The following observations were made:

- The plywood panels located along the east and west sides of the reservoir are in relatively poor condition and appear to have evidence of moisture damage and potential rot. Replacement is recommended as conditions worsen. However, panels should not be replaced with preservative-treated members for reasons previously discussed. We recommend that panels be replaced with Marine grade plywood or an alternative material that is more durable. Marine grade plywood will likely last much longer than conventional plywood, but replacement should be anticipated at some time in the future as well due to degradation.

- The glulam beam ends that are exposed on the north and south walls of the reservoir are painted and flashed around. The paint and caulking on the south side are in poor condition. We recommend capping the ends of the glulam beams with a heavy mastic coating and replacing the flashing with a durable material.

Carollo Engineers contracted with Converse Consultants to review a previous slope stability analysis prepared by American Geotechnical in April 2014 and incorporate new fault data for the Peralta Hills Fault. The results of that review are presented in Appendix C of this report.

2.4 Develop and Analyze Alternatives

2.4.1 Alternative 1 – New Aluminum Roof – System Vulnerability

In this alternative, a new aluminum roof structure will be installed with its own structural column structure for gravity support and lateral system for seismic load resistance. All existing components of the existing roof, including wood glulam beams, purlins, and concrete columns will be demolished and removed from the site. It was estimated by Brady that the total construction time for this option would be nine months.

Replacement of the existing reservoir roof assumes that the footprint of the existing reservoir would remain unchanged. The cost to re-size the reservoir was not considered in the estimate provided by Brady. Such work would involve additional demolition and earthwork that will increase the cost and extend the duration of construction. This alternative, as it is currently being evaluated, would not provide additional space on the site for future development. Alternatives that involve replacement of the reservoir with a circular tank inherently allow for site re-development.

2.4.2 Alternative 2 – Replacement with New Prestressed Concrete Reservoir

We reviewed the quotation from DN Tanks for a replacement reservoir dated September 28, 2014 and the Brady update report from September 2014 in regards to the budget and construction duration. As noted by Brady a new prestressed concrete reservoir will provide the highest reduction in seismic risk and a reliable source of water storage for a service life of at least 50 years.

The following elements need to be considered while evaluating the replacement PT tank alternative.

- Temporary isolation of Peters Canyon Reservoir
- Demolition of the existing reservoir
- New piping
- The prestressed tank diameter is 165 ft and hence the hydraulic grade line will need to be raised to 803 ft to maintain the same volume. The current hydraulic grade line is at 790 ft. This higher-grade line may require installation of a pressure-reducing valve.
- The smaller size prestressed concrete tank can be located towards the east or west end of the current reservoir and the additional space of approximately 140 ft can be used for other purposes.
- Tank appurtenances
- Backfill and site work

In order to mitigate the loss of reservoir capacity during construction, a much smaller temporary bolted steel tank or tanks can be provided on the site to meet the emergency/mandatory water supply requirements while the prestressed concrete tank is constructed. This assumes that the water storage capacity required to be maintained at the site is a fraction of the full storage volume.

Figures 3, 4, and 5 show a conceptual layout and sections of a new prestressed concrete tank describing the construction work anticipated. We used the layout shown in these Figures to estimate the rough construction cost. We located the tank towards the west end of the existing reservoir to provide additional space on the east end, which could be valuable space used for other necessary site developments, such as new or expanded water treatment facilities.

For planning purposes, the following simplifying assumptions have been made in the development of our cost and schedule estimate:

- The existing soils require no additional improvement or replacement.
- Deep foundations, such as piles, are not required.
- The cost of temporary steel bolted tank(s) was not included.
- The replacement reservoir will be partially buried.
- The bottom of a new reservoir will not be any deeper than the existing reservoir.
- Water quality improvements, provided by baffling and mixing, for example, are not considered.
- Dewatering is not considered.
- Cost of a pressure relief valve was not considered.
- Soft costs for permitting, engineering, etc. are not included.

Based on our estimates the total construction cost for a new prestressed concrete tank having a storage capacity of 5.8 MG along with the associated infrastructure will be $8.1 million. This cost involves a substantial amount of earthwork because the layout considered pushes the tank to the west to maximize space on the site to the east. If a smaller volume tank is feasible, the project cost would be reduced due to a smaller tank size and potentially less earthwork. Other potential tanks sizes that can reduce earthwork, increase available space on the site, and reduce the project cost are as follows:

- A 165-ft diameter prestressed concrete tank with a storage capacity of 4.0 MG with the high operating level matching existing (cost estimate of $6.4 million).

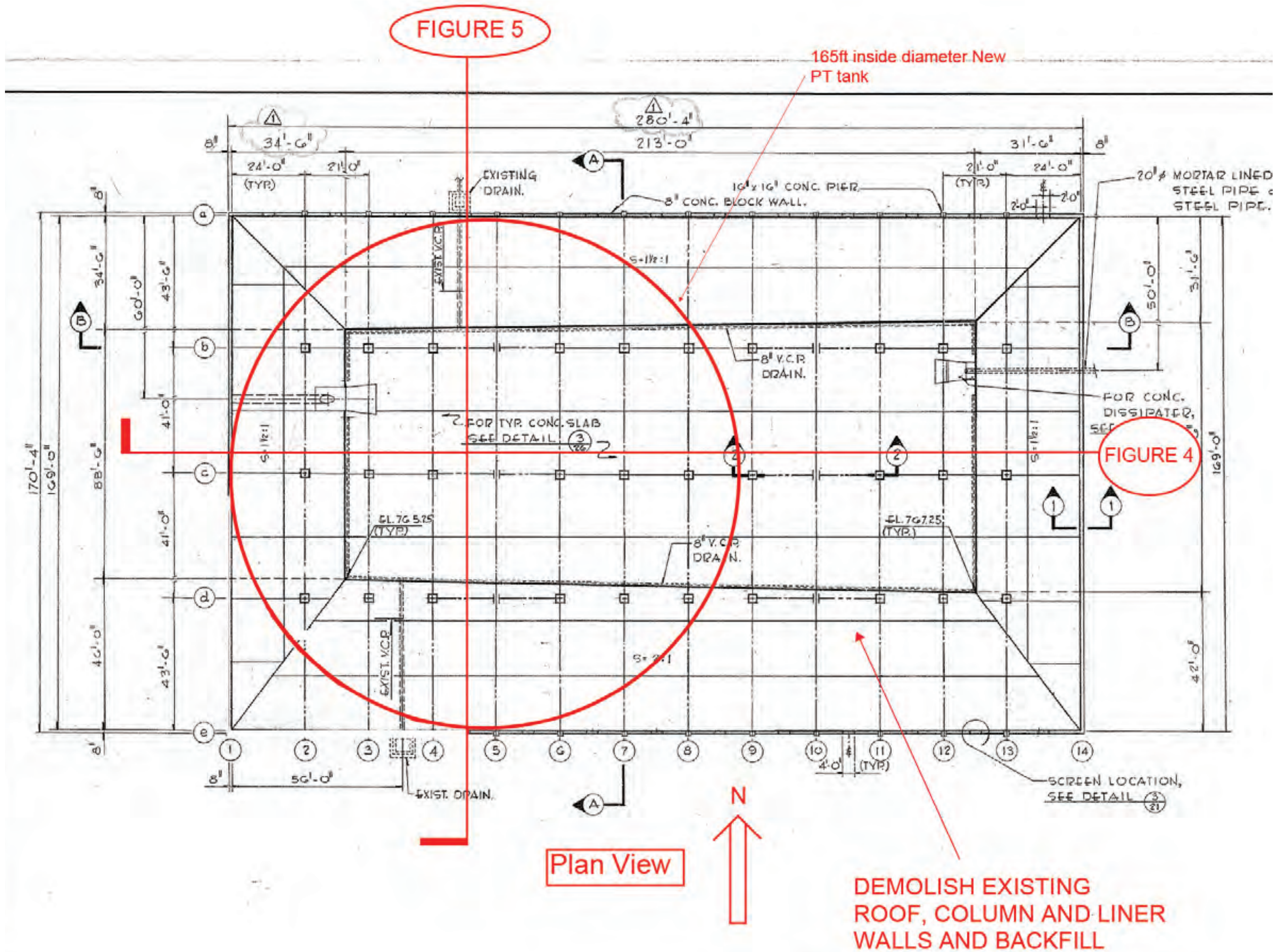

Figure 3
Peters Canyon Reservoir - Layout of New PT tank

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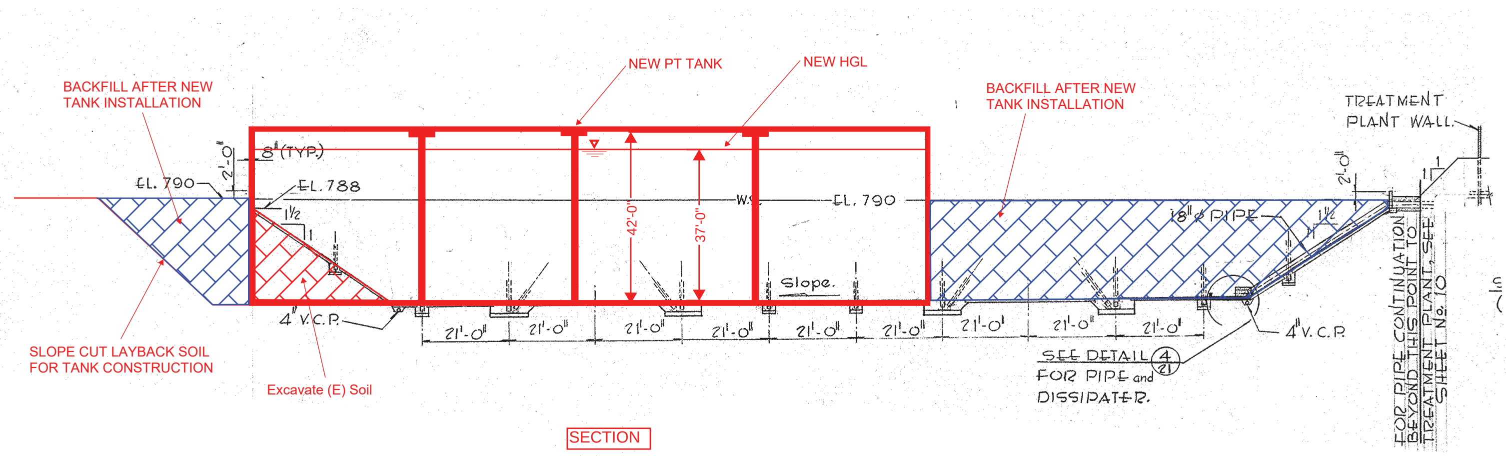

Figure 4
Peters Canyon Reservoir - East West Section of New PT tank

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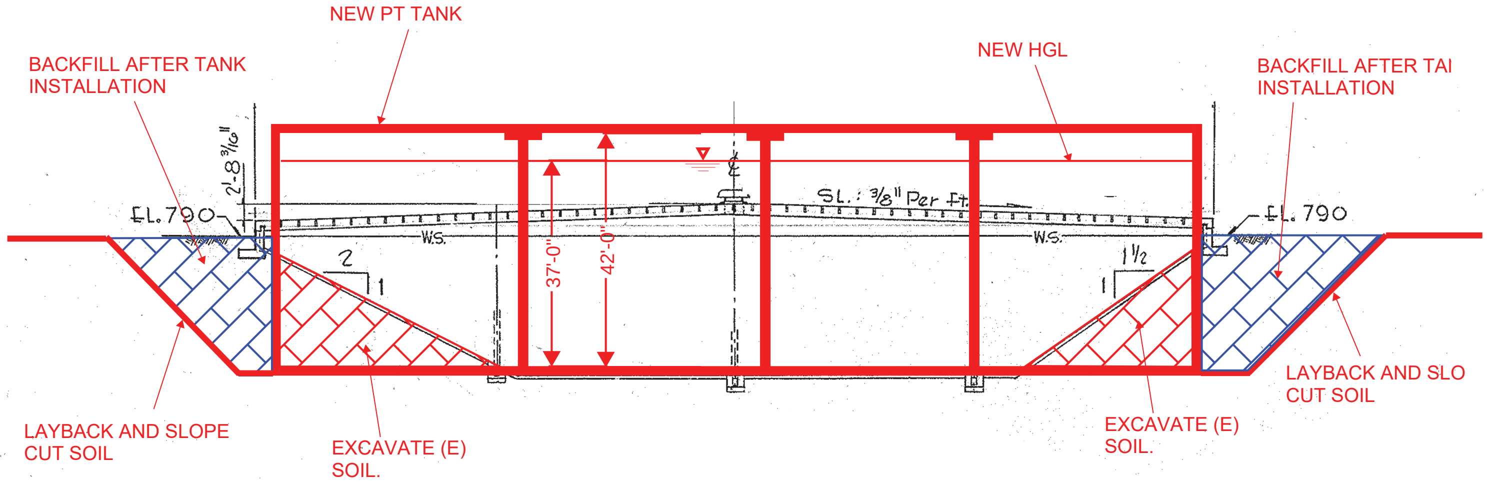

Figure 5
Peters Canyon Reservoir
-North South Section
of new PT tank

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- A 143-ft diameter prestressed concrete tank with a storage capacity of 3.0 MG with the high operating level matching existing (cost estimate of $5.6).

We estimate that the construction of a new prestressed concrete tank will take about 12 months excluding the construction of a new road around the tank. Therefore, depending on the tank size and layout of the tank at the site, the cost to replace the existing reservoir with a prestressed concrete tank can vary from $5.6 million to $8.1 million.

2.4.3 Alternative 3 – Flexible-membrane Floating Cover - Hypalon

Flexible-membrane linings and floating covers can be incorporated into many types of water-storage facilities, both new construction and rehabilitated structures. The water industry began using floating covers using ethylene-propylene-diene monomer (EPDM) synthetic rubber and less commonly polyvinyl chloride (PVC), during the mid-1960s. Since then, improvements in design, materials, and operating procedures have resulted in hundreds of successful floating-cover installations in the United States and throughout the world. In the United States, more recently, most have been constructed using reinforced elastomeric membranes of chlorosulfonated polyethylene (CSPE-R), commonly know as Hypalon™. The following sections provide a detailed review of Hypalon as a floating cover, feasibility of using it at Peters Canyon, and a cost estimate for planning purposes.

2.4.3.1 Background Information

Hypalon is the registered trademark for a series of chlorosulfonated polyethylene (CSPE) synthetic rubbers manufactured by DuPont Dow Elastomers. Vulcanizates of Hypalon are highly resistant to the deteriorating effects of ozone, oxygen, weather, heat, oil, and chemicals. It can be compounded to give excellent mechanical properties-for example, high tensile strength and abrasion resistance. Several types and grades of Hypalon are available for a variety of end use requirements. One such end use is in the potable water industry for lining and floating covers over reservoirs. Hypalon used as a floating cover is manufactured into plies that are combined over reinforced polyester scrim layer or layers. It is available in up to 5 ply layers. Hypalon as a floating cover is available in 36, 45, and 60 mil thicknesses and up to 90 mils by special order.

Hypalon as a floating cover is fabricated by heat welding smaller panels into larger panels in a fabrication facility. The panels are finally field seamed together into one large cover using either heat-welding or solvent adhesives.

The two most common floating cover designs are the “defined sump” and “Mechanically Tensioned”. The defined sump covers use floats and weights to create rainwater collection sumps in the cover and to accommodate changes in water level. Tensioned covers use cables attached to tensioners to control slack as the cover moves up and down. Tensioned covers are mechanically tensioned around the periphery, which eliminates sumps and the main panel cover is maintained flat.

Hypalon floating covers have been used by numerous water districts within United States and elsewhere successfully for about 35 years. Below are a list of general advantages and disadvantages associated with Hypalon floating covers:

Advantages

- The capital cost of fabrication and installation is low compared to any other traditional structural roof system.
- Hypalon is flexible and hence easy to install and can be installed in a short duration of time.
- Hypalon is durable and possess desirable properties for a potable tank cover, such as resistance to chemicals, extreme temperatures, and UV degradation.
- Manufacturers provide standard warranty and extended warranty can be negotiated at the time of order for the material. Warranty for material from the manufacturer is typically 30 years. The warranty from the bonding installer is about 1 to 2 years.
- Hypalon covers have proven repair and maintenance procedures.

Disadvantages

- The service life of a floating cover is around 20 to 25 years, which is shorter than a service life of a new structural roof framing system.
- Floating covers will have additional life cycle costs associated with operation and maintenance. Some of these include regular inspections, dewatering of rain water, disinfection of interior covers, cleaning of the cover regularly to remove foreign objects, inspection of pumps and auxiliary equipment, immediate repair of pin-holes, etc.
- Although some floating-cover designs incorporate inflation or suspension as a means of providing access for interior cleaning, operators should remember that even a moderate wind could cause severe damage or destruction of the cover under certain conditions.
- Modifications to the existing reservoir have to be performed to accommodate a floating cover roof. Some of the modifications may involve new concrete curbs, chafing strips, baffle-walls and perimeter anchor posts.
- Maintenance crew and personnel have to undergo training to safely and properly operate a floating cover roof.
- Hypalon covers need to be inflated for inspection and cleaning inside of the reservoir.

- Inspection of the floating cover may be required on a frequent basis to immediately repair any damage to the floats and membrane for proper operation of the floating cover.

Cover owners may choose to use their own crew to perform yearly maintenance or they can hire the installer. A detailed non-technical account about the design, operation, and maintenance is provided in AWWA Manual M25, Flexible-Membrane Covers and Linings for Potable-Water Reservoirs. It is highly recommended that EOCWD personnel review this document to get an understanding of installation and operation of a floating cover system before a decision is made to pursue this alternative.

2.4.3.2 Peters Canyon Reservoir Floating Cover Feasibility

Based on our review of the existing Peters Canyon Reservoir and the site around, it is our conclusion that a Hypalon floating cover is a feasible alternative provided it is properly designed and installed. The drawings and specifications for a cover system should be prepared by a licensed engineer having specialized knowledge and experience in the design and operation of flexible-membrane floating-cover systems. The following list provides a summary of the things to consider for designing, installing and operating a floating cover specific to this reservoir. The typical advantages and disadvantages associated with floating covers described in the previous section should be considered in conjunction with the following:

- Demolition of the existing structural roof for the installation of a new floating roof cover will likely damage the existing Hypalon liner on the floor and slopes of the reservoir. A new Hypalon liner on the reservoir floor and slopes may need to be installed to replace the damaged liner.

- Anticipate improvements and repair of the existing concrete liner slab and piping.

- The concrete columns have concrete fill around them at the column foundation. The removal of the concrete column will require placing of new concrete fill and joint at these locations.

- Along the perimeter 4-ft tall CMU wall, existing wall vent openings and voids created by removal of Glulam beam seats and purlins will need to be infilled with concrete.

- Existing perimeter CMU wall will have to be checked for the loads imposed by the Hypalon cover to verify if any additional strengthening of the wall is required.

- 10-ft of access around of the perimeter of the reservoir will be required for installation of the floating cover. This may require removal of existing soil and installation of shoring on the east side of the reservoir adjacent to the existing water treatment plant.

- Environmental conditions such as local vegetation, nearby trees and likely tree debris, animals, smog and other conditions should be reviewed.
- If the surrounding area is prone to animal intrusion that could cause damage to the cover, a security fence should also be designed and provided.
- The slopes of the reservoir are 1.5:1 (horizontal to vertical) on three sides and 2:1 on the fourth side. The ideal slope for maintenance crews without needing any special harnesses would be 2:1 and flatter (moderate slope). So on the three surfaces where the slope is 1.5:1, safety gear may need to be incorporated for maintenance crews to access the reservoir cover.
- The required minimum freeboard for a sloshing wave due to the design-level earthquake for this reservoir is approximately 2 ft in the east-west direction and 3.1 ft in the north-south direction. Floating covers are not typically designed for surcharge loads from sloshing waves. If the floating cover is installed such that the top of the cover is flush with the top of the perimeter wall at elevation 792 ft, the water level will have to be dropped down from current operating level of 790 ft to 789 ft. This will result in a loss of about 356,000 gallons of storage capacity.

We estimate that the total construction cost of a new floating cover will be on the order of $2.4 million. In addition a yearly operation and maintenance cost $12,000 per year shall be considered. In addition, the service life of a floating cover is around 25 years. Since other structural replacement alternatives have a greater expected service life, for comparison purposes, an additional replacement cost should be considered. We performed a life cycle cost analysis over a 40 year interval that includes operation and maintenance costs and capital costs. An interest rate and inflation rate of 3 percent and 2 percent were assumed, respectively. Based on our calculations, an additional $2.7 million should be added to the current capital cost to account for the replacement of the floating cover at year 25. Therefore, the total cost of the floating cover reservoir will be on the order of $5.1 million. We estimate that the total construction time needed for installing a floating cover will be approximately five months.

The floating cover system that was considered in this evaluation assumes that the cover would be placed over the footprint of the existing reservoir. The cost to resize the reservoir was not estimated. Such work would involve additional demolition and earthwork that will increase the cost and extend the duration of construction. This alternative, as it is currently being evaluated, would not provide additional space on the site for future development. Alternatives that involve replacement of the reservoir with a circular tank inherently allow for site redevelopment.

The cost of providing temporary tankage during construction was not evaluated for the floating cover or the prestressed concrete tank alternatives.

3.0 VISTA PANORAMA RESERVOIR

3.1 Review of Existing Drawings and Reports

Based on the review of the report prepared by Simon Wong Engineering (SWE) and our site visit, the following are our detailed conclusions about the condition of the various structural elements. We generally concur with the conclusions presented by SWE in their report with the exceptions and additional issues set forth in this report. The SWE proposed recommendations to mitigate leakage in the tank is a feasible alternative.

3.1.1 <u>Existing Wall Thickness and Reinforcing</u>

There are no structural drawings available to review the construction details of this reservoir. The SWE report notes that the thickness of the tank walls are 12 to 13 in with one layer of horizontal reinforcing based on a concrete core sample provided by EOCWD. The core sample indicates that the concrete tank wall has one horizontal layer of 5/8 in square rebar at 4 in on center located near the outside face of the wall with 1 in of concrete cover. There was no vertical rebar present in the core sample.

We measured the thickness of the concrete wall by measuring the distance of the wall exterior and interior faces relative to the roof hatch opening. Based on our measurements the wall thickness is approximately 8 in. We conclude that the concrete core may not be representative of the Vista Panorama tank walls. Furthermore, the surfaces of the concrete core sample did not appear to match that of the tank, which is painted pink. Also, evidence of patching of a concrete core was not evident based on our visual observations. Accurately knowing the size and spacing of the wall reinforcing steel is paramount to determining the structural adequacy of the tank construction with respect to seismic loading. If the core sample used as the basis of the reinforcing steel is not associated with the wall, to confirm the as-built conditions, we recommend that additional cores be taken through both horizontal and vertical steel by first mapping the rebar locations inside the wall using a rebar location detector.

3.1.2 <u>Proposed new concrete or masonry water containment wall</u>

A 3 ft tall concrete or masonry wall was proposed by SWE for delaying the water flow onto the street in case of reservoir rupture due to a major earthquake. The following are the additional factors, which need to be considered for proposal:

- The available space around the existing concrete tank is limited and it may not be feasible to build a concrete wall around the tank. Existing controls that are located very near to the wall of the tank would need to be relocated.

- In an event that the existing tank was to rupture, the proposed wall may not be effective to restrain water resulting in an uncontrolled release that can damage existing improvements at the site (footing scour and erosion) and an unknown impact to the adjacent private residences.

Based on this, we conclude that this proposed new wall may not be effective in mitigating potential release and damage.

3.2 Additional Seismic Calculations

We performed seismic analyses and capacity checks of the existing structure to identify potential structural deficiencies. We made certain assumptions for the structural parameters to perform this evaluation, as information about existing construction was limited. As discussed in Section 3.1 we determined that the thickness of the concrete wall is approximately 8 in based on our field measurements. The information regarding reinforcing in the wall is not known to be reliable at this time, therefore two scenarios for the rebar layout in the wall were considered to obtain the range of wall capacities possible. The two scenarios we evaluated are

- Case 1 – Assumed 5/8 in square rebar at 4 in on center in both horizontal and vertical direction. Assumed that the vertical rebar is in the middle of the wall.
- Case 2 – Assumed 5/8 in square rebar at 12 in on center in both horizontal and vertical direction. Assumed that the vertical rebar is in the middle of the wall.

The analysis was performed using ASCE 7-10, ACI 350-06 and ACI 350.3-06. Table 1 provides a summary of the material properties assumed based on the age of construction using the data provided in ASCE 41-13.

Table 1 Material Properties – Concrete and Reinforcing Steel
Peters Canyon and Vista Panorama Reservoir Condition Assessment
East Orange County Water District

Material	Property	Code/Source Reference
Concrete	$f'_c = 2{,}000\ psi$ $f'_{ce} = 3{,}000\ psi$	ASCE 41-13 Tables 10-1 and 10-2
Reinforcing Steel	$f_{ymin} = 33\ ksi$ $f_{ye} = 41\ ksi$	ASCE 41-13 Tables 10-1 and 10-3

Based on the assumed rebar size and spacing the following three cases are possible. These three cases describe the structural elements demand capacity ratios and corresponding operating storage capacity for the tank. Use of ultrasonic pulse echo or taking field concrete cores of the tank are recommended to verify the rebar size and spacing for comparison against the assumptions and conclusions described in this report. Once the as-built information is verified, the structural deficiencies from a seismic load resistance perspective can be finalized and appropriate mitigation techniques can be explored. The three cases identified below provide a range of behavior of the existing concrete tank in a seismic event:

3.2.1 Case 1

In this case, we assumed that the rebar is 5/8-in square at a spacing of 4 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at the high operating level of 11.8 ft above the floor of the tank. The storage volume at this height will be 150,000 gallons. The structural walls are capable of resisting the seismic loads imposed on them for this condition. The seismic sloshing wave height is approximately 3.4 ft. The available freeboard for this case is 0.2 ft and is not sufficient to accommodate the 3.4 ft sloshing wave height. The wood-framed roof will be damaged and will likely collapse due to the surcharge loads imposed by a sloshing wave. The spillage caused by this sloshing wave may also cause secondary damage by eroding the soil underneath existing structures near the tank and compromise their load carrying capacity. A mitigation strategy would be to lower the operating height of the water to 8.6 ft above the base of the tank. The volume at this reduced operating water level would be 109,000 gallons. Alternatively, replacement of the roof framing system with one that is capable of resisting the surcharge load or increasing the wall height to provide the necessary freeboard would allow utilization of the full storage capacity.

3.2.2 Case 2

In this case, we assumed that the rebar is 5/8-in square and spaced at 12 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at the high operating level of 11.8 ft above the floor of the tank. The storage volume at this height will be 150,000 gallons. The horizontal hoop rebar for this case is overstressed by 62 percent from height 2.5 ft to 9.5 ft (6.9 ft of wall height) measured from the top of the tank. The seismic sloshing wave height is approximately 3.4 ft. The available freeboard for this case is 0.2 ft and is not sufficient to accommodate the 3.4 ft sloshing wave height. The wood-framed roof will be damaged and will likely collapse due to the surcharge loads imposed by a sloshing wave. The spillage caused by this sloshing wave may also cause secondary damage by eroding the soil underneath existing structures near the tank and compromise their load carrying capacity. A mitigation strategy would be to lower the operating height of the water to 8.6 ft above the base of the tank. The volume at this operating water level would be 109,000 gallons. Alternatively, replacement of the roof framing system with one that is capable of resisting the surcharge load or increasing the wall height to provide the necessary freeboard would allow utilization of the full storage capacity.

3.2.3 Case 3

In this case, we assumed that the rebar is 5/8-in square and spaced at 12 in on center in both the horizontal and vertical directions. We assumed that the operating water level would be at a high operating level of 8.6 ft above the floor of the tank. The storage volume at this height would be 109,000 gallons. For this case, the existing tank rebar assumed is sufficient to resist the seismic loads imposed and the seismic sloshing wave height is less than the freeboard provided.

3.3 Onsite External Assessment

The exterior assessment of the reservoir was limited to a visual assessment of the reservoir from what could be seen from a walk around the perimeter of the reservoir and through the roof hatch. In general, the roof wood structure appears to be in good condition. The tank wall appears to be in good condition. The water level was being operated at around 3 ft so no visible leakage was observed. We concur with the SWE documented issues with water leakage and rusting of rebar. The following additional observations were made:

- The wood members do not seem to have been preservative treated for long-term durability. Decay and rot is expected in the future as the wood is exposed to moisture inside the tank and may have to be replaced at some time in the future. We recommend making regular inspections of the condition of the roof framing and the hardware.

- Based on the review of the photos taken by SWE, the steel posts at the bottom of the tank do not seem to be positively anchored to the floor slab. We recommend that a positive connection be provided by using anchor bolts from the base plate to the concrete floor slab.

3.4 Develop and Analyze Alternatives

3.4.1 Alternative 1 – Maintain Current Operating Condition - System Vulnerability

Due to concerns that EOCWD has about the condition of the reservoir and its potential vulnerability to leakage, it is currently operated with only 5,000 gallons of water in it. In the current partial operation condition, the water height would be at 3 ft above the base. One alternative would be to continue operating in this mode, but this is not the best use of the storage volume.

3.4.2 Alternative 2 – Seismic Rehabilitation

A complete full compliance seismic rehabilitation should be based on the structural deficiencies of the existing structure. Performing a complete seismic analysis of the existing structure would involve performing a detailed analysis and checking the existing foundation system, tank walls, and wood-framed roof structure. In Section 3.2 the tank walls, which are the main structural elements, have been analyzed in detail to identify potential structural deficiencies. The available structural element information of the tank structure is very limited and is required to provide the most suitable seismic retrofit alternatives. Based on the observed deficiencies a feasible seismic retrofit alternative may involve strengthening portions of the tank walls. This may be accomplished by one of the following or similar approaches:

- Provide supplemental circumferential reinforcing steel in the areas where the hoop rebar is deficient along the exterior of the tank wall. This approach would also require epoxied dowels into the existing wall at regular intervals. The reinforcing steel would need to be encapsulated in shotcrete or a formed concrete wall.

- Provision of an exterior steel shell and/or steel beam sections is another approach that appears to have been proposed by Premier Tank. This approach appears to be capable of providing ample supplemental reinforcing steel in the form of steel plate and beam sections. However, bonding of this retrofit should require small dowels, which can be located between the shell and the wall. The steel should be left exposed and properly coated for protection from corrosion. Attempting to shotcrete cover or finish over with a cementitious material is not advised as the difference in the coefficient of thermal expansion between the steel and large sections of steel may cause brittle finishes to crack and spall.

- Provision of post-tensioned 7-wire strand encapsulated in a shotcrete finish may be feasible provided that the additional post-tensioning does not induce excessive bending moments in the vertical reinforcing steel of the tank. This system imposes an active compressive load on the tank shell that can be designed to limit the development of tensile forces, thereby reducing the demand on the existing reinforcing steel. The feasibility of this retrofit approach would need to be verified by checking the required pre-compression force required and the additional bending moments and shear forces that would be induced by the post-tensioning.

The SWE report is limited to providing recommendations to prevent leakage issues with the tank and does not provide any recommendations for seismic retrofit of the existing tank. The Premier tank quotation does not seem to be based on an engineered solution to rectify existing structural deficiencies. In our opinion, the Premier tank proposed steel plates, angle and wide flange hoop at the top seems to be an excessive strengthening which may or may not rectify the structural deficiencies in the existing structure. We recommend that as-built structural information be identified through non-destructive testing and a limited number of core samples. Using the as-built information, a more detailed structural analysis using the Section 3.2 analysis as the basis to provide an appropriate full compliance seismic retrofit option.

3.4.3 Alternative 3 – Construct a new replacement reservoir

The most appropriate replacement reservoir for the Vista Panorama would be a bolted or welded steel tank. Steel tanks typically have a lower capital cost for at-grade construction compared to other alternatives and can be erected relatively fast. However, these tanks require a protective coating on the interior and exterior to protect it from corrosion. The tank will require a recoating every 20 to 30 years. Estimates for recoating a tank can be highly variable depending on the type of coating, condition of the tank steel, and air quality regulations. Cathodic protection can also provide additional protection.

The seismic performance of properly designed steel tanks can be excellent, provided those inherent vulnerabilities are carefully addressed. Such vulnerabilities include the tendency for the shell to buckle, excessive pipe restraint, tank uplift, and sloshing of water surcharge to the tank roof. These vulnerabilities were manifested in the 1994 Northridge Earthquake and subsequent large earthquakes throughout the world since that time. Numerous welded steel tanks failed with collapse, severe damage, foundation scouring, and loss of the tank contents. Steel tanks designed in accordance with current AWWA D100 standards are anticipated to have a significantly improved seismic performance compared to its predecessors. Fittings at pipe inlets and outlets should include flexible connections that allow for differential movement between the tank and the surrounding grade.

Leakage from steel tanks are expected to be minimal provided the tank is maintained in excellent condition.

When considering steel tanks, to understand the true cost of ownership, a life cycle cost analysis is recommended. A life cycle cost over a period of 40 years has been estimated. In development of our cost estimates we made the following assumptions:

- The existing soils require no additional improvement or replacement.
- Deep foundations, such as piles, are not required.
- Vista Panorama Reservoir may be taken offline, demolished as required, and replaced without the need to maintain temporary water storage to offset the lost volume during construction.
- The bottom of a new reservoir will not be any deeper than the existing reservoir.
- Dewatering is not considered.
- Soft costs for permitting, engineering, etc. are not included.

The total estimated replacement cost, including life cycle costs, for a welded steel tank is on the order of $500,000. The total estimated replacement cost including life cycle cost for a bolted steel tank is on the order of $400,000. We estimate that the total construction duration will be three months.

4.0 CONCLUSION

Our scope of services included a variety of tasks that included assessment of existing structural conditions, review of evaluations prepared by other consultants, evaluation of potential seismic vulnerabilities not addressed to date, re-evaluation of slope stability and seismic design parameters incorporating data from the Peralta Hills Fault, and evaluations of different replacement and retrofit alternatives for both the Peters Canyon Reservoir and the Vista Panorama Reservoir. The goal in carrying out these tasks is to assist EOCWD by providing useful information and recommendations that will help advance efforts to secure a safe and reliable water supply system.

4.1 Peters Canyon Reservoir

In general, our findings concur with previous findings for the Peters Canyon Reservoir. However, we have identified some additional issues that EOCWD should be aware of for the Peters Canyon Reservoir, which include the following:

- The existing wood-framed roof is constructed with CCA-treated wood. CCA is pesticide that is used as a wood preservative. Its use has come under the regulation of the EPA, which does not support its use where direct or indirect contact with potable water is possible. Decisions to retrofit the existing wood-framed roof structure should be carefully considered against the current regulations and any potential or otherwise perceived impact on water quality.

- Corrosion of the existing roof fasteners is expected to continue unabated. Deterioration of these connections is anticipated to worsen over time.

- The seismic load paths evaluated by others followed the more traditional concept, assuming the need for a diaphragm. Our evaluation of the existing load paths looked at load transfer through weak-axis bending and axial load transfer. We generally concur that provision of a diaphragm will resolve multiple vulnerabilities, both seismic and wind related.

- The nearby Peralta Hills Fault marginally increases the code-based seismic ground accelerations at the site. However, the slope stability evaluation results from the previous study by American Geotechnical remain valid.

To provide for a safe and reliable water supply, we believe that the Peters Canyon Reservoir should be seismically retrofitted or replaced to address a number of vulnerabilities and material issues identified. While a seismic retrofit is a potential path forward to help rectify the vulnerabilities, we believe the material concerns may be an over-riding consideration for EOCWD and its stakeholders. Accordingly, we also evaluated the feasibility of a reservoir replacement with a prestressed concrete tank or installation of a floating cover over the existing reservoir.

For planning purposes, we estimate that the cost to rehabilitate or replace the existing reservoir will be in the range of $5 to $8 million, depending on the alternative selected. Understanding that parallel work is underway to establish the land requirements at the site for a potential new water treatment facility, and modeling work is also underway to establish the long-term water storage needs of the District, a clear choice on the best alternative cannot yet be made. Once the land use needs at the site have been established and the storage volume needs confirmed, a further analysis to select the best rehabilitation/replacement approach can be made.

4.2 Vista Panorama Reservoir

Our findings for the Vista Panorama reservoir suggest that the reservoir requires additional non-destructive testing to identify as-built conditions. In the absence of detailed as-built information, we evaluated the tank based on visual observations and some assumptions regarding construction and materials. Based on these results, the tank should continue to be operated at a reduced height, although we conclude that the water level could be increased from the current depth of 3 ft to a maximum of 8.6 ft, which would increase the storage volume to around 100,000 gallons.

In addition, the reservoir does require some form of seismic retrofit or replacement. For planning purposes, we estimate that the cost of this work will range from $100,000 for a retrofit and up to $400,000 for replacement.

5.0 REFERENCES

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APPENDIX A – EXISTING DRAWINGS

AVAILABLE UPON REQUEST

APPENDIX B – PHOTOS

Photo B.1
Peters Canyon Reservoir Roof Looking South West

Photo B.2
Peters Canyon Reservoir Roof and North Wall

20-EOWWD1-15PhotoB1&2-9711A00.AI

APPENDIX B

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Photo B.3
Peters Canyon Reservoir Roof Central Vent

Photo B.4
Peters Canyon Reservoir Typical Pilaster

20-EOWWD1-15PhotoB3&4-9711A00.AI

APPENDIX B

Photo B.5
Peters Canyon Reservoir CMU wall Vent and Zip-Rib Roof

Photo B.6
Peters Canyon Reservoir - Typical Roofing and Flashing

Photo B.7
Peters Canyon Reservoir West Wall Looking South East

Photo B.8
Peters Canyon Reservoir Typical Purlin and brace at Perimeter Wall

Photo B.9
Peters Canyon Reservoir Typical Brace Bottom Connection at Perimeter wall

Photo B.10
Peters Canyon Reservoir Typical Framing at Central Vent

Photo B.11
Peters Canyon Reservoir Purlin to Glulam Connection

Photo B.12
Peters Canyon Reservoir Interior Concrete Columns and Braces

20-EOWWD1-15PhotoB11&12-9711A00.AI

APPENDIX B

Photo B.13
Peters Canyon Reservoir Hypalon Liner

Photo B.14
Peters Canyon Reservoir Zip-Rib Interior Clip Connection

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APPENDIX B

Photo B.15
Peters Canyon Reservoir Typical Hanger Connection at Purlin

Photo B.16
Peters Canyon Reservoir South Wall Looking North-East

Photo B.17
Peters Canyon Reservoir East Wall Looking North

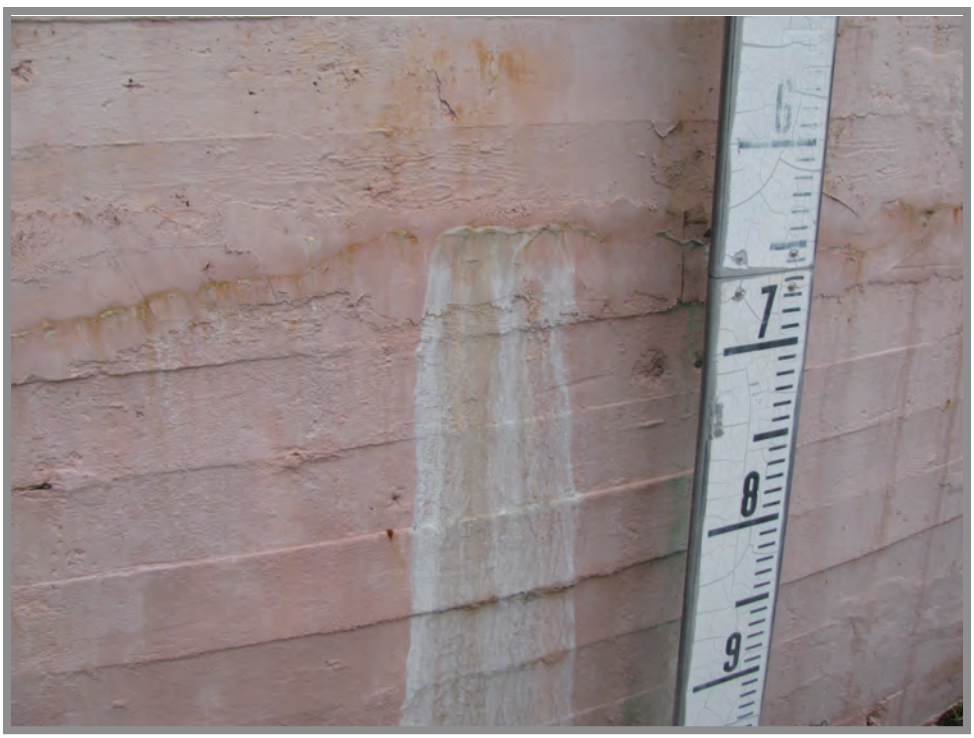

Photo B.18
Vista Panorama Reservoir Wall Elevation with Evidence of Water Leak

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APPENDIX B

APPENDIX C – CONVERSE CONSULTANTS REVIEW LETTER

Converse Consultants
Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

January 21, 2015

James A. Doering, P.E., S.E.
Principal Structural Engineer
Carollo Engineers, Inc.
3150 Bristol Street, Suite 500
Costa Mesa, California 92626

Subject: **GEOTECHNICAL REVIEW OF SLOPE STABILITY ANALYSIS AND PROVIDING UPDATED SEISMIC DESIGN PARAMETERS**
Existing 6-Million Gallon Reservoir
East Orange County Water District
Orange, California
Converse Project No. 15-31-112-01

Dear Mr. Doering:

In accordance with your request, Converse Consultants (Converse) has prepared this letter to provide our geotechnical review of slope stability analysis and provide updated seismic design parameters for the Geotechnical and Seismic Hazard Assessment Report by American Geotechnical, Inc.

Based on our review of the April 2, 2014 *Geotechnical and Seismic Hazard Assessment Report*, the geotechnical recommendations presented in the referenced report with regards to slope stability analyses are acceptable. This slope stability analyses also seem to be in agreement with a previous *Materials Report for Design Section 10 for State Routes 241 and 261*, dated July 18, 1997.

Our updated seismic design parameters, including fault modeling to account for the Peralta Hills Fault based on recent published information, are presented below. The location of the Peralta Hills Fault with reference to the subject site is shown in Drawing No. 1, *Project Site*.

CBC Seismic Design Parameters

Seismic parameters based on the 2013 California Building Code are calculated using the United States Geological Survey *U.S. Seismic Design Maps* website application and the site coordinates (33.7765 degrees North Latitude, 117.7552 degrees West Longitude). The seismic parameters are presented in the table below.

N

0 5000 10000

APPROXIMATE SCALE IN FEET

PROJECT SITE

Converse Consultants

EAST ORANGE COUNTY WATER DISTRICT
EXISTIN 6 MILLION GALLON RESERVOIR
ORANGE, CALIFORNIA
FOR: CAROLLO ENGINEERS, INC.

Project No.	Drawing No.
15-31-112-01	1

Table No. 1, CBC Seismic Design Parameters

Seismic Parameters	2013 CBC
Site Class	C
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.500 g
Mapped 1-second Spectral Response Acceleration, S_1	0.581 g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.3
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.500 g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.755 g
Design Spectral Response Acceleration for short period, S_{DS}	1.000 g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.504 g
Seismic Design Category	D

Site-Specific Response Spectra

The subject site is not located in a Seismic Hazard Zone, defined as a mapped California State Earthquake (previously Alquist-Priolo Earthquake Fault Zone), Liquefaction or Earthquake Induced Landslide zone per latest California Geologic Survey (CGS) state maps. Based on 2013 CBC Section 1616A.1.3, a site-specific ground motion analysis is not required A site-specific response spectrum was developed for the project for a Maximum Considered Earthquake (MCE), defined as a horizontal peak ground acceleration that has a 2 percent probability of being exceeded in 50 years (return period of approximately 2,475 years).

In accordance with ASCE 7-10, Section 21.2 the site-specific response spectra can be taken as the lesser of the probabilistic maximum rotated component of MCE ground motion and the 84th percentile of deterministic maximum rotated component of MCE ground motion response spectra. The design response spectra can be taken as 2/3 of site-specific MCE response spectra, but should not be lower than 80 percent of CBC general response spectra. The risk coefficient C_R has been incorporated at each spectral response period for which the acceleration was computed in accordance with ASCE 7-10, Section 21.2.1.1.

The 2013 CBC mapped acceleration parameters are provided in the following table. These parameters were determined using the United States Geological Survey *U.S. Seismic Design Maps* website application, and in accordance with ASCE 7-10 Sections 11.4, 11.6, 11.8 and 21.2.

Table No. 2, 2013 CBC Mapped Acceleration Parameters

Site Class	C	Seismic Design Category	D
S_S	1.500	C_{RS}	1.028
S_1	0.581	C_{R1}	1.056
F_a	1	$0.08\ F_v/F_a$	0.104
F_v	1.3	$0.4\ F_v/F_a$	0.520
S_{MS}	1.500	T_0	0.101
S_{M1}	0.755	T_S	0.504
S_{DS}	1.000	T_L	8
S_{D1}	0.504		

A Site-Specific response analysis, using faults within 100 kilometers of the sites, was developed using the computer program EZ-FRISK by Risk Engineering (v. 7.62) and the 2008 USGS Fault Model database. Attenuation relationships proposed by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008) were used in the analysis. These attenuation relationships are based on Next Generation Attenuation (NGA) project model. Maximum rotated components were determined using Huang (2008) method. An average shear wave velocity at upper 30 meters of soil profile (V_{s30}) of 550 meters per second, depth to bedrock of with a shear wave velocity 1,000 meters per second at 50 meters below grade, and depth of bedrock where the shear wave velocity is 2,500 meters per second at 3,000 meters below grade were selected for EZ-Frisk Analysis.

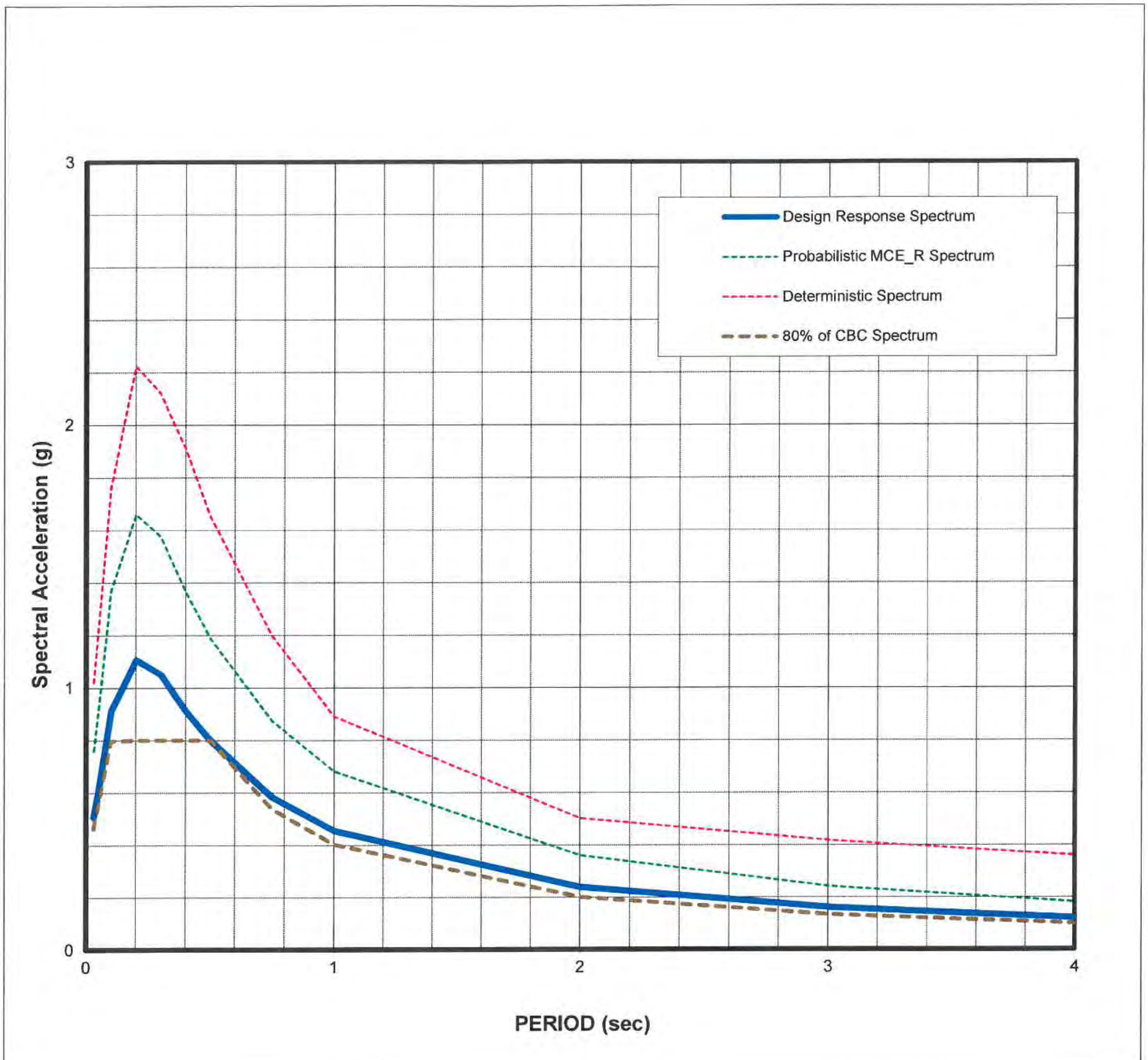
The Peralta Hills fault, which is not found in the available seismic sources database of the EZ-Frisk program, was modeled based on available reference materials from fault studies (Bryant and Fife) as well as USGS Quaternary Fault Google Earth (kmz) files. With the limited data available for the Peralta Hills Fault, horizontal and vertical geometry of the fault as well as deterministic magnitude, and dip orientation along with several other parameters were modeled into the EZ-Frisk seismic sources database to account for the Peralta Hills Fault as part of the site specific seismic design parameters analysis.

Applicable response spectra data are presented in the table below and on Drawing No. 2, *Site-Specific Design Response Spectrum.* These curves correspond to response values obtained from above attenuation relations for horizontal elastic single-degree-of-freedom systems with equivalent viscous damping of 5 percent of critical damping.

Table No. 3, Site-Specific Response Spectrum Data

Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g)	Risk Coefficient C_R	Probabilistic MCE_R Spectral Acceleration (g)	84th Percentile Deterministic MCE Response Spectra, (g)	Deterministic CBC Lower Level, (g)	Site Specific MCE_R Spectral Acceleration (g)	80% CBC Design Response Spectrum	Site Specific Design Spectral Acceleration (g)
0.03	0.739	1.028	0.760	1.021	0.860	0.760	0.463	**0.506**
0.05	0.896	1.028	0.921	1.223	1.033	0.921	0.558	**0.614**
0.10	1.332	1.028	1.369	1.764	1.465	1.369	0.797	**0.913**
0.20	1.615	1.028	1.660	2.224	1.500	1.660	0.800	**1.107**
0.30	1.528	1.032	1.576	2.120	1.500	1.576	0.800	**1.051**
0.40	1.320	1.035	1.366	1.909	1.500	1.366	0.800	**0.911**
0.50	1.144	1.039	1.188	1.654	1.500	1.188	0.800	**0.800**
0.75	0.834	1.047	0.873	1.198	1.040	0.873	0.537	**0.582**
1.00	0.646	1.056	0.682	0.891	0.780	0.682	0.403	**0.455**
2.00	0.341	1.056	0.360	0.501	0.390	0.360	0.201	**0.240**
3.00	0.229	1.056	0.242	0.417	0.260	0.242	0.134	**0.161**
4.00	0.173	1.056	0.182	0.360	0.195	0.182	0.101	**0.122**

Vertical acceleration at the site may be calculated using the ASCE 7-10, Section 12.4.



Note: Calculated using EZFRISK program Risk Engineering, version 7.62
and USGS 2008 fault model database.

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

East Orange County Water District Existing 6-Million Gallon Reservoir

Orange, CA

For : Corollo Engineers, Inc.

Project Number:

15-31-112-01

Converse Consultants

Drawing No.

2

The site-specific design response parameters are provided in the following table. These parameters were determined from Design Response Spectra presented in table above, and following guidelines of ASCE Section 21.4.

Table No. 4, Site-Specific Seismic Design Parameters

	Design Parameters (5% Damping)	**Lower Limit, 80% of CBC Design Spectra**
Site-Specific 0.2-second period Spectral Response Acceleration, S_{MS}	1.660	1.200
Site-Specific1-second period Spectral Response Acceleration, S_{M1}	0.720	0.604
Site-Specific Design Spectral Response Acceleration for short period S_{DS}	1.107	0.800
Site-Specific Design Spectral Response Acceleration for 1-second period, S_{D1}	0.480	0.403

We appreciate the opportunity to be of continued service to Carollo Engineers, Inc. If you have any questions or require additional information, please call the undersigned at (626) 930-1275.

CONVERSE CONSULTANTS

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Senior Engineering Geologist

Dist: 2/Addressee
MM/MBS/SKS/jjl

References

1) *Geotechnical and Seismic Hazard Assessment Report, Existing 6-Million Gallon Reservoir, East Orange County Water District, Handy Creek Road, Orange, California*, prepared by American Geotechnical, Inc., File No. 33615-01, dated April 2, 2014.

2) *Municipal Water District of Orange County Earthquake Vulnerability Study, Five Earthquake Scenario Ground Motion Maps for Northern Orange County, prepared by Earth Consultants International, ECI Project No. 2509, dated June 20, 2005.*

3) *The Peralta Hills Fault, a Transverse Ranges Structure in the Northern Peninsular Ranges, Geology and Mineral Wealth of the California Transverse Ranges, South Coast Geological Society, prepared by Mark E. Bryant and Donald L. Fife, Converse Consultants, Inc., dated 1982.*

4) *Geotechnical Report (Addendum), Proposed Tunnels for Diemer Intertie Project Phase IV, Design, Municipal Water District of Orange County, prepared by W. A. Wahler & Associates, Project BEC105A, dated September, 1978.*

5) *Seismic Refraction Study of the El Modeno Fault, Orange County, California, California Geology, prepared by J. A. Ryan, Jon N. Burke, A. F. Walden, and Daniel P. Wieder, dated January, 1982.*

6) *Revised Final Materials Report for Design Section 10, State Routes 241 and 261, Eastern Transportation Corridor, Orange County, CA, prepared by Silverado Constructors, Submittal No. E10GXX-01, dated July 18, 1997.*

7) *Revised Final Analyses and Calculations Materials Report for Design Section 10, State Routes 241 and 261, Eastern Transportation Corridor, Orange County, CA, prepared by Silverado Constructors, Submittal No. E10GXX-01, dated July 18, 1997.*

8) *Risk Engineering Inc., EZ_FRISK, Version 7.62, 2011, A Computer Program for Site Specific Seismic Hazard Analyses.*

9) *CALIFORNIA BUILDING CODE (CBC), 2013, International Conference of Building Officials.*

APPENDIX A

PERALTA HILLS FAULT REFERENCE INFORMATION

American Geotechnical, Inc.
GEOTECHNICAL ENGINEERING / MATERIALS TESTING & INSPECTION

April 2, 2014 File No. 33615-01

Brady Engineering
3710 Ruffin Road
San Diego, CA 92123

Attention: Mr. Sean Sudol, Smarter Water Program Manager

Subject: **GEOTECHNICAL AND SEISMIC HAZARD ASSESSMENT REPORT**
Existing 6-Million Gallon Reservoir
East Orange County Water District
Handy Creek Road
Orange, California

References: See Appendix A

Dear Mr. Sudol:

In accordance with your request, American Geotechnical has completed a geotechnical investigation and seismic hazard assessment for the existing 6-Million gallon reservoir site located on Handy Road in Orange, California, as indicated on Plate 1. The purpose of this investigation was to evaluate the site geotechnical conditions related to slope stability conditions, to identify potential geologic hazards associated with the project site, and make recommendations for site improvements related to hillside stability and surface drainage structures. Our findings, conclusions, and preliminary recommendations for earthwork and foundations are presented below.

1.0 SCOPE OF WORK

The scope of the work performed during this investigation included the following:

- Review of engineering plans for construction of the reservoir;
- Review engineering and hydrogeology reports for the project site prepared by others;
- Performing a preliminary geologic reconnaissance of the site and vicinity;
- Drilling and logging, using visual and tactile methods, four (4) soil borings;

- Collection of undisturbed and bulk samples of representative materials encountered in the soil borings;
- Performing laboratory testing of the selected soil samples;
- Performing engineering analyses of the field and laboratory data;
- Performing a geologic hazard assessment of the project site;
- Evaluation of groundwater conditions beneath the project site;
- Performing slope stability assessments of the existing slopes surrounding the reservoir; and,
- Preparation of this report summarizing our field investigation, laboratory testing, findings, conclusions, and recommendations.

2.0 SITE DESCRIPTION

We understand that the project site (6 MG reservoir) was constructed along a northwest-southeast trending ridgeline through a combination of excavation (cut) and placement of fill soils (fill) and is approximately 290 feet long, 165 feet wide, and approximately 16 to 17 feet deep. The reservoir is a side sloped concrete structure lined with a flexible hypalon membrane (based on visual appearance approximately 40-45 mil thick) with a perimeter masonry block wall supporting the existing roof. There is a perimeter asphalt driveway surrounding the reservoir.

It is understood that the water district is evaluating replacement of the existing roof deck and has requested evaluation of the stability of the hillsides surrounding the reservoir prior to proceeding with site improvements.

3.0 SITE GEOLOGY

The project site is underlain by sedimentary deposits mapped by J.E. Shoellhamer and D.M. Kinney, R.F. Yerkes, and J.G. Vedder (1954) as Eocene to Miocene age " Undifferentiated Vasqueros Formation and Sespe Formation" and Late Miocene age "La Vida Member of the Puente Formation". The geologic map prepared by Shoellhamer et. al. identify that the existing reservoir straddles the contact between the Vasqueros/Sespe and La Vida/Puente sediments and they further define the contact as a "fault contact". This map also identifies that the La Vida/Puente sediments are inclined (dip) to the north at angles ranging from 21° to 29° (angle from horizontal).

Subsequent geologic mapping by P.K. Morton and R.V. Miller (1981) also identifies that the site is underlain by the “Undifferentiated Vasqueros/Sespe Formation” and the "La Vida Member of the Puente Formation" and similarly identifies the contact between the Vasqueros/Sespe and La Vida/Puente sediments as a "fault contact". This map also identifies that the La Vida/Puente sediments dip to the north at an angle of 25° beneath the project site.

A subsequent geologic map prepared by D.M. Morton (2004) continues to depict that the site is underlain by both the “Vasqueros/Sespe Formation” and the "La Vida Member of the Puente Formation"; however, the contact between the two units is no longer identified as a "fault contact". Morton identifies that the La Vida/Puente sediments dip to the north at an angle of 25° beneath the project site.

Geologic mapping of the project site during the site reconnaissance and during the geotechnical investigation identified sediments on the easterly hillside cut slope and native sediments observed in the soil borings as being consistent with the La Vida sediments, while the sediments west of the reservoir property perimeter fence and the Cox Communication facilities were consistent with the Vasqueros/Sespe sediments.

Prior grading for construction of the reservoir likely encountered both the Vasqueros/Sespe and La Vida formations, and construction of the fill sediments around the reservoir currently masks the geologic contact. In addition, the geologic contact between these units could not be observed or distinguished beyond the reservoir property boundary due to the prior grading and existing soil horizon developed at the ground surface. Based on the consistency of the native sediments encountered in the geotechnical soil borings, as well as the previous soil borings advanced by GeoPentech, it is our opinion that the contact between the Vasqueros/Sespe and La Vida/Puente formations exists along the eastern margin of the site beneath the reservoir structure.

4.0 SUBSURFACE INVESTIGATION

Our subsurface investigation included advancing four (4) soil borings (AGSB-1 thru AGSB-4) at the project site on February 10, 2014, at the approximate locations shown in Plate 2.

The materials encountered in the soil borings consisted of approximately 10-feet (Boring AGSB-1) to 26-feet (AGSB-3) of gray silty sand (fill soil) overlying native colluvium consisting of orange to yellow and gray silty sand with some gravel. Native sediments consisting of dark gray silty sand (siltstone) of the La Vida/Puente Formation were encountered beneath the fill soils and beneath the colluvium to the depth of 36.5 feet (limits of exploration) across the project site. Groundwater seepage was not encountered in the soil borings, and regional high groundwater levels are reported to exceed 50-feet below the ground surface.

The soil borings were logged by our field personnel using both visual and tactile methods. Detailed boring logs are presented in Appendix B. Representative samples of the subsurface materials were collected from all borings and forwarded to the laboratory for the purpose of estimating material properties for use in subsequent engineering evaluations.

GeoPentech advanced soil borings along the southern area the project site in 2010 at approximate locations depicted on Plate 3 to evaluate groundwater seepage conditions associated with a shallow hillside slope failure. Their borings extended to depths of 51-feet below the ground surface and encountered similar fill soil, colluvium, and light- to dark gray siltstone sediments identified as La Vida/Puente Formation. Groundwater was not observed in the soil borings by GeoPetech personnel at the time of drilling, and monitoring wells were constructed in each of the borings for long-term monitoring. Information provided from East Orange County Water District personnel indicated that groundwater has not been observed in these monitoring wells to date.

Five different geologic cross-sections (Sections A-A' thru E-E') were developed to depict the underlying fill soil and bedrock conditions at the locations depicted on Plate 3. These cross-sections (see Plates 4 thru 6) were used to assist in selection of critical sections for the slope stability analyses (presented later in this report).

The soil borings were backfilled with soil cuttings derived from the drilling activities to match the existing surface conditions.

5.0 LABORATORY TESTING

Laboratory testing was performed on samples collected during our field exploration. Samples were tested for the purpose of estimating material properties for use in the subsequent engineering evaluation. Tests included in-situ moisture and density, maximum density and optimum moisture content, gradation, direct shear, and chemical testing. A summary of our laboratory test results is presented in Appendix C.

6.0 SEISMIC HAZARDS

The project site is situated within an area of southern California which is traversed by numerous active faults capable of generating moderate to large magnitude earthquakes. The closest active fault to the project site is the Whittier Fault located approximately 7.8 miles to the north. A summary of the local faults, the distances to the project site, estimated maximum magnitude earthquake, and estimated ground movement at the site resulting from an earthquake along each of these faults is included in Appendix D. A summary of the nearby active faults is summarized in Table 1 below:

TABLE 1

SUMMARY OF FAULT CHARACTERIZATIONS AND DISTANCE FROM SITE

FAULT	APPROXIMATE DISTANCE FROM SITE	TYPE OF FAULT	MAXIMUM EARTHQUAKE MAGNITUDE (Mw)
Whittier	7.8 miles	Strike Slip	6.8
Elsinore-Glen Ivy	8.8 miles	Strike Slip	6.8
Chino-Central Ave. (Elsinore)	9.8 miles	Strike Slip	6.7
Elysian Park Thrust	12.2 miles	Thrust	6.7
Compton Thrust	14.1 miles	Thrust	6.8
Newport Inglewood (L.A. Basin Segment)	15.3 miles	Strike Slip	6.9
Newport Inglewood (Offshore Segment)	15.8 miles	Strike Slip	6.9

Note: Data derived from EQFault Analysis

Seismic hazards for any site in southern California include primary hazards (ground rupture and ground shaking) and secondary hazards (liquefaction, ground settlement, lurch cracking, lateral spreading, and landslides). The primary and secondary hazards for the project site are addressed below:

Fault Rupture

Surface rupture occurs when movement along a fault breaks through to the ground surface. The ground rupture almost always follows pre-existing faults, which are zones of weakness, and may occur suddenly during an earthquake or slowly in the form of fault creep. Sudden displacements are more damaging to structures because they are accompanied by ground shaking.

The project area is not located within an Alquist-Priolo Earthquake Fault Zone, and no active faults are mapped to pass through the project site. However, previous geologic mapping has identified the contact between the Vasqueros/Sespe and La Vida/Puente formations to be a "fault contact" and this contact represents a zone of weaker soil conditions within the footprint of the reservoir. As such, there is low to moderate risk of localized ground displacement (on the order of millimeters to centimeters) adjacent to or beneath the reservoir structure as a result of a local large magnitude earthquake.

Ground Shaking

The intensity of the seismic shaking or strong ground motion at the project site during an earthquake depends on the distance between the project area and the epicenter of the earthquake, the magnitude of the earthquake, and the geologic conditions underlying and surrounding the site. Earthquakes occurring on faults closest to the project site would most likely generate the largest ground motions within the project area.

The EQFault analysis (Appendix D) estimates that the degree of ground shaking reported as peak ground acceleration (%g) at the project site would be 0.316g. This data suggests that the project site is at moderate risk of experiencing strong ground shaking for future earthquakes along the local active faults. The EQFault analysis also indicates that the site could experience Intensity IX ground shaking effects as described by the Modified Mercalli Intensity Scale.

A regional probabilistic seismic hazard analysis was performed for the project site utilizing the United States Geologic Survey Probabilistic Seismic Hazard Analysis procedures. This analysis utilizes historic earthquake records combined with specified earthquake return periods for various magnitude earthquakes on individual faults. The combined analysis results in an estimation of the probability of any magnitude earthquake occurring during a given exposure period. Table 2 presents the findings of the probabilistic analysis:

TABLE 2

EARTHQUAKE PROBABILITY ANALYSIS RESULTS

SPECIFIED EARTHQUAKE MAGNITUDE (Magnitude X)	PROBABILITY OF AN EARTHQUAKE EQUAL TO OR GREATER THAN THE SPECIFIED MAGNITUDE (X) OCCURRING WITHIN 50 YEARS AND WITHIN 50 KILOMETERS OF PROJECT SITE
M=5	100%
M=6	60% to 80%
M=6.5	40% to 80%
M=7.0	15% to 40%
M=7.5	3% to 12%
M=8.0	0% to 1%

Further evaluation identified that the project site has a cumulative risk of ground motion originating from an earthquake occurring on any given fault segment for any period of exposure. The analysis indicated that the site has a 10 percent (10%) probability of experiencing or exceeding a ground motion of 0.3g to 0.4g during a 50-year exposure period. This 10% in 50-year risk analysis is consistent with the deterministic results derived from the EQFault analysis.

The probabilistic analysis further indicated that the site has a 2 percent (2%) probability of experiencing or exceeding a ground motion of 0.5g to 0.6g for the same 50-year exposure period.

Liquefaction and Differential Compaction

Liquefaction is a phenomenon in which saturated granular sediments temporarily lose their shear strength during periods of earthquake-induced strong ground shaking. The susceptibility of a site to liquefaction is a function of the depth, density, and water content of the sediments and the magnitude of an earthquake. Saturated, unconsolidated silts, sands, silty sands, and gravels within 50 feet of the ground surface are most susceptible to liquefaction. Typical effects of liquefaction include loss of bearing strength, lateral spreading, and settlement. Differential compaction occurs when unsaturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

The sediments beneath the project site consist of compacted fill soils and dense silty sand deposits, and the water table exceeds a depth of 50-feet below the ground surface; therefore, the project site has a low susceptibility for liquefaction and differential compaction.

Earthquake-Induced Lurch Cracking, Lateral Spreading, and Landslides

The project site is situated along an existing hillside cut slope; therefore, the project site has a low to moderate susceptibility for lurch cracking, lateral spreading, or landslide activity during a strong earthquake event.

The existing hillside fill slopes have been buried with a thick mantle of vegetation and tree trimmings/debris resulting in a very soft and loose slope surface condition. In addition, there are numerous gopher holes present throughout the hillside areas and there are no perimeter curbs or surface drains on the existing paved perimeter road to intercept and reduce surface water runoff from flowing directly over the hillside slopes.

A preliminary slope stability assessment was performed for the existing hillside fill slopes surrounding the reservoir, and the findings are discussed in the following section of this report.

Tsunamis and Seiches

The project site is a reservoir site, therefore, the surrounding hillside slopes have a high susceptibility for inundation from seiches (water leaving the reservoir through vents or by partial wall/roof failure); however, given the great distance from the ocean and elevation of the ground surface (approximately 790 feet msl) the susceptibility due to tsunamis is very low.

7.0 SLOPE STABILITY ANALYSIS

A preliminary slope stability assessment was performed for the existing hillside slopes surrounding the reservoir and two (2) cross-sections (Sections D-D' and E-E') were selected as "critical slopes" for further assessment. Cross-section E-E' was separated into two distinct areas identified as: (1) "upper slope area" (hillside slope from the reservoir to just above the entrance service road) and (2) "lower slope area" (hillside slope from the entrance service road to the northerly property boundary) as depicted on Plate 7.

The slope stability analysis was performed on Sections D-D' and E-E' for the following conditions:

1) No groundwater present within 50 feet of the ground surface (dry soil-no leak conditions).
2) No groundwater present within 50 feet of the ground surface (dry soil-no leak conditions) under seismic loading.
3) Groundwater present at or below the fill soil-native soil contact (long-term minor leak condition).
4) Groundwater present at or below the fill soil-native soil contact (long-term minor leak condition) under seismic loading.
5) Groundwater present within the fill soils (moderate, long-term leak condition).
6) Groundwater present within the fill soils (moderate, long-term leak condition) under seismic loading.
7) Groundwater present within the fill soils rising to near the ground surface (catastrophic severe leak condition).
8) Groundwater present within the fill soils rising to near the ground surface (catastrophic severe leak condition) under seismic loading.

The stability of the site based on the available information was analyzed using the GSTABL computer program. The following shear strength parameters were utilized in our analyses:

Material	Cohesion (psf)	Angle of Internal Friction (Deg.)
Fill Soil	0	37.5
Colluvium	0	36.0
Bedrock	400	30.0

The soil shear strength parameters indicated above are based on direct shear test results presented in Appendix C.

The input and output data of the slope stability analyses are presented in Appendix E. The results of the stability analyses are also summarized in Table E1, Appendix E. As shown in Table E1, the preliminary assessment identifies that Sections D-D', E-E' (upper), and E-E' (lower) have a factor-of-safety exceeding 1.5 under static dry fill soil (no leak conditions) and minor-leak conditions (where groundwater is confined to at or below the fill/native soil contact region).

Section D-D' and Section E-E' (upper) have reduced factor-of-safety of 1.448 and 1.413, respectively, under moderate leak, partially saturated conditions (groundwater rising half-way in the fill soils). However, Section E-E' (lower) has a significantly reduced factor-of-safety of 0.667 under these same partially saturated conditions.

Further reductions in the factor-of-safety under static, fully saturated conditions (groundwater rising in the fill soils to near ground surface) occur in all cross-sections with Section D-D' reducing to 0.822, Section E-E' (upper) reducing to 0.941, and Section E-E' (lower) reducing to 0.667.

These analyses identified that the lower portion of hillside slope for Section E-E (located from the entrance road northward to the toe of the fill slope) has a factor-of-safety below 1.5 under static conditions with moderate to severe leak conditions.

Further slope stability evaluation under seismic loaded (pseudostatic) conditions identified that the factor-of-safety for Sections D-D', E-E' (upper) and E-E' (lower) remain above to 1.1 under dry fill soil conditions (no leak) or minor-leak conditions (where groundwater is confined to the fill/native soil contact region).

As water is introduce into the fill soils the factor-of-safety under seismic loaded, partially saturated conditions reduces to below 1.0 for all cross-sections evaluated. For moderate and severe leak conditions (groundwater rising half-way in the fill soils or all the way to the ground surface), the factor-of-safety for Section D-D' reduces to 0.978 and 0.560, respectively.

Similarly, the factor-of-safety for Section E-E' (upper) reduces to 1.010 and 0.708 for moderate and severe leak conditions, respectively. The factor-of-safety for Section E-E' (lower) reduces to 0.429 for both moderate and severe leak conditions.

The pseudostatic analysis identified that the hillside slopes for Sections D-D' and E-E' have a factor-of-safety below 1.0 under partially saturated conditions and appear unstable under moderate to severe groundwater conditions.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 Slope Stability

Geotechnical exploration, analyses, experience, and judgment result in the conclusion that the existing reservoir site is stable from a geotechnical and geologic perspective under the current site conditions. Our investigation indicated that the site is underlain by fill over native colluvium and bedrock materials. No groundwater was encountered up to the depths of our exploration. Assuming that the groundwater remains at or below the contact between the fill and native, our slope stability analyses indicated more than the required factor of safeties of 1.5 and 1.1 under static and seismic conditions, respectively. In other words, the site slopes are stable under the current conditions or even under a minor leak in the water tank that results in the groundwater rising up to the contact between the fill and native soils.

Further analyses indicate that the hillside slopes would have reduced factor-of-safeties if the ground water rises above the contact between fill and native soils. A significant long-term leak in the tank can result in saturation of soil around the tank and eventual perched groundwater conditions above the bedrock contact. Under the scenario of this groundwater condition, the site slopes would have reduced factor of safeties with the potential instability.

Various treatment options are available to improve the stability of site slopes. These options include re-grading of the slopes with soil reinforcements such as geogrids. If desired, further site investigation and analysis can be performed to provide detailed recommendations for such options. However, in our opinion, monitoring site ground water conditions and performing preventive measures are more practical for the site.

We recommend that the groundwater conditions be monitored around the existing water tank, as a minimum. Noting that the native sediments underlying the reservoir dip to the north, and that the two (2) existing monitoring wells are situated "up dip" from the reservoir and may not detect water releases from beneath the reservoir, it is recommended a minimum of two (2) additional monitoring wells be installed at the site. The first monitoring well should be installed along the top of the easterly fill slope between American Geotechnical soil borings AGSB-2 and AGSB-3 to monitor potential water level rise in the underlying fill soils. The second monitoring well should be installed along the northerly fill slope just upslope from American Geotechnical soil boring AGSB-4. This monitoring well would be situated "down-dip" of the reservoir and could be an early indicator of water release from the reservoir seeping into the underlying native sediments (as well as the fill soils). It is suggested that the monitoring be performed at least every three months. If shallow ground water conditions are detected, the tank should be checked for any leaks along with other preventing measures.

We also recommend that the site drainage around the tank be improved to prevent water infiltration into the fill mass and improve the site stability. The absence of an existing curb and surface water control along the northern or southern perimeter road areas which surround the reservoir allows rain runoff to flow directly onto the hillsides and into the underlying soils. Introduction of water into the fill soils results in a reduction of the factor of safety for stability of the hillside fill slopes and should be avoided by runoff control and drainage improvements. It is recommended that a perimeter curb be constructed along the entire top of fill slope to intercept all rain runoff to a catch basin and into a discharge pipe which discharges the water away from the hillside slope.

8.2 Seismic Considerations

A moderate to large earthquake (magnitude 5.5 or greater) could result in intense to severe ground shaking at the site, and there is a potential for sympathetic movement (horizontal and/or vertical offset) on the order of millimeters to centimeters to occur along the geologic contact/fault contact located beneath the reservoir as a result of this ground motion. This differential movement along the geologic contact (potential lateral or vertical propagation of reactivated or new fissures) could be partially mitigated if there was a layer of fill soils beneath the reservoir concrete slab. However, details of the earthwork beneath the reservoir (such as depth of over-excavation and compaction) are not known, and as such are considered to be less than 1-foot thick (if any) and would provide minimal mitigation effects.

The closest known active fault to the site is the offshore segment of the Whittier Fault, located approximately 8.4 miles from the site. The following seismic parameters, based the 2013 Edition of the California Building Code (CBC), Chapter 16, Section 1613, are provided below for consideration in the design and analysis.

1. Site Class : C

2. Site Coefficients
   - F_a : 1.0
   - F_v : 1.3

3. Mapped spectral accelerations
   - S_s (for short periods) : 1.500
   - S_1 (for 1-second period) : 0.581

4. Site adjusted spectral accelerations
   - S_{MS} (for short periods) : 1.500
   - S_{M1} (for 1-second period) : 0.755

5. Design spectral accelerations
   - S_{DS} (for short periods) : 1.000
   - S_{D1}(for 1-second period) : 0.504

It should be realized that the purpose of the seismic design/analysis utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of a large earthquake. This is the case for essentially all structures in southern California.

8.3 Asphalt Pavement

The existing asphalt paving ranges from 1-inch to 2-inches of asphalt placed on less than 4-inches of aggregate base (or placed directly on the fill soils) and is highly deteriorated and cracked which allows water to percolate directly into the underlying fill soils. It is recommended that the perimeter access road be removed and replaced with a proper pavement section with a minimum of 3-inches of asphalt over a minimum of 6-inches of compacted aggregate base (based on the R-Value of 30 and Traffic Index of 5).

After removing the existing asphalt within the new pavement area, the upper 1 foot of subgrade soil should be reworked, moisture conditioned, and compacted to a minimum of 90 percent of relative compaction. The actual depth of recompaction can be determined by the project soil engineer at the time of construction and based on the site condition. Prior to start of the subgrade compaction, all utility lines should be located and marked in the field so that they can be protected and/or relocated, if necessary. All debris and perishable material should be removed from the site.

After completion of recompaction of subgrade materials, an aggregate should be placed over the reworked/compacted fill. The aggregate base is assumed to have a minimum R-value of 78 and should conform to Caltrans Standard Specifications Section 26 or Green Book specifications (Section 200-2.2). The base should be placed with a minimum compaction of 95 percent. In all new pavement areas, adequate surface drainage should be provided.

8.4 Concrete

Laboratory testing indicated that the surface soil at the site has low levels of sulfates, and as such no special sulfate resistant concrete mix design is required for any proposed concrete construction. However, we recommend that low-permeable concrete be utilized at the site for better performance. For this purpose, the water-to-cement ratio in the concrete should be limited to 0.5 (0.45 preferred). Use of utilizing Type V cement is also preferred. Limited use of a water-reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. The code dictates at least seven days of moist curing. Two to three weeks is preferred to minimize cracking. Special surface-applied curing compounds can be used subject to acceptance by the design engineer. One-inch hard rock mixes are recommended. Pea-gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements, provided the mix designs consider limiting shrinkage. Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Alternatively, the concrete mix can also incorporate W.R. Grace/Eclipse shrinkage reducing admixture dosed for maximum benefit. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

8.5 Corrosion Potential

In addition to sulfate tests, Chloride, pH, and resistivity tests on near-surface site soil were performed. Results of these tests are presented in Appendix C. Appropriate design considerations should be made to reduce the risk of damage from corrosion.

9.0 REMARKS

Only a portion of subsurface conditions have been reviewed and evaluated. Conclusions, recommendations, and other information contained in this report are based upon the assumption that subsurface conditions do not vary appreciably between and adjacent to the observation points. Although no significant variation is anticipated, it must be recognized that variations can occur.

This report has been prepared for the sole use and benefit of our client. The intent of this report is to advise our client on geotechnical matters involving the proposed improvements. It should be understood that the geotechnical consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report and/or any other geotechnical aspect of the project should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive the advice. Subsequent use of this report can only be authorized by the client. Any transferring of information or other directed use by the client should be considered "advice by the client."

Geotechnical engineering is characterized by uncertainty. Geotechnical engineering is often described as an inexact science or art. Conclusions and recommendations presented herein are based upon the evaluation of technical information gathered, experience, and professional judgment. The conclusions and recommendations presented should be considered "advice." Other consultants could arrive at different conclusions and recommendations. Typically, "minimum" recommendations have been presented. Although some risk will always remain, lower risk of future problems would usually result if more restrictive criteria were adopted. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

Respectfully submitted,

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Enclosures: Plates 1-7
Appendix A – References
Appendix B – Boring Logs
Appendix C – Summary of Laboratory Data
Appendix D – EQFault and Probabilistic Ground Motion Data
Appendix E – Slope Stability Analysis

Distribution: 2 – Addressee (Regular Mail and Email: ssudol@rbrady.net and jmendzer@eocwd.com)

wpdata/oc/33615-01.CDG.AA.April.2.2014.GeotechnicalAndSeismicHazardAssessmentReport

Source: Mapquest

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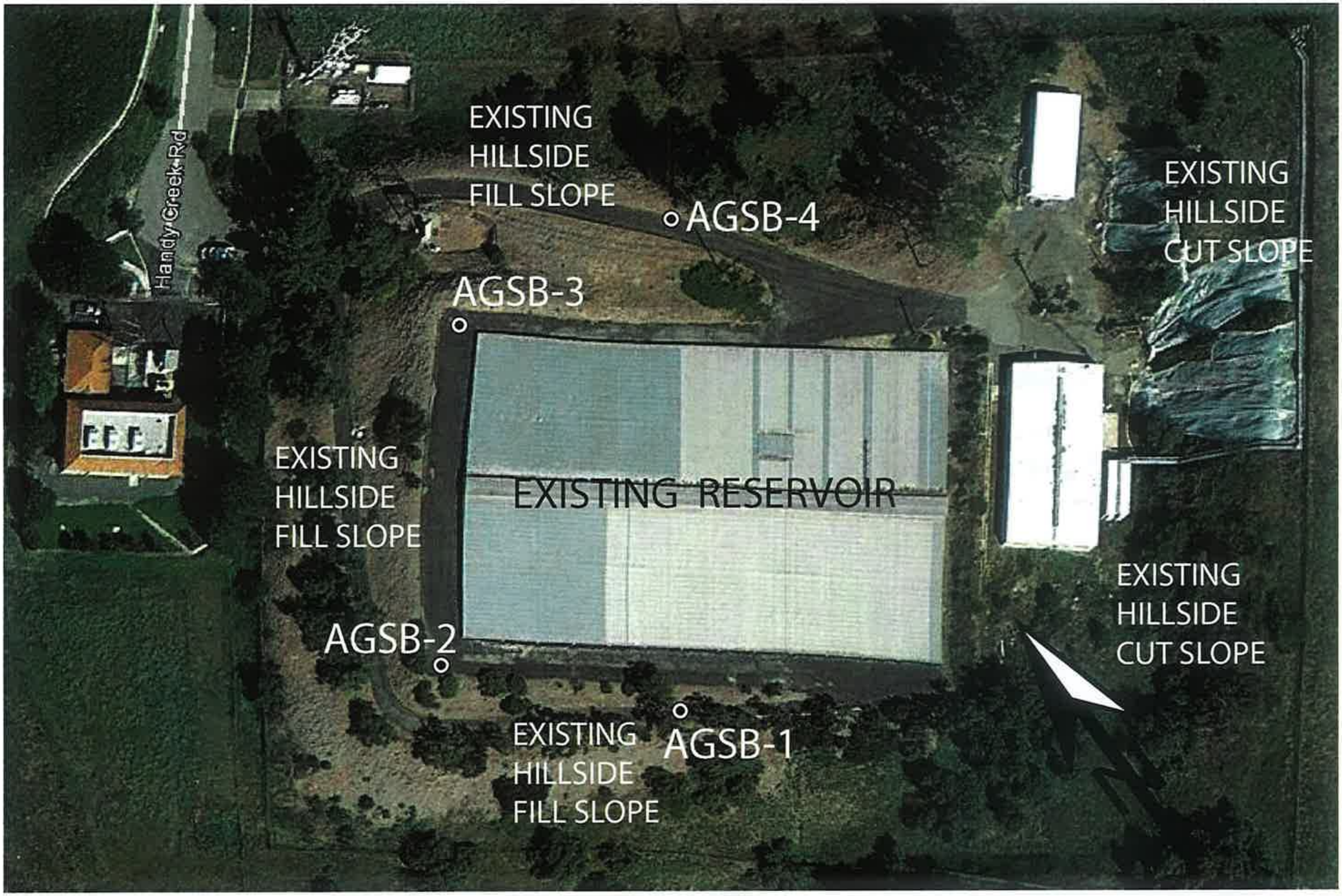
EAST ORANGE COUNTY WATER DISTRICT
PETERS CANYON FACILITY
HANDY CREEK ROAD RESERVOIR
ORANGE, CA

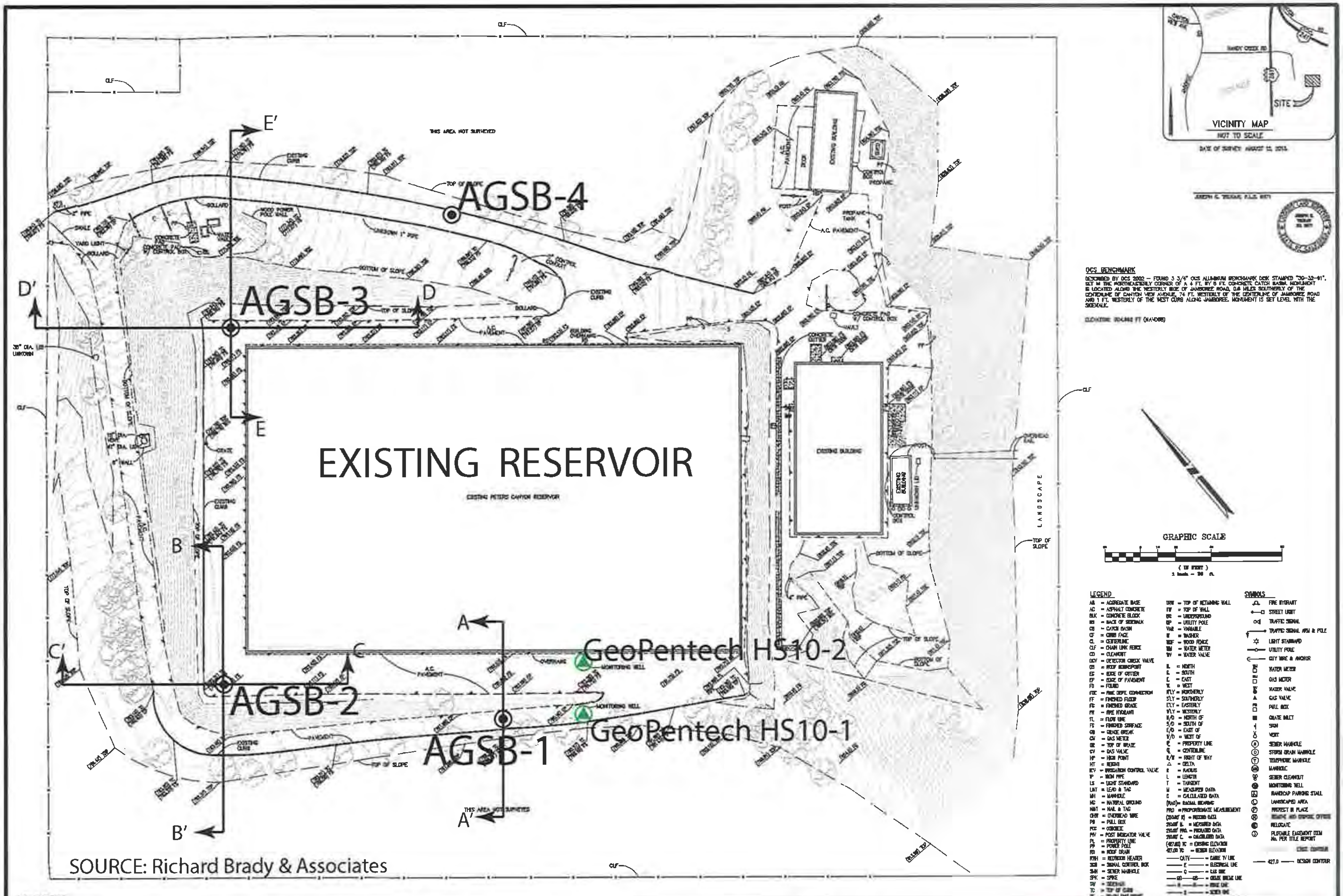
PROJECT LOCATION MAP

F.N. 33615.01 | February, 2014

Plate 1



American Geotechnical, Inc. 22725 Old Canal Road, Yorba Linda, CA 92887 Phone: (714) 685-3900, Fax: (714) 685-3909	EAST ORANGE COUNTY WATER DISTRICT PETERS CANYON FACILITY HANDY CREEK ROAD RESERVOIR ORANGE, CA	BORING LOCATION MAP F.N. 33615.01 February, 2014	Plate 2



SOURCE: Richard Brady & Associates

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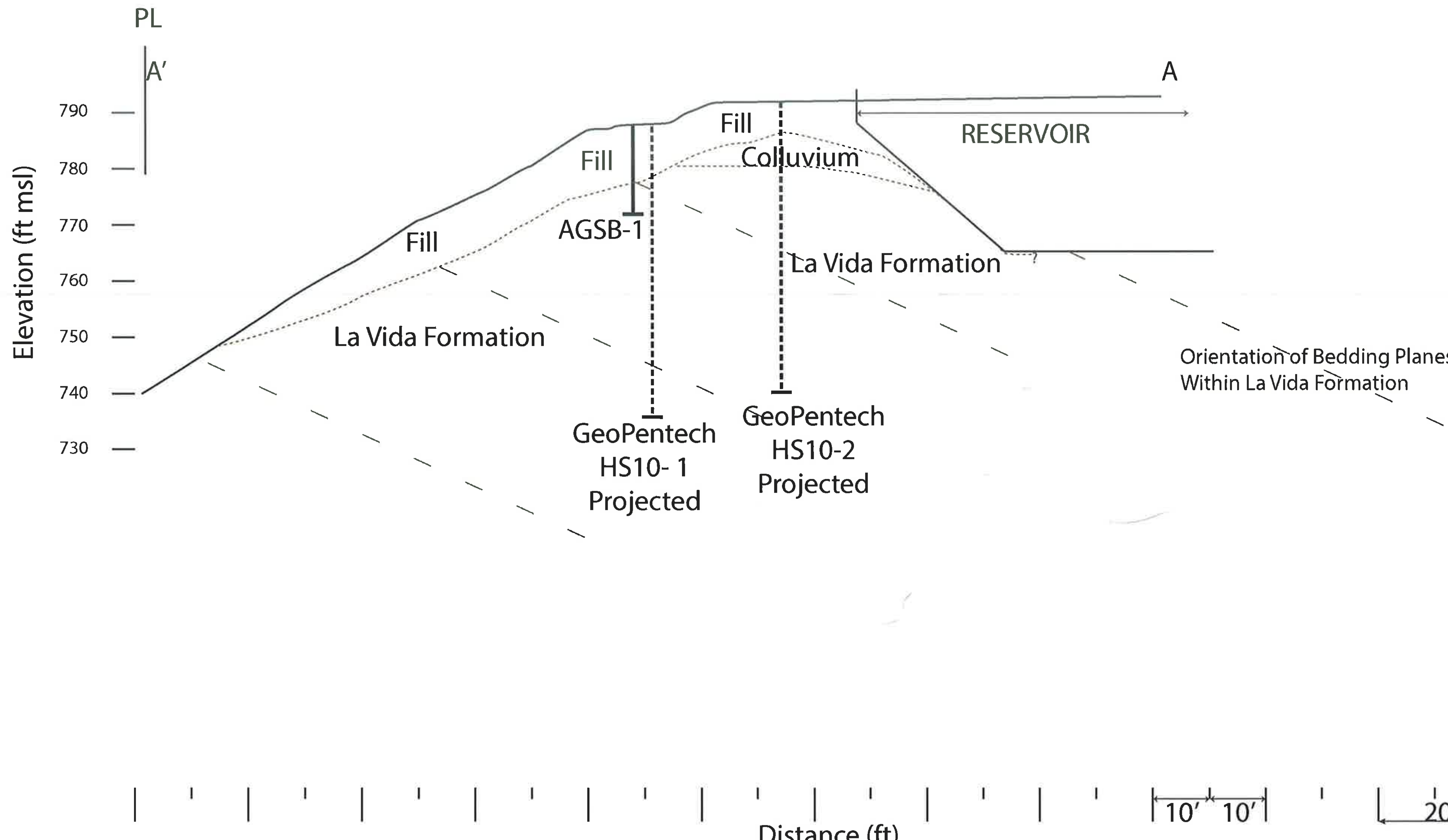
SITE TOPOGRAPHIC MAP

F.N. 33615.01

February, 2014

Plate 3



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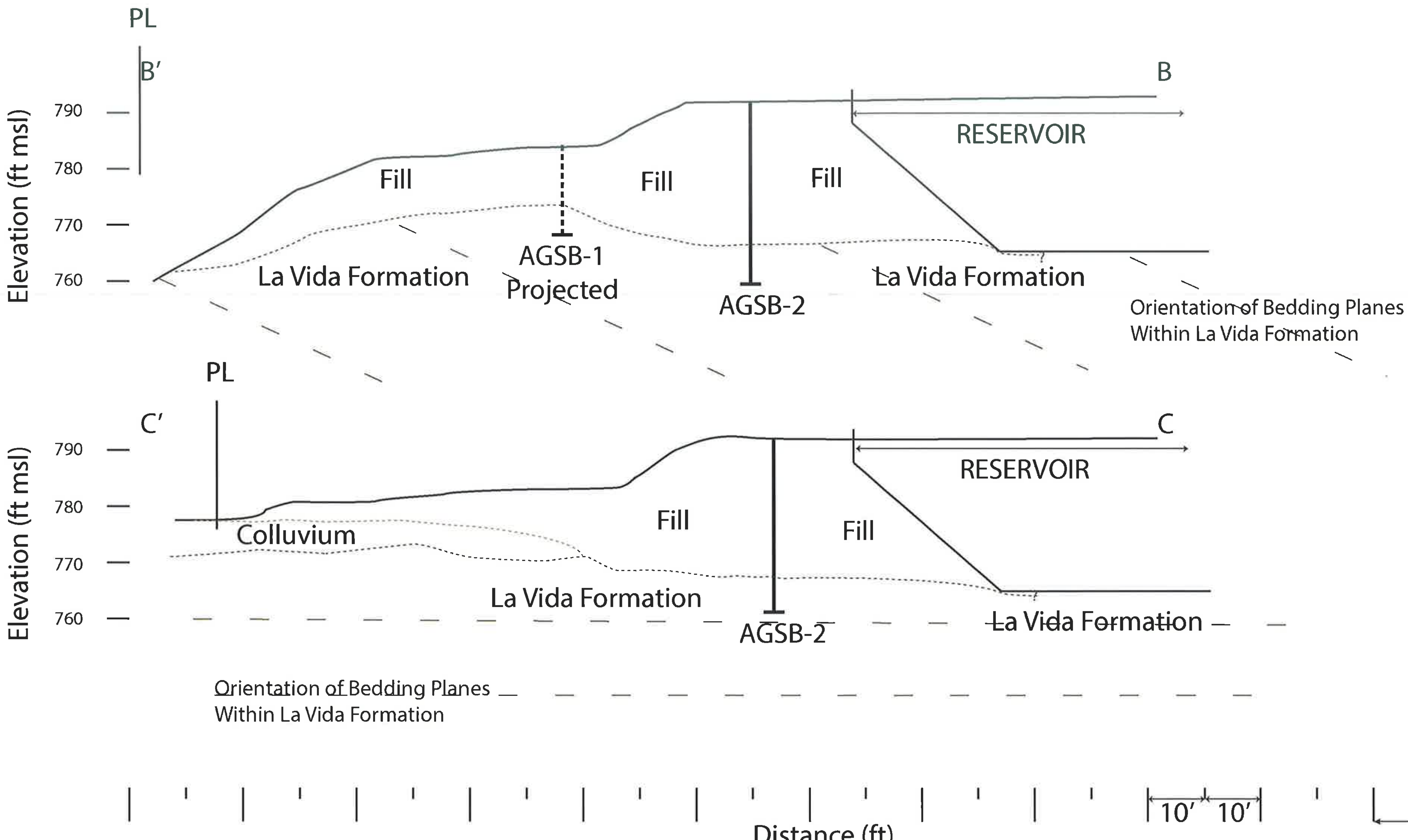
GEOLOGIC CROSS-SECTION A-A'

F.N. 33615.01

February, 2014

Plate 4



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GEOLOGIC CROSS-SECTIONS B-B' and C-C'

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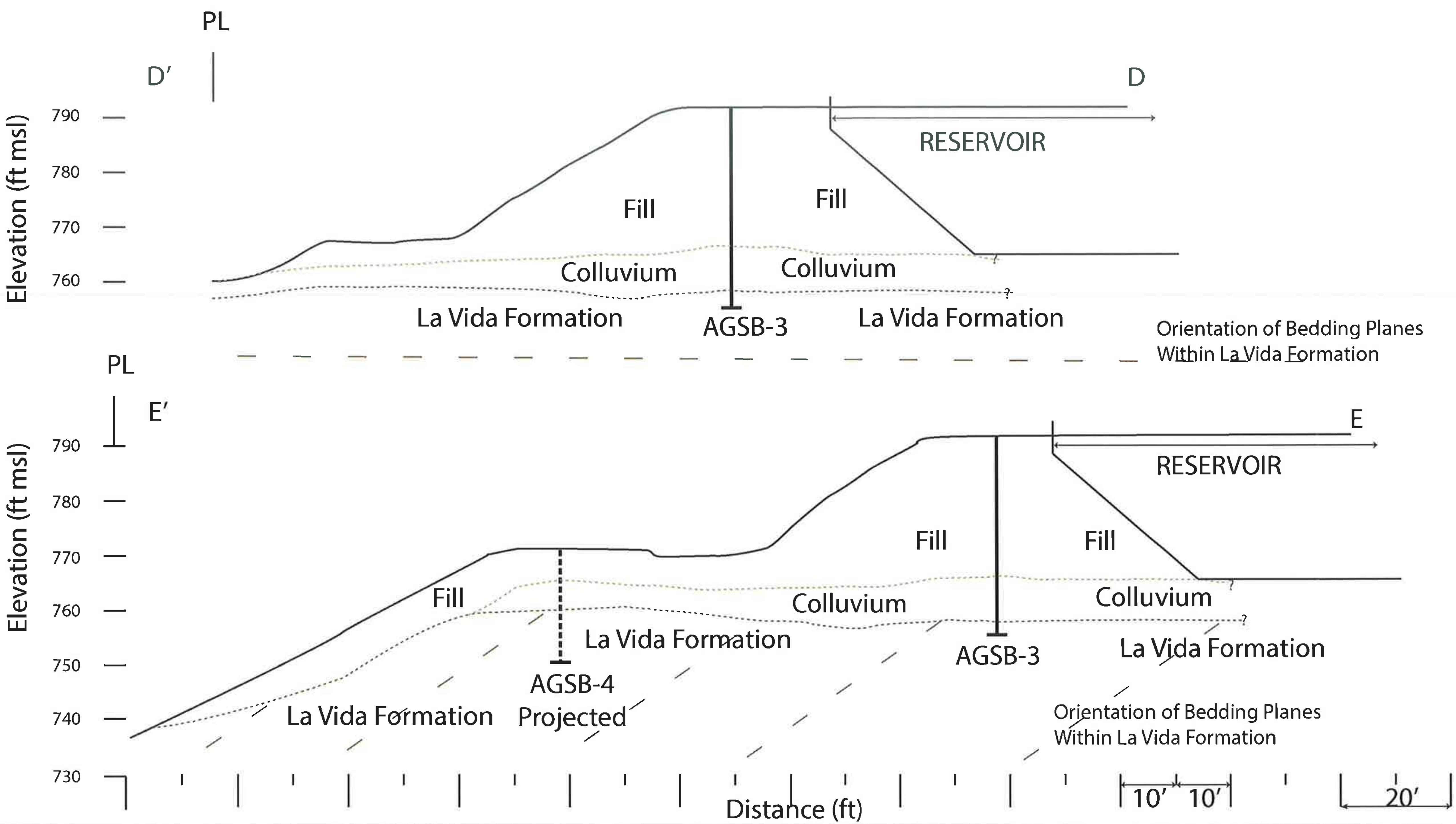
February, 2014

Plate 5



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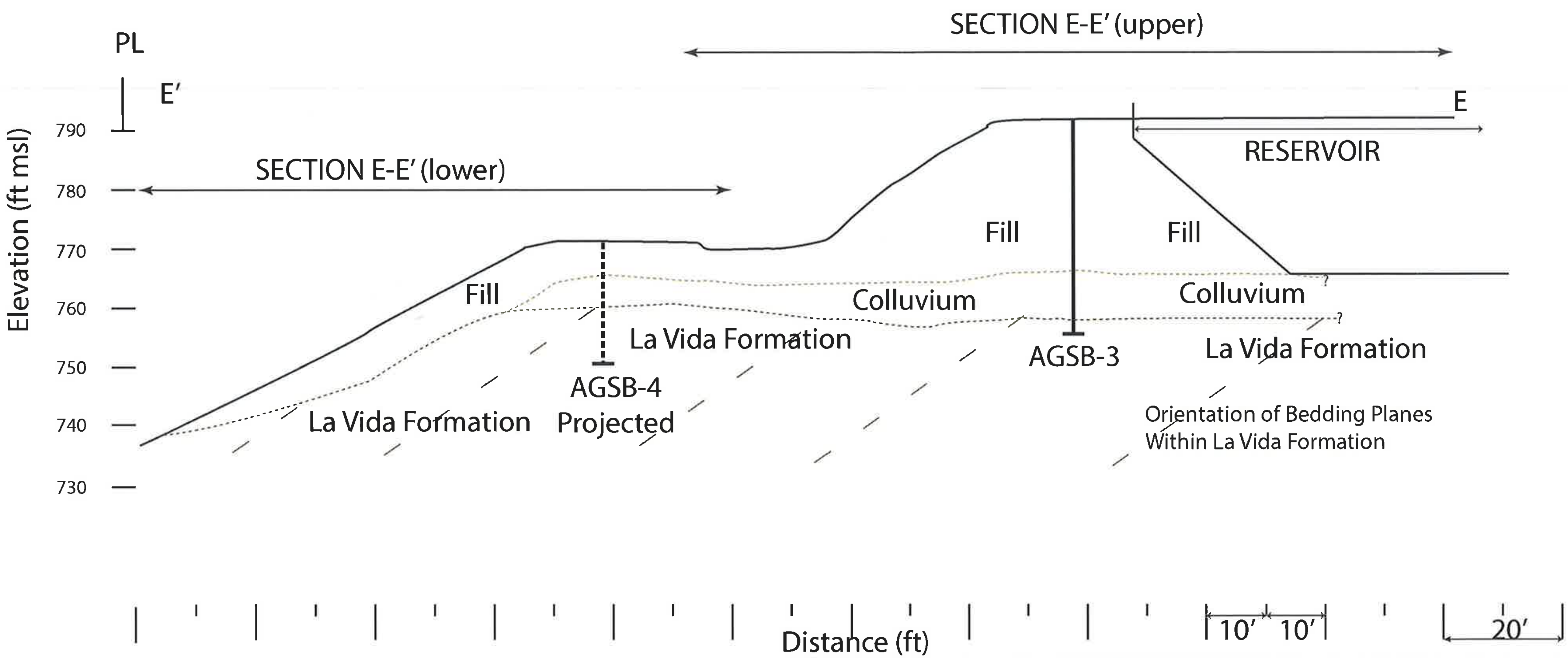
GEOLOGIC CROSS-SECTIONS D-D' and E-E'

Plate 6

F.N. 33615.01

February, 2014



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GEOLOGIC CROSS-SECTION E-E'
FOR SLOPE STABILITY ANALYSIS

F.N. 33615.01

February, 2014

Plate 7

APPENDIX A

REFERENCES

1. J.E. Shoellhamer and D.M. Kinney, R.F. Yerkes, and J.G. Vedder, Geologic Map of the Northern Santa Ana Mountains, Orange and Riverside Counties, California, United States Geologic Survey Oil and Gas Investigations Map OM 154, 1954.
2. P.K. Morton and R.V. Miller, Geologic Map of Orange County California Showing Mines and Mineral Deposits, California Division of Mines and Geology Bulletin 204, 1981.
3. Morton, D.M., “Preliminary Digital Geologic Map of the Santa Ana 30’ X 60’ Quadrangle, Southern California”, California Division of Mines and Geology, Open-File Report 99-192, Version 2.0, 2004.
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5. Richard Brady & Associates, Engineering Evaluation of the Peters Canyon Six Million Gallon Reservoir, prepared for East Orange County Water District, July 2012.
6. Joseph C. Truxaw and Associates, Inc., Topographic Survey East Orange County Water District, Peters Canyon Reservoir Plan Sheet, prepared for Richard Brady & Associates, dated 8/22/13.
7. Boyle Engineering, Construction Plans For The Earthwork In Connection With 6 M.G. Storage Reservoir for the East Orange County Water District Title Sheet 1 of 7, undated.
8. Boyle Engineering, E.O.C.W.D. Reservoir Plot Plan and Horizontal Control Plan Sheet 2 of 7, undated.
9. Boyle Engineering, E.O.C.W.D. 6 M/G. Reservoir Fill Disposal Areas Plan Sheet 7 of 7, undated.
10. Boyle Engineering, E.O.C.W.D. Reservoir Plot Plan and Horizontal Control Plan Sheet 2A of 6, undated.
11. Boyle Engineering, E.O.C.W.D. Reservoir Plot Plan and Horizontal Control Plan Sheet 2 of 7, undated, revised 9/21/65 for Trailer Pad Construction.

APPENDIX B

Boring Logs

BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Southern Access Road, Approx. 40-feet Southwest of Existing Monitoring Well

Ground Elev. (ft): **Total Depth (ft):** 16'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-1

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
0'						**1-1/2 Inch Asphalt/6-inch of Aggregate Base** **GRAVELLY CLAY**, olive-gray-brown, damp, dense (FILL)
						SILTY SAND, gray, damp, dense with some gravel (FILL)
5'			24/22 SPT 12/19/21			
10'			43/28 SPT 40/50/5"			**SILTY SAND (SILTSTONE)**, gray, damp, dense (La Vida Formation) moderate carbonate development
15'			33/50			Bottom of exploration 16-feet, no water in boring
20'						

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Southwest Corner of Perimeter Access Road

Ground Elev. (ft): **Total Depth (ft):** 31'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-2

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
0'						**1-Inch Asphalt** **SILTY SAND**, olive-gray, damp, dense with some gravel (FILL)
5'			23/30 SPT 14/20/23			
10'			25/26 SPT 18/24/26			
15'			16/26			**SILTY SAND**, orange-gray, damp, dense (FILL)
						SILTY SAND, olive, damp, dense with some gravel (FILL)
20'			18/14			

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Southwest Corner of Perimeter Access Road

Ground Elev. (ft): **Total Depth (ft):** 31'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-2 (continued)

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
20'						**SILTY SAND**, olive, damp, dense with some gravel (FILL)
			SPT 10/14/22			
25			50/6"			**SILTY SAND (SILTSTONE)**, olive-gray, damp, dense (La Vida Formation)
			SPT 50/6"			
30'			SPT 47/50/4"			**SILTY SAND (SILTSTONE)**, gray, damp, dense (La Vida Formation)
						Bottom of exploration 31-feet, no water in boring
35'						
40'						

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Northwest Corner of Perimeter Access Road

Ground Elev. (ft): **Total Depth (ft):** 36.5'

Drill Company: **J&H Drilling** **Drill Method:** **CME-75** **Vertical Drop (in):** **30"**

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-3

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
0'						**1-Inch Asphalt** **SILTY SAND**, olive-gray, damp to moist, dense with some gravel (FILL) interbedded lenses of medium-grained sand
5'			25/32 SPT 10/16/21			
10'			16/25 SPT 20/20/25			
15'			15/30 SPT 12/20/26			**SILTY SAND**, orange-gray, moist, dense (FILL) interbedded with lenses of olive-gray silty sand
20'			19/24			**SILTY SAND**, olive, damp to moist, dense (FILL)

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Northwest Corner of Perimeter Access Road

Ground Elev. (ft): **Total Depth (ft):** 36.5'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-3 (continued)

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
20'						**SILTY SAND**, olive, damp, dense (FILL)
			SPT 20/25/26			interbedded lenses of yellow sand and gray sand
25			13/16/29			
			SPT 23/24/26			**SAND**, yellow and gray, damp, dense (COLLUVIUM)
						mottled texture with some carbonate development
30'			SPT 21/22/27			
			18/23			
						SILTY SAND (SILTSTONE), gray, damp, dense (La Vida Formation)
35'			SPT 30/35/50			
						Bottom of exploration 36.5-feet, no water in boring
40'						

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Northern Access Road

Ground Elev. (ft): **Total Depth (ft):** 22.5'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-4

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
0'						**1-Inch Asphalt** **SILTY SAND**, mottled olive-gray and orange, moist, dense (FILL)
5'			22/35 SPT 11/27/31			**SILTY SAND**, orange, damp, dense, some gravel (COLLUVIUM)
10'			19/35			**SILTY SAND**, orange, damp, dense (COLLUVIUM)
15'			17/20 SPT 15/17/18			
20'			24/30			**SILTY SAND (SILTSTONE)**, dark gray, damp, dense (La Vida Formation)

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BORING LOG

Project Name: EOCWD Peters Canyon Facility 6MG Reservoir **File Number:** 33615-01

Logging By: c.glick **Drill Date(s)** February 10, 2014

Boring Location: Northern Access Road

Ground Elev. (ft): **Total Depth (ft):** 22.5'

Drill Company: J&H Drilling **Drill Method:** CME-75 **Vertical Drop (in):** 30"

Drive Weights: 140 lb. **Boring Diameter (in):** 8

Instrumentation Details:

BORING No. AG-SB-4 (continued)

Depth (Feet)	In-situ Samples	Bulk Samples	Blow Counts	Dry Unit Wt (PCF)	Moisture Content (%)	LITHOLOGIC DESCRIPTION
20'						**SILTY SAND (SILTSTONE)**, dark gray, damp, dense (La Vida Formation)
		SPT 16/19/24				
						Bottom of exploration 22.5-feet, no water in boring
25						
30'						
35'						
40'						

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APPENDIX C

Summary of Laboratory Testing

Exca. No.	Depth (feet)	In-Situ Dry Density (Lbs/cu. ft.)	In-Situ Moisture Content (%)	Maximum Dry Density (Lbs/cu. ft.)	Optimum Moisture Content (%)	Relative* Compaction (%)	Degree of** Saturation (%)	Soil Chemical Testing Chloride (mg/kg)	pH (-)	Resistivity (Ohm-cm)	Sulfate (mg/kg)
AGSB-1	2.0-5.0	-	-	117.2	13.8	-	-	-	-	-	-
AGSB-1	5.5-6.0	98.1	7.4	-	-	84	28	-	-	-	-
AGSB-1	10.5-11.0	104.9	12.5	-	-	-	56	-	-	-	-
AGSB-1	15.5-16.0	94.4	21.8	-	-	-	75	-	-	-	-
AGSB-2	1.0-5.0	-	-	-	-	-	-	16.2	7.9	3570	285
AGSB-2	5.5-6.0	93.6	20.4	-	-	81	69	-	-	-	-
AGSB-2	12.0-15.0	-	-	115.2	14	-	-	-	-	-	-
AGSB-2	10.5-11.0	96.3	11.2	-	-	84	40	-	-	-	-
AGSB-2	15.5-16.0	95.4	12.6	-	-	83	45	-	-	-	-
AGSB-2	20.5-21.0	87.0	13.6	-	-	76	39	-	-	-	-
AGSB-2	25.0-25.5	108.3	6.8	-	-	-	33	-	-	-	-
AGSB-3	5.5-6.0	99.4	15.0	-	-	84	58	-	-	-	-
AGSB-3	10.5-11.0	102.8	18.7	-	-	87	79	-	-	-	-
AGSB-3	15.0-19.0	-	-	118	13.4	-	-	-	-	-	-
AGSB-3	15.5-16.0	104.4	11.2	-	-	88	49	-	-	-	-
AGSB-3	20.5-21.0	95.4	21.6	-	-	81	76	-	-	-	-
AGSB-3	25.5-26.0	96.4	17.4	-	-	-	63	-	-	-	-
AGSB-4	5.5-6.0	103.2	8.6	-	-	-	37	-	-	-	-
AGSB-4	10.5-11.0	100.1	10.0	-	-	-	39	-	-	-	-
AGSB-4	15.5-16.0	103.4	11.3	-	-	-	49	-	-	-	-

Note: * Relative compaction was calculated using the nearest maximum dry density
Note: ** Degree of saturation was calculated using specific gravity of 2.70

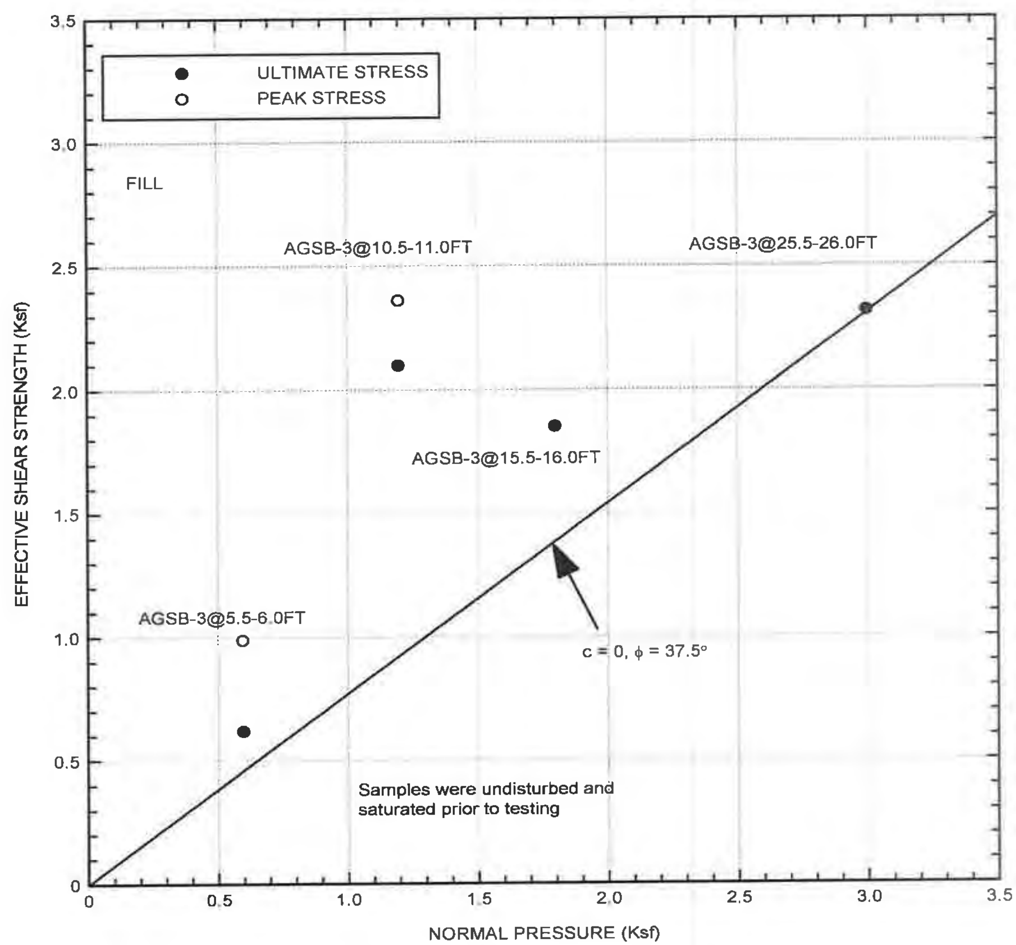
SUMMARY OF LABORATORY TESTING DATA				TABLE C1
AMERICAN GEOTECHNICAL	EOCWD	F.N. 33615.01	APRIL 2014	

Exca. No.	Depth (ft)	Soil Classification USCS	Grain Size Distribution Gravel (%)	Sand (%)	Silt (%)	Clay (%)	% Passing No. 200 (%)	Atterberg Limits LL (%)	PL (%)	PI (%)
AGSB-1	2-5	CL	0	45.1	39.9	15	54.9	32	20	12
AGSB-2	12-15	CL	0	55.9	33.9	10.2	44.1	-	-	-
AGSB-3	5-10	CL	0	44.4	38.3	17.3	55.6	-	-	-
AGSB-3	15-19	CL	0	49.1	39.2	11.7	50.9	31	22	9

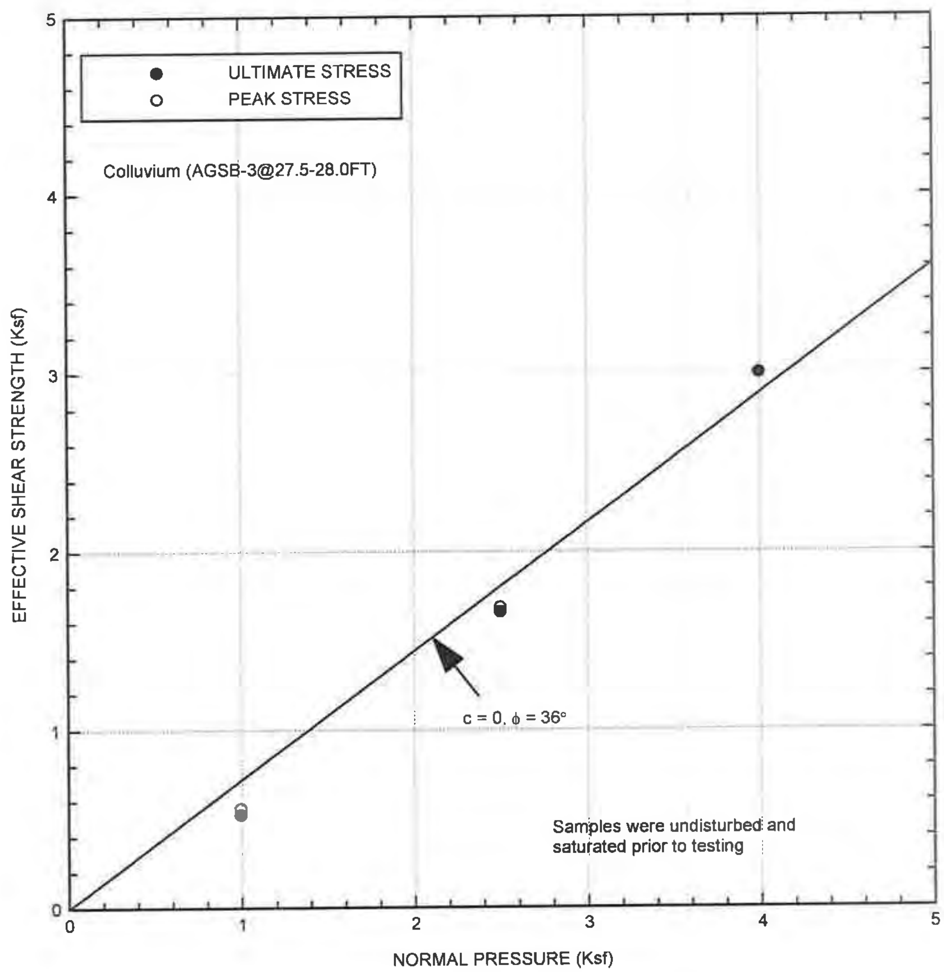
SUMMARY OF LABORATORY TESTING DATA

AMERICAN GEOTECHNICAL | EOCWD | F.N. 33615.01 | APRIL 2014

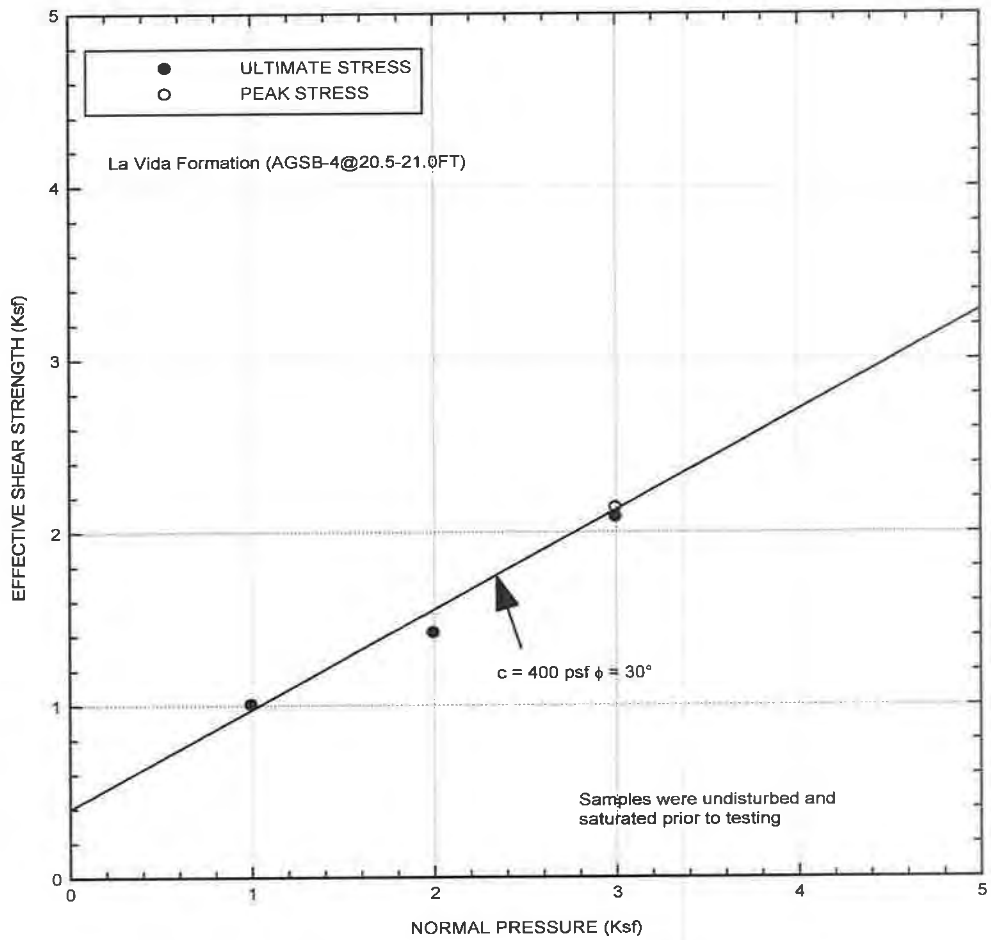
TABLE C2



DIRECT SHEAR TEST PLOT				FIGURE C1
AMERICAN GEOTECHNICAL	EOCWD	F.N. 33615.01	APRIL 2014	

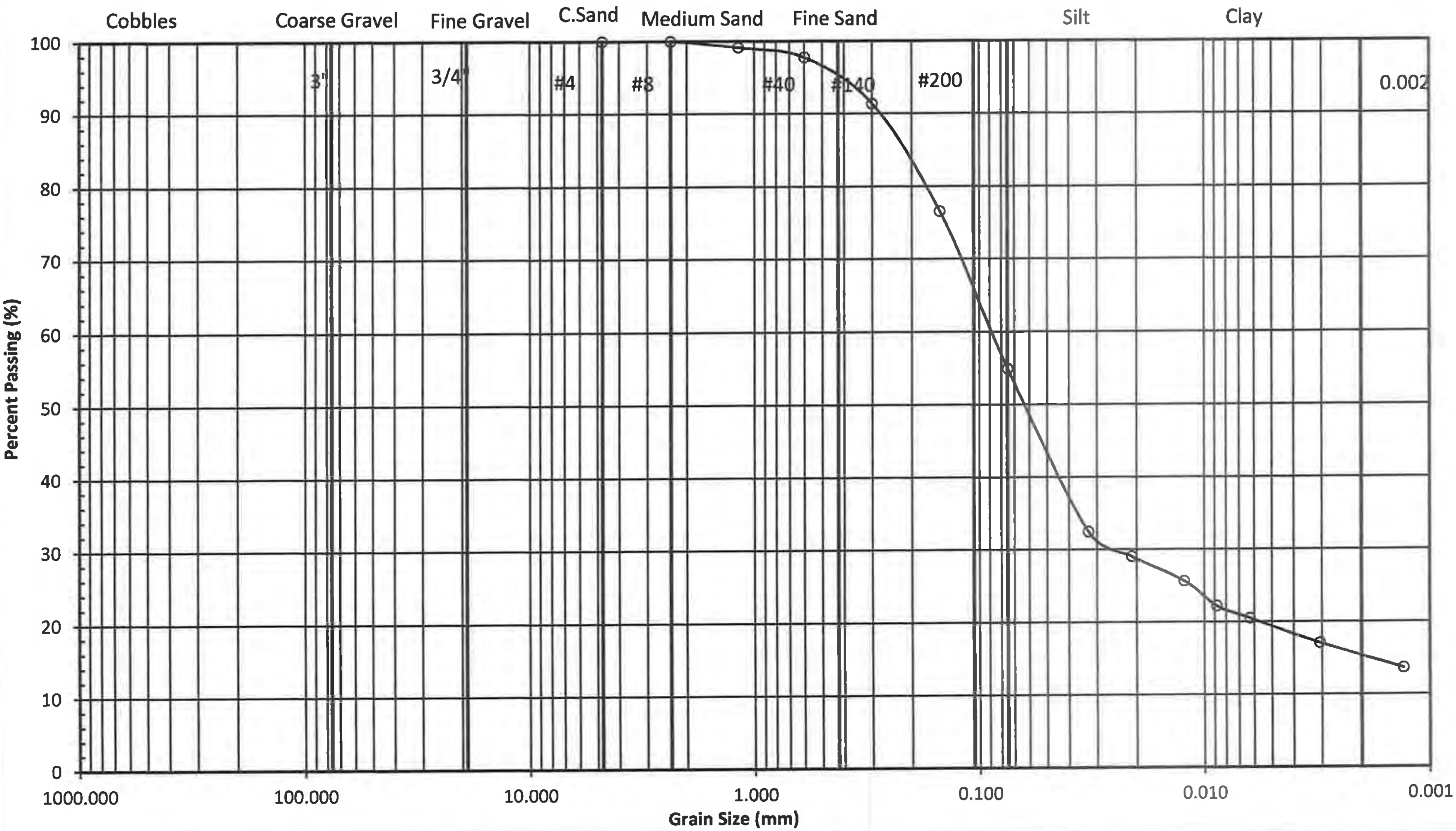


DIRECT SHEAR TEST PLOT				FIGURE C2
AMERICAN GEOTECHNICAL	EOCWD	F.N. 33615.01	APRIL 2014	



DIRECT SHEAR TEST PLOT				FIGURE C3
AMERICAN GEOTECHNICAL	EOCWD	F.N. 33615.01	APRIL 2014	

Grain Size



Project Name: EOCWD
Location: Handy Creek Road
File No: 33615-01
Date: 2/17/2014
Excavation: AGSB-1 Bag 1
Depth: 2' - 5'
By: SM

% Gravel =	0.0
% Sand =	45.1
% Silt =	39.9
% Clay =	15.0
Sum =	100.0

LL= 32 PL= 20 PI= 12

%>#200 = 45.1
%<#200 = 54.9

Soil Classification: Sandy Lean Clay (CL)

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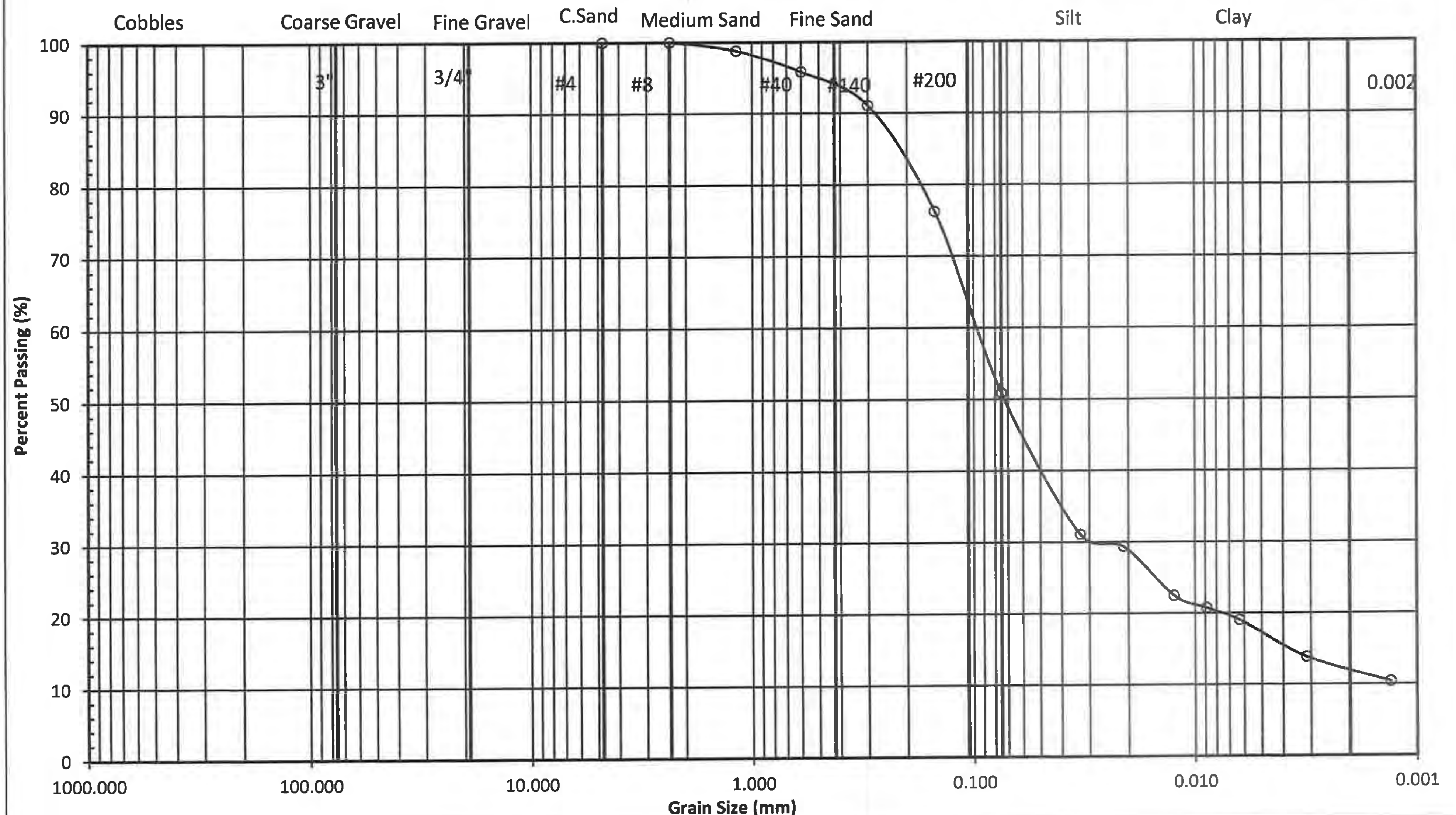
5600 Spring Mountain Road Suite 201 Las Vegas NV 89146 - (702) 562-5046 - FAX (702) 562-2457

Grain Size



Project Name: EOCWD
Location: Handy Creek Road
File No: 33615-01
Date: 2/18/2014
Excavation: AGSB-3 Bag-2
Depth: 15' - 19'
By: KV

% Gravel =	0.0
% Sand =	49.1
% Silt =	39.2
% Clay =	11.7
Sum =	100.0

LL= 31 PL= 22 PI= 9

%>#200 = 49.1
%<#200 = 50.9

Soil Classification: Sandy Lean Clay (CL)

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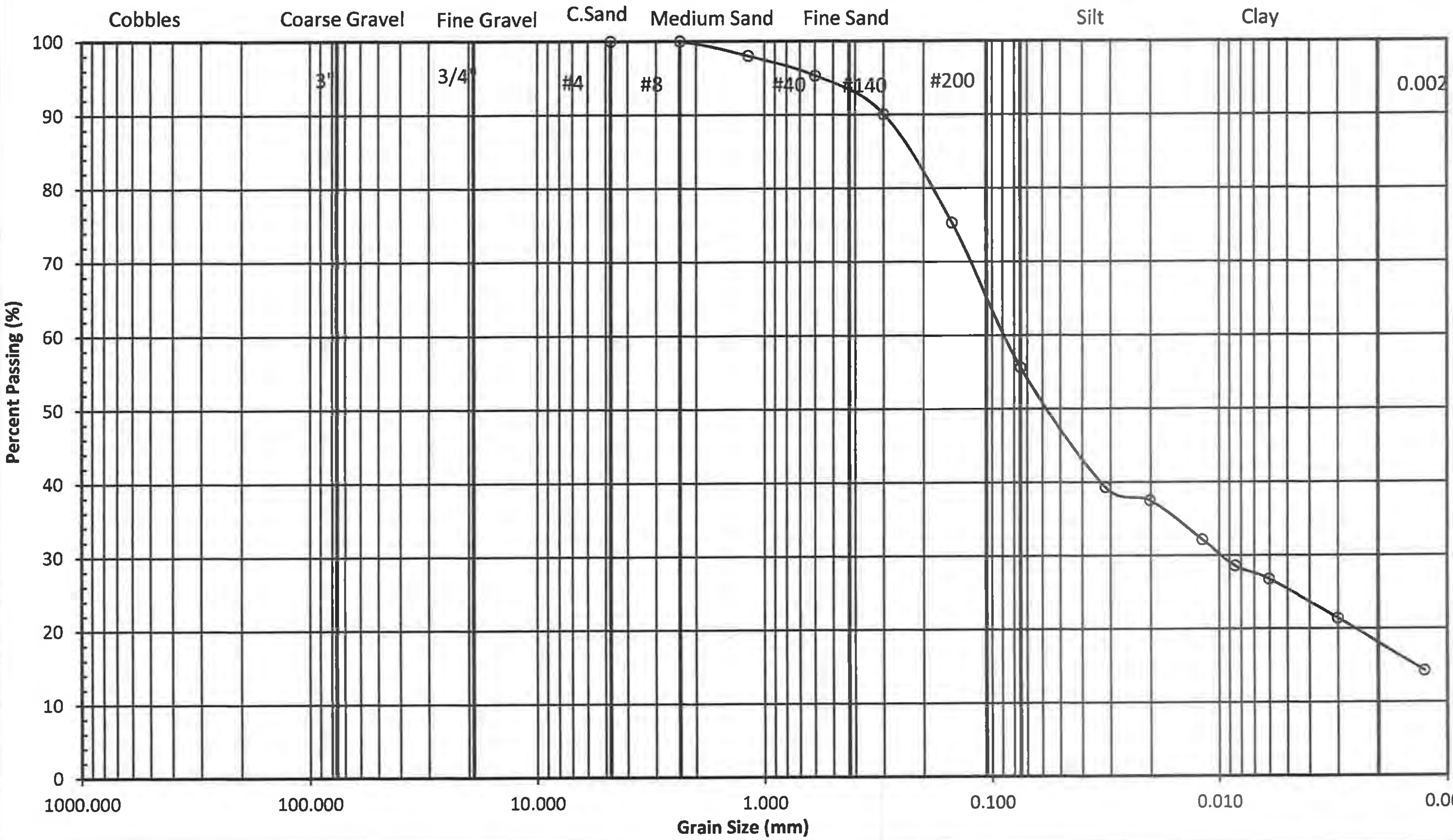
3100 Fite Circle Suite 103 Sacramento CA 95827 - (916) 368-2088 - FAX (916) 368-2188
5600 Spring Mountain Road Suite 201 Las Vegas NV 89146 - (702) 562-5046 - FAX (702) 562-2457

Grain Size



Project Name: EOCWD
Location: Handy Creek Road
File No: 33615-01
Date: 2/28/2014
Excavation: AGSB-3 Bag-1
Depth: 5' - 10'
By: KV

% Gravel =	0.0
% Sand =	44.4
% Silt =	38.3
% Clay =	17.3
Sum =	100.0

LL= N/A PL= N/A PI=

%>#200 = 44.4
%<#200 = 55.6

Soil Classification: No atterberg assigned

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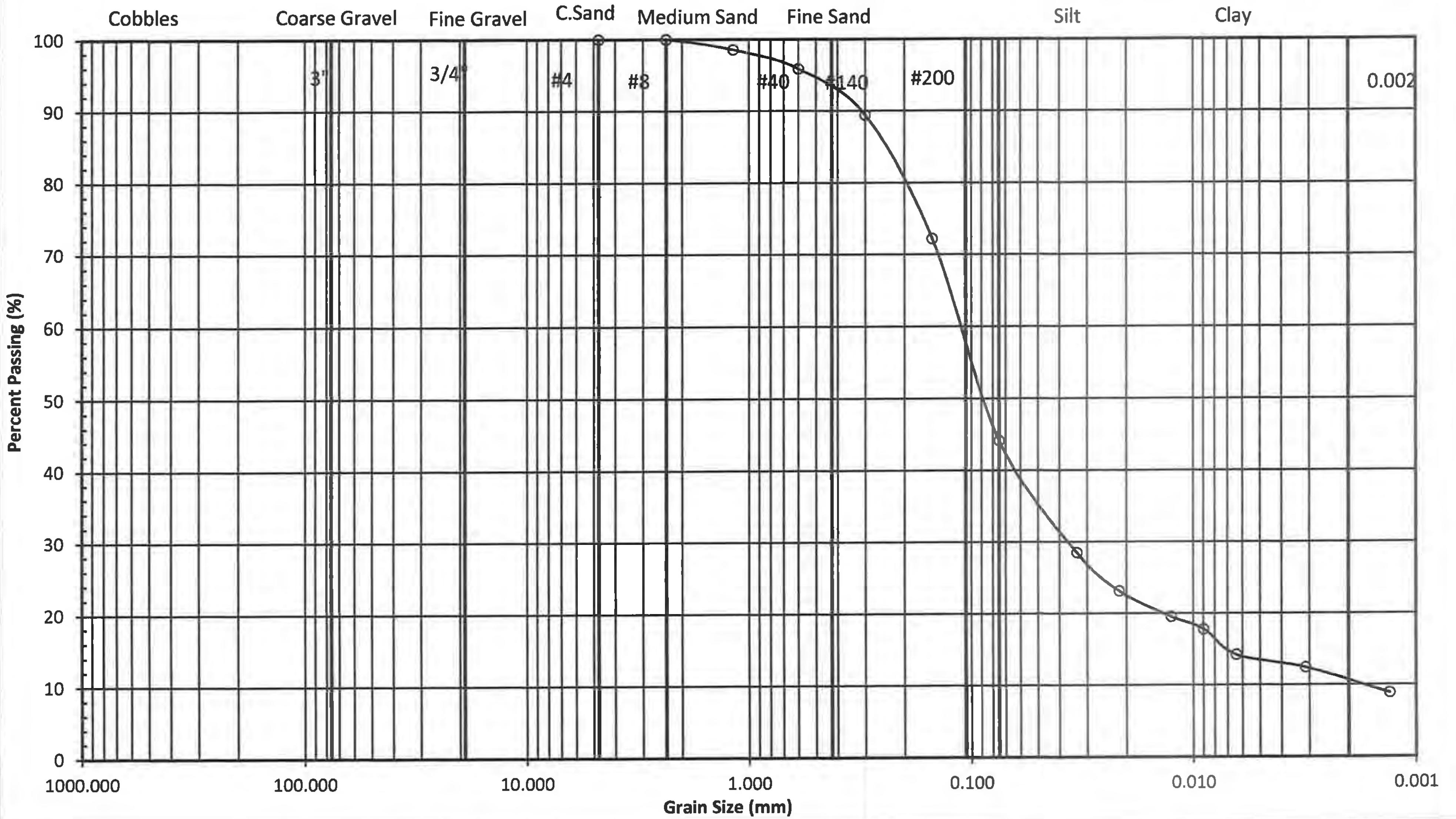
Grain Size



Project Name: EOCWD
Location: Handy Creek Road
File No: 33615-01
Date: 2/17/2014
Excavation: AGSB-2 Bag 2
Depth: 12' - 15'
By: KV

% Gravel =	0.0
% Sand =	55.9
% Silt =	33.9
% Clay =	10.2
Sum =	100.0

LL= N/A PL= N/A PI=

%>#200 = 55.9
%<#200 = 44.1

Soil Classification: Non-Plasitc

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3100 Fite Circle Suite 103 Sacramento CA 95827 - (916) 368-2088 - FAX (916) 368-2188
5600 Spring Mountain Road Suite 201 Las Vegas NV 89146 - (702) 562-5046 - FAX (702) 562-2457

APPENDIX D

EQFault and Probabilistic Ground Motion Data

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************************
*                      *
*    E Q F A U L T     *
*                      *
*    Version 3.00      *
*                      *
************************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 33615-01

DATE: 02-21-2014

JOB NAME: EOCWD 6MG Reservoir

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.7765
   SITE LONGITUDE: 117.7552

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 15) Campbell & Bozorgnia (1997 Rev.) - Soft Rock
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 1   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
WHITTIER	7.8(12.5)	6.8	0.316	IX
ELSINORE-GLEN IVY	8.8(14.2)	6.8	0.282	IX
CHINO-CENTRAL AVE. (Elsinore)	9.8(15.7)	6.7	0.287	IX
ELYSIAN PARK THRUST	12.2(19.6)	6.7	0.219	IX
COMPTON THRUST	14.1(22.7)	6.8	0.193	VIII
NEWPORT-INGLEWOOD (L.A.Basin)	15.3(24.6)	6.9	0.166	VIII
NEWPORT-INGLEWOOD (Offshore)	15.8(25.5)	6.9	0.159	VIII
SAN JOSE	20.1(32.3)	6.5	0.095	VII
ELSINORE-TEMECULA	25.2(40.6)	6.8	0.079	VII
CUCAMONGA	25.8(41.6)	7.0	0.094	VII
SIERRA MADRE	25.8(41.6)	7.0	0.094	VII
PALOS VERDES	26.2(42.2)	7.1	0.096	VII
RAYMOND	31.1(50.1)	6.5	0.049	VI
SAN JACINTO-SAN BERNARDINO	32.6(52.4)	6.7	0.051	VI
CLAMSHELL-SAWPIT	32.9(53.0)	6.5	0.044	VI
VERDUGO	34.1(54.8)	6.7	0.049	VI
SAN JACINTO-SAN JACINTO VALLEY	34.1(54.9)	6.9	0.056	VI
CORONADO BANK	36.5(58.8)	7.4	0.078	VII
HOLLYWOOD	36.7(59.0)	6.4	0.035	V
SAN ANDREAS - Southern	38.5(61.9)	7.4	0.073	VII
SAN ANDREAS - San Bernardino	38.5(61.9)	7.3	0.067	VI
SAN ANDREAS - Mojave	39.1(63.0)	7.1	0.055	VI
SAN ANDREAS - 1857 Rupture	39.1(63.0)	7.8	0.099	VII
CLEGHORN	40.6(65.3)	6.5	0.031	V
SANTA MONICA	43.5(70.0)	6.6	0.031	V
NORTH FRONTAL FAULT ZONE (West)	45.7(73.6)	7.0	0.039	V
SIERRA MADRE (San Fernando)	47.9(77.1)	6.7	0.029	V
SAN GABRIEL	48.1(77.4)	7.0	0.037	V
MALIBU COAST	48.2(77.5)	6.7	0.029	V
SAN JACINTO-ANZA	48.2(77.6)	7.2	0.044	VI
ROSE CANYON	48.6(78.2)	6.9	0.034	V
NORTHRIDGE (E. Oak Ridge)	50.5(81.3)	6.9	0.031	V
ELSINORE-JULIAN	50.8(81.8)	7.1	0.038	V
ANACAPA-DUME	57.0(91.7)	7.3	0.035	V
SANTA SUSANA	57.8(93.1)	6.6	0.020	IV
PINTO MOUNTAIN	62.4(100.5)	7.0	0.026	V
HOLSER	62.7(100.9)	6.5	0.016	IV
NORTH FRONTAL FAULT ZONE (East)	62.9(101.3)	6.7	0.019	IV
HELENDALE - S. LOCKHARDT	65.3(105.1)	7.1	0.026	V
OAK RIDGE (Onshore)	68.8(110.8)	6.9	0.019	IV

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SIMI-SANTA ROSA	70.2(112.9)	6.7	0.016	IV
SAN ANDREAS - Coachella	74.6(120.0)	7.1	0.021	IV
SAN CAYETANO	74.6(120.1)	6.8	0.015	IV
SAN JACINTO-COYOTE CREEK	75.0(120.7)	6.8	0.016	IV
LENWOOD-LOCKHART-OLD WOMAN SPRGS	75.7(121.9)	7.3	0.025	V
SAN ANDREAS - Carrizo	77.1(124.1)	7.2	0.022	IV
EARTHQUAKE VALLEY	79.0(127.1)	6.5	0.012	III
BURNT MTN.	80.0(128.8)	6.4	0.010	III
JOHNSON VALLEY (Northern)	80.5(129.6)	6.7	0.013	III
LANDERS	81.4(131.0)	7.3	0.023	IV
EUREKA PEAK	81.8(131.7)	6.4	0.010	III
SANTA YNEZ (East)	85.7(138.0)	7.0	0.016	IV
OAK RIDGE(Blind Thrust Offshore)	86.4(139.1)	6.9	0.013	III
EMERSON So. - COPPER MTN.	87.3(140.5)	6.9	0.014	IV
CHANNEL IS. THRUST (Eastern)	87.9(141.5)	7.4	0.019	IV
VENTURA - PITAS POINT	89.1(143.4)	6.8	0.012	III
GRAVEL HILLS - HARPER LAKE	89.4(143.8)	6.9	0.014	III
CALICO - HIDALGO	93.3(150.2)	7.1	0.015	IV
MONTALVO-OAK RIDGE TREND	93.3(150.2)	6.6	0.009	III
GARLOCK (West)	93.9(151.1)	7.1	0.015	IV
M.RIDGE-ARROYO PARIDA-SANTA ANA	95.0(152.9)	6.7	0.010	III
BLACKWATER	96.1(154.6)	6.9	0.012	III
ELSINORE-COYOTE MOUNTAIN	97.7(157.3)	6.8	0.011	III
PLEITO THRUST	97.7(157.3)	7.2	0.014	IV
PISGAH-BULLION MTN.-MESQUITE LK	98.1(157.8)	7.1	0.014	IV
SAN JACINTO - BORREGO	98.1(157.9)	6.6	0.009	III
RED MOUNTAIN	98.7(158.9)	6.8	0.010	III

****

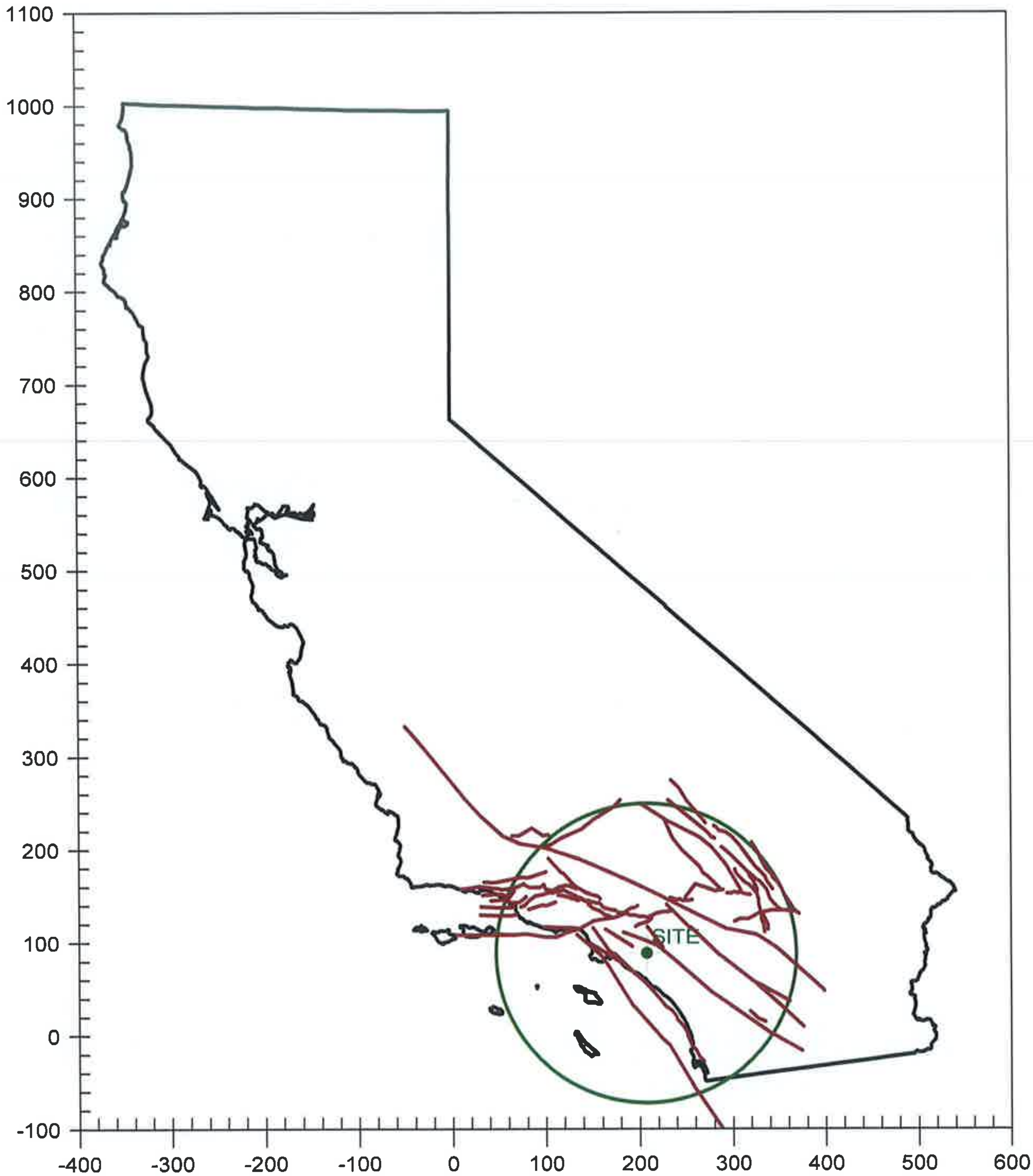
-END OF SEARCH- 67 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE WHITTIER FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.8 MILES (12.5 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3162 g

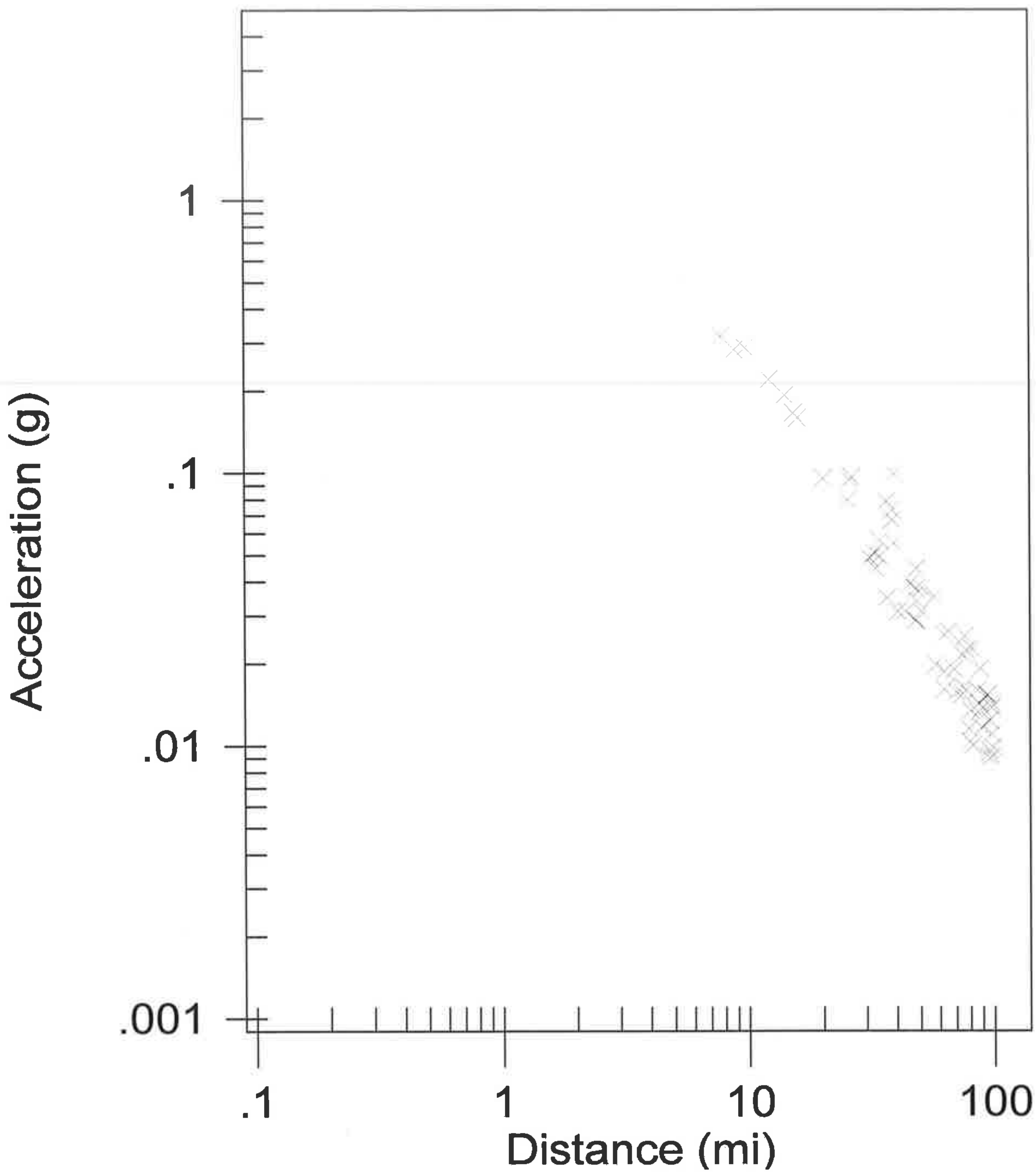
CALIFORNIA FAULT MAP

EOCWD 6MG Reservoir

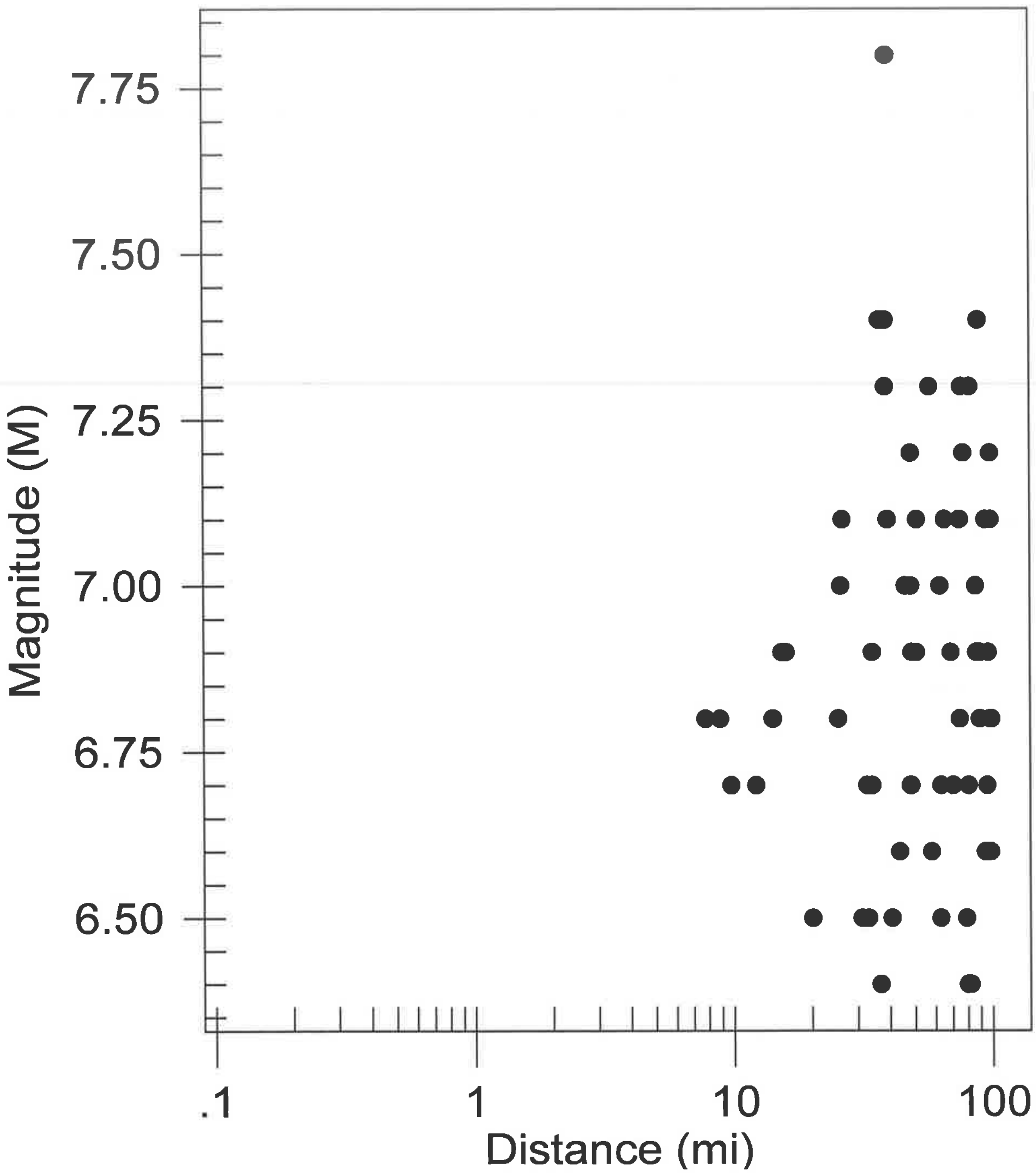


MAXIMUM EARTHQUAKES

EOCWD 6MG Reservoir

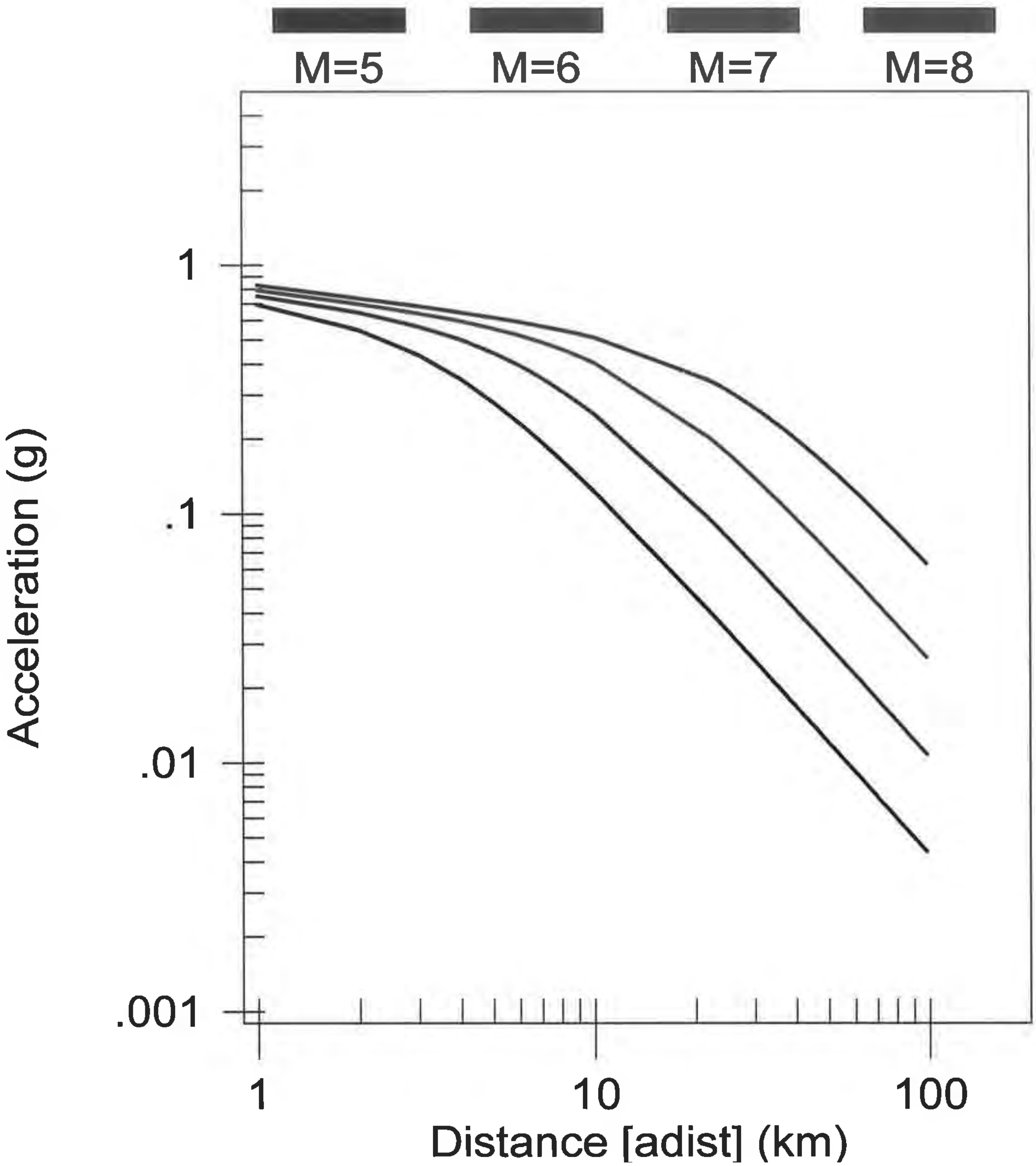


EARTHQUAKE MAGNITUDES & DISTANCES
EOCWD 6MG Reservoir



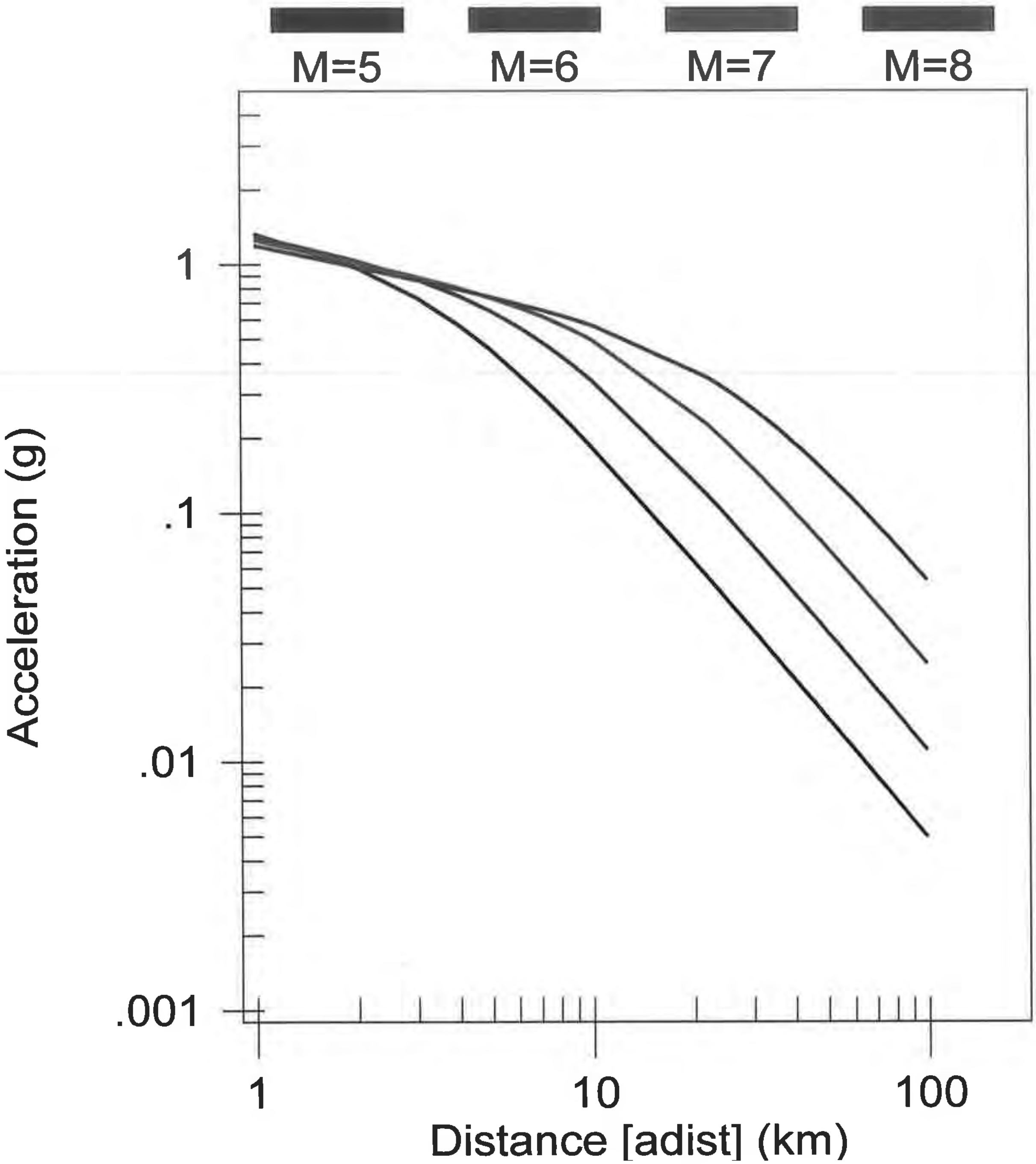
STRIKE-SLIP FAULTS

15) Campbell & Bozorgnia (1997 Rev.) - Soft Rock



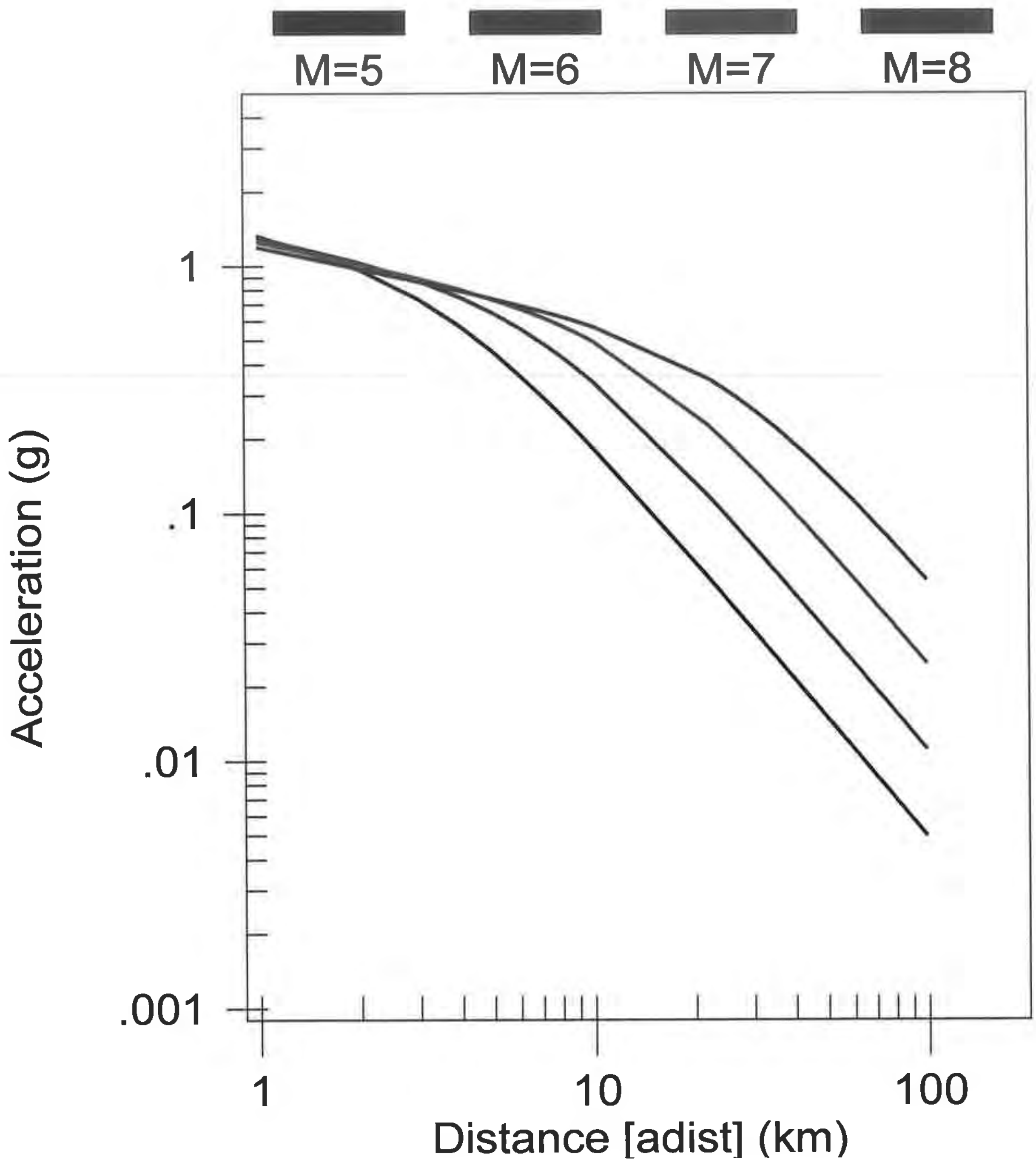
DIP-SLIP FAULTS

15) Campbell & Bozorgnia (1997 Rev.) - Soft Rock



BLIND-THRUST FAULTS

15) Campbell & Bozorgnia (1997 Rev.) - Soft Rock



Earthquake Hazards Program

US Seismic Hazard 2008

Layers Tools

Hazard maps

Transparency

Peak ground acceleration (%g) with 10% probability of exceedance in 50 years

Peak ground acceleration (%g) with 2% probability of exceedance in 50 years

0.2-second acceleration with 10% probability of exceedance in 50 years

0.2-second acceleration with 2% probability of exceedance in 50 years

1.0-second acceleration with 10% probability of exceedance in 50 years

1.0-second acceleration with 2% probability of exceedance in 50 years

Hazard map faults

Basemap

Transparency

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USGS
science for a changing world

Earthquake Hazards Program

US Seismic Hazard 2008

Layers Tools

Hazard maps

Transparency

Peak ground acceleration (%g) with 10% probability of exceedance in 50 years

Peak ground acceleration (%g) with 2% probability of exceedance in 50 years

0.2-second acceleration with 10% probability of exceedance in 50 years

0.2-second acceleration with 2% probability of exceedance in 50 years

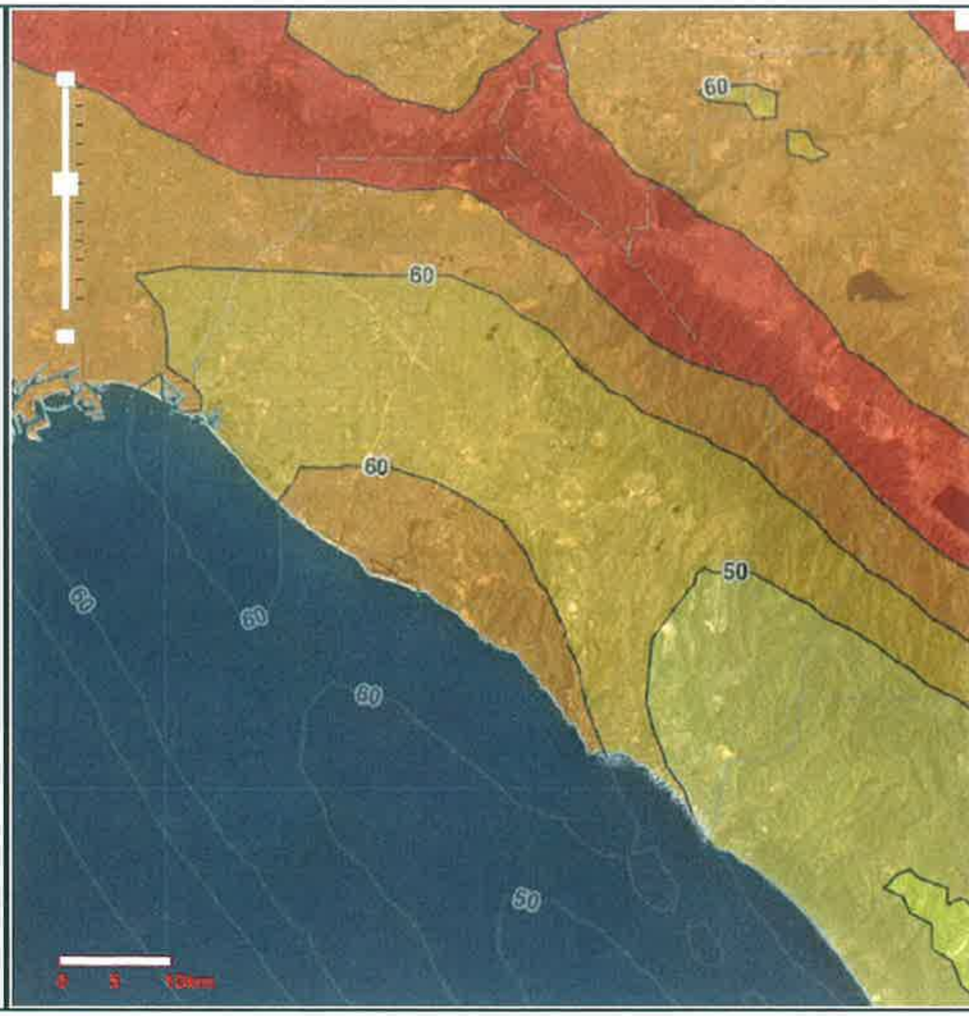
1.0-second acceleration with 10% probability of exceedance in 50 years

1.0-second acceleration with 2% probability of exceedance in 50 years

Hazard map faults

Basemap

Transparency



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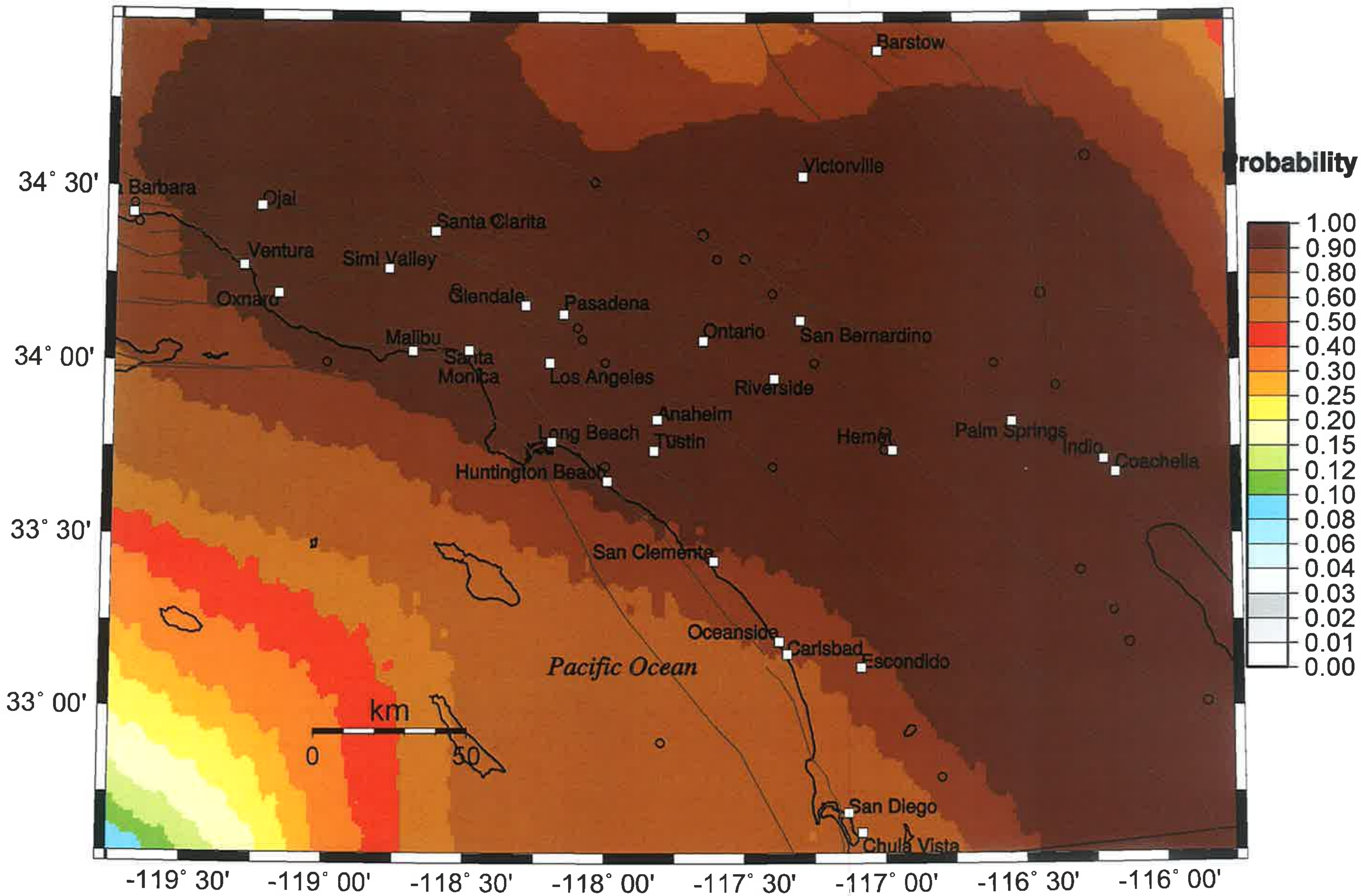
Probability of earthquake with M > 5.0 within 50 years & 50 km

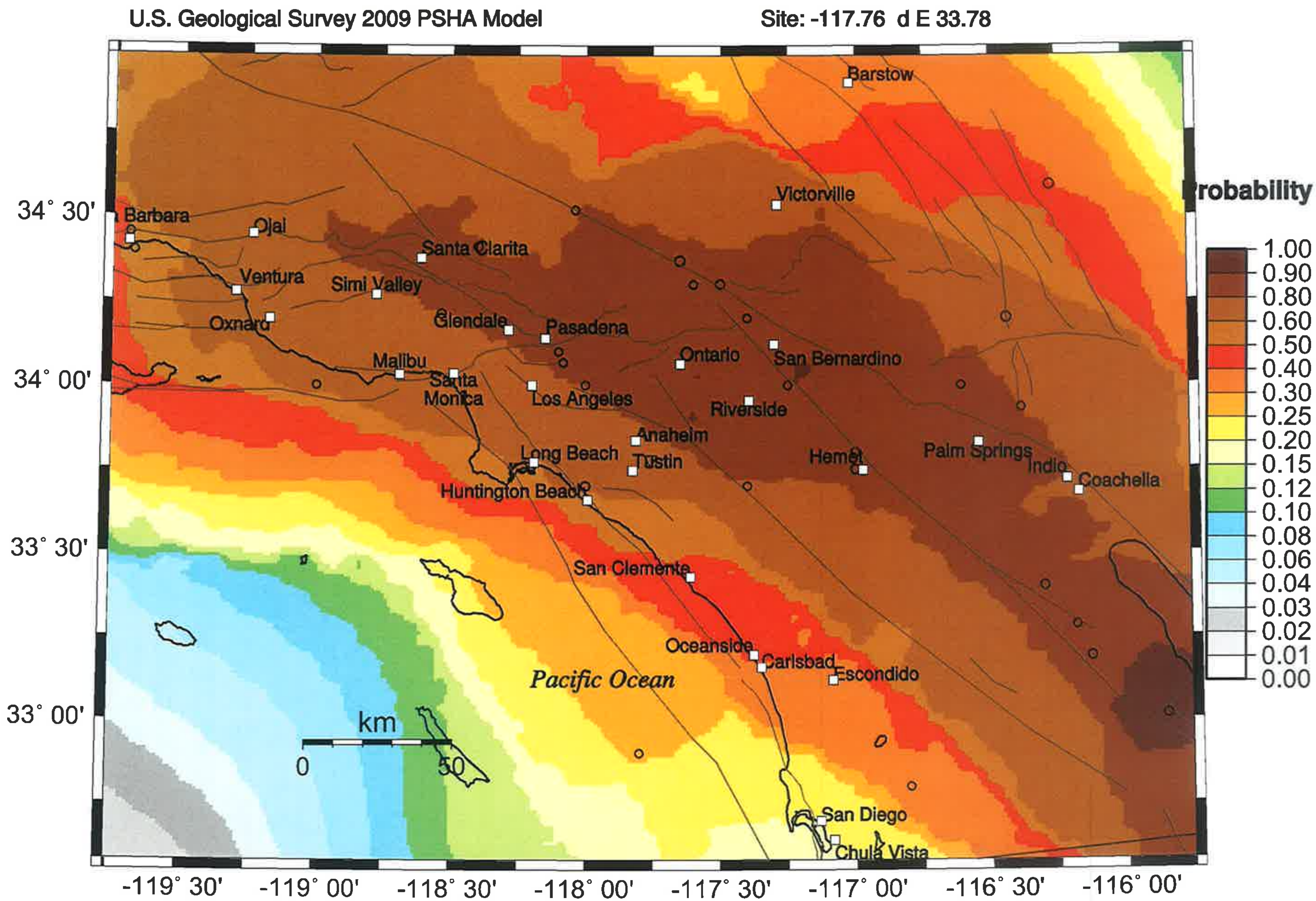
U.S. Geological Survey 2009 PSHA Model

Site: -117.76 d E 33.78



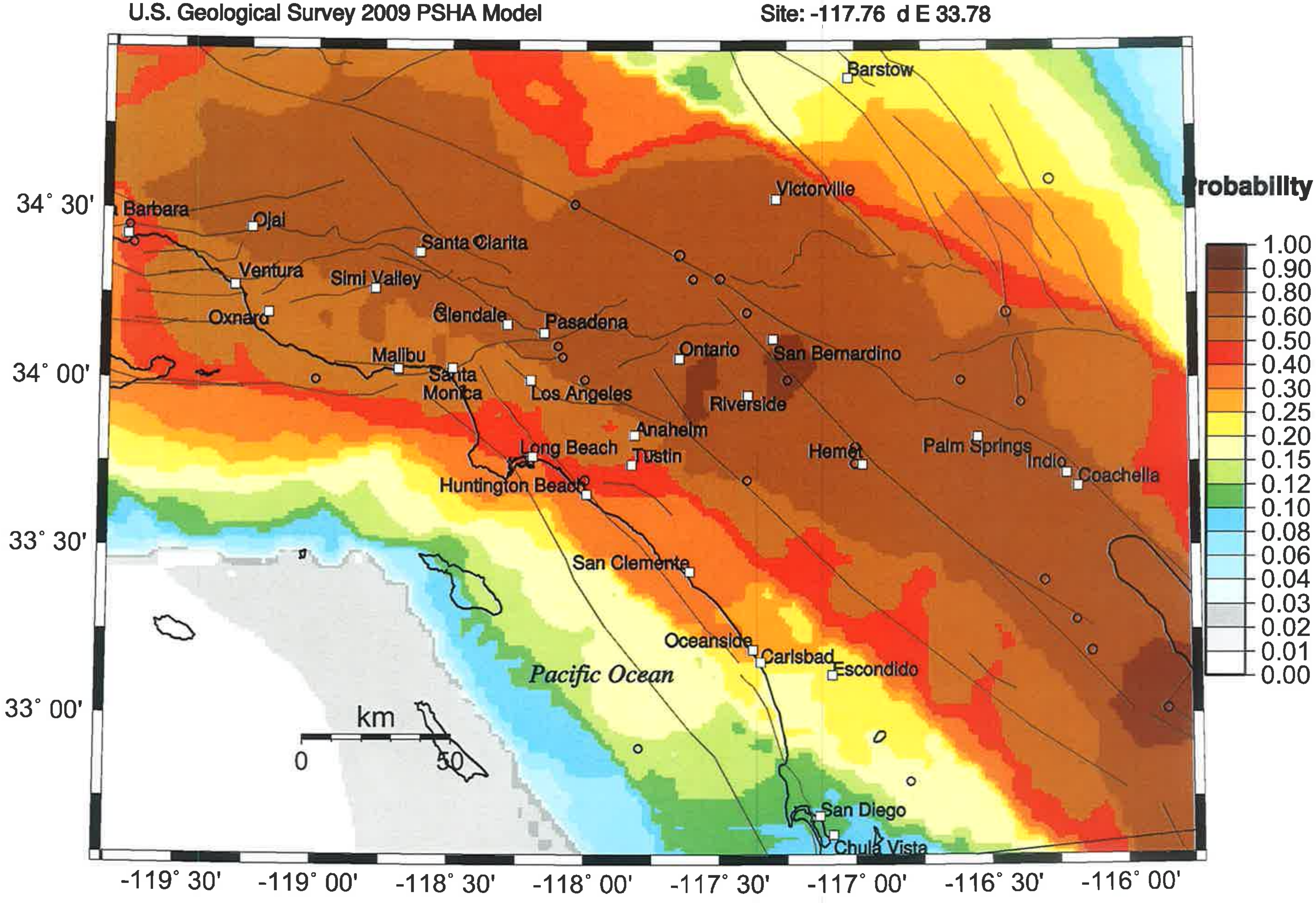
GMT 2014 Feb 20 18:56:45 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Probability of earthquake with M > 6.0 within 50 years & 50 km



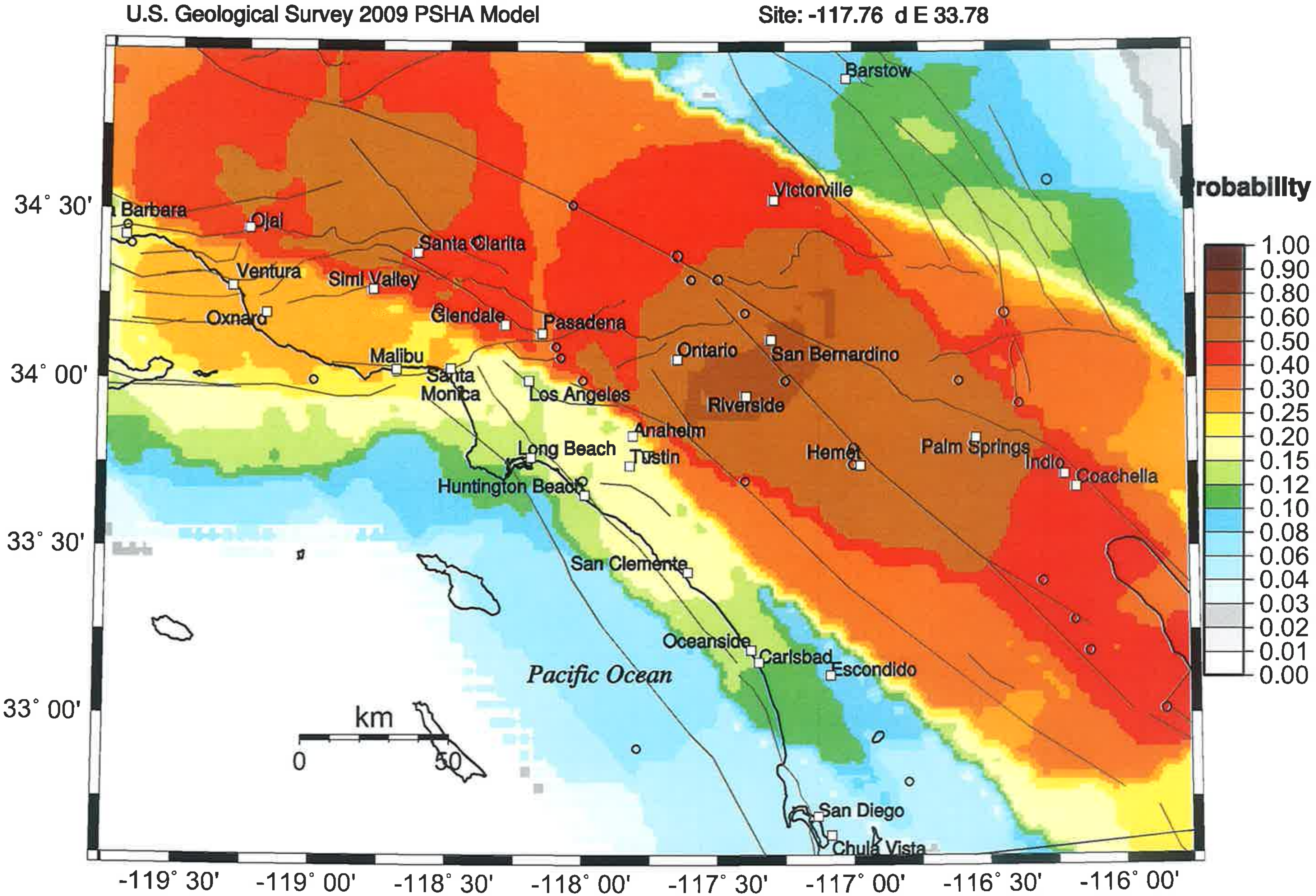
GMT 2014 Feb 20 18:59:17 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of Interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Probability of earthquake with M > 6.5 within 50 years & 50 km



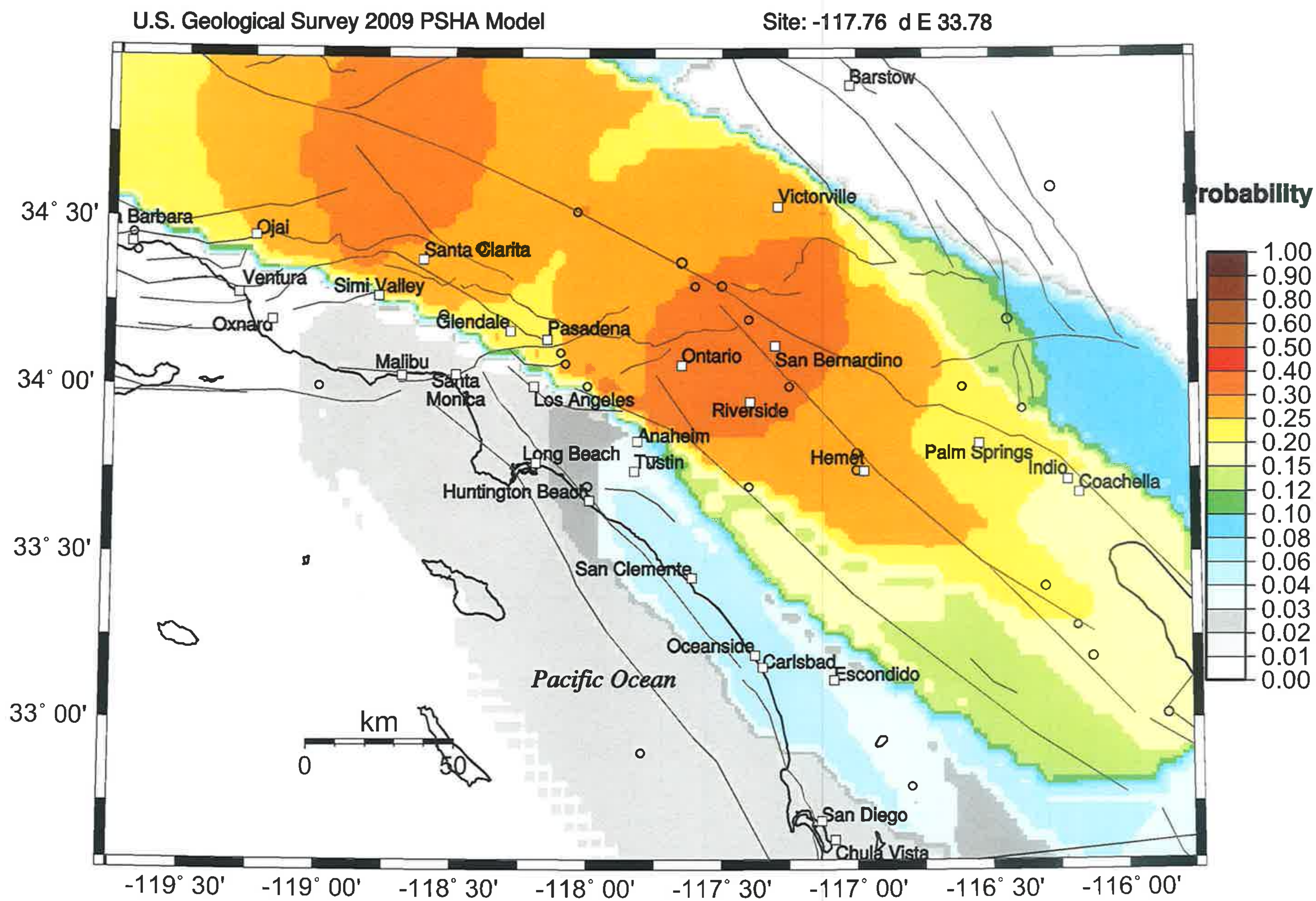
GMT 2014 Feb 20 19:02:22 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of Interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Probability of earthquake with M > 7.0 within 50 years & 50 km



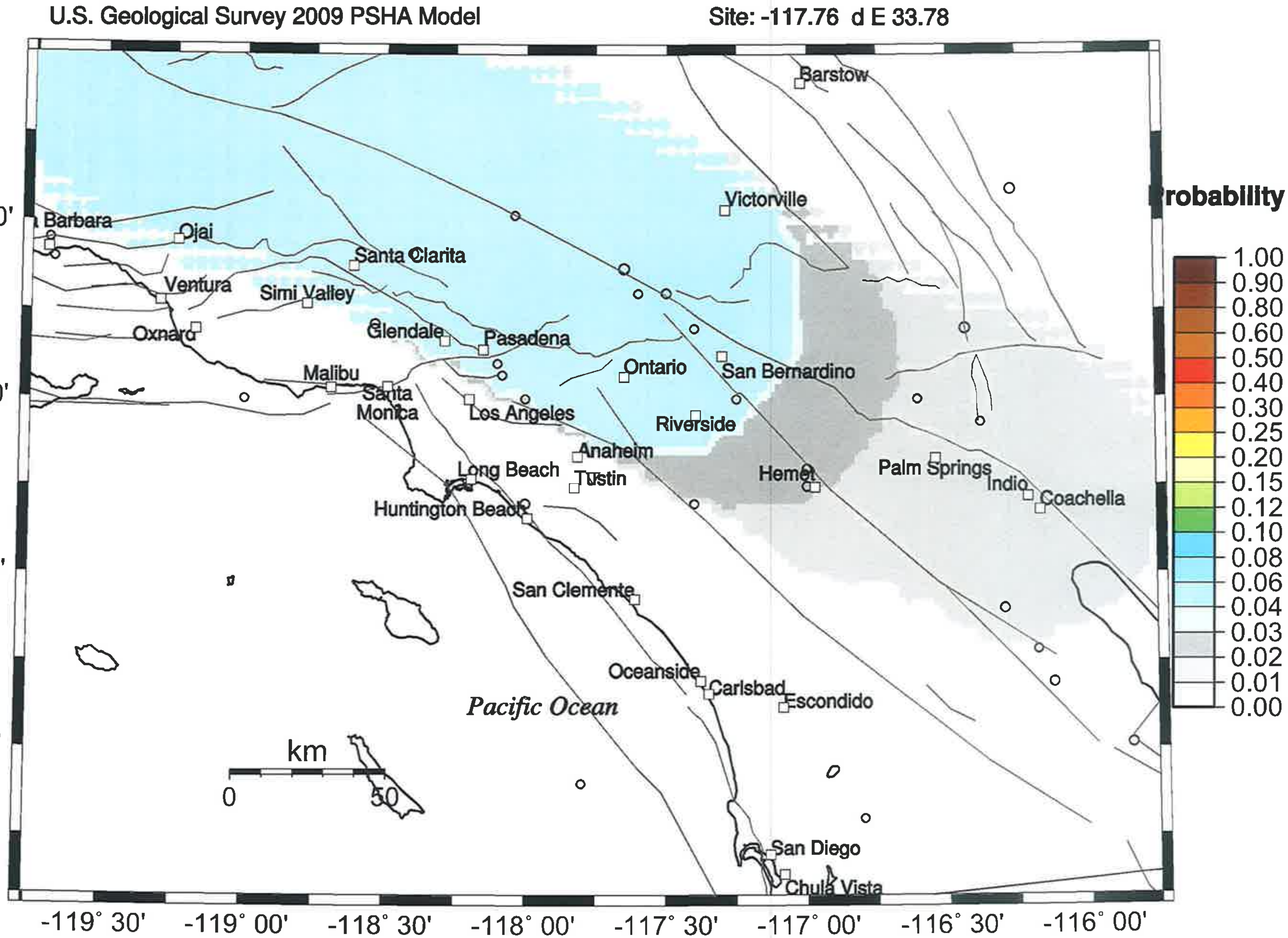
GMT 2014 Feb 20 19:00:54 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of Interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Probability of earthquake with M > 7.5 within 50 years & 50 km



GMT 2014 Feb 20 19:01:38 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Pr [Crustal Earthquake with M > 8.0 within 50 years & 50 km]



GMT 2014 Feb 20 19:03:01 EQ probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of Interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

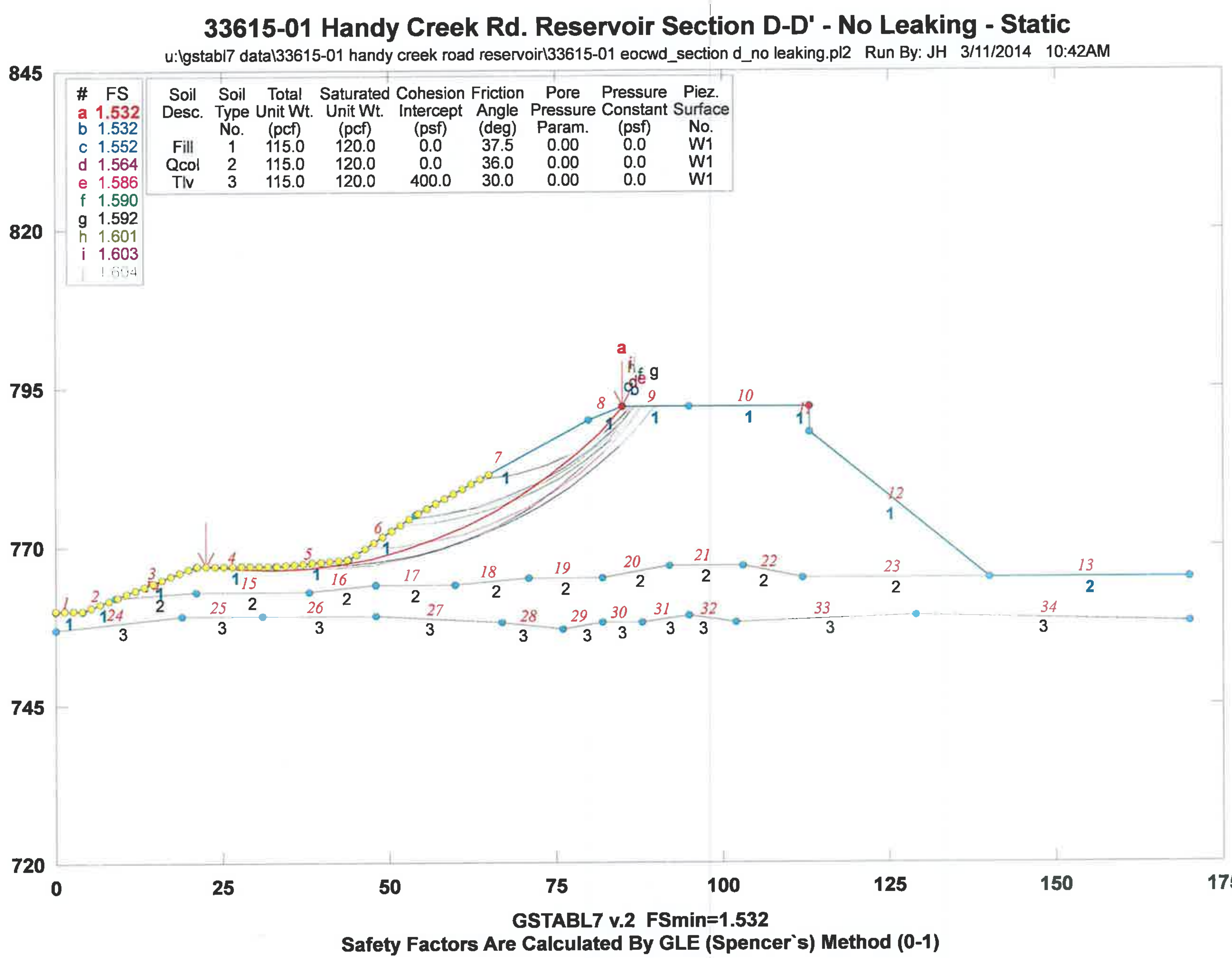
APPENDIX E

Slope Stability Ana

F.N. 33615-01 East Orange County Water District
Peters Canyon Facility
Handy Creek Road Reservoir

Summary of Slope Stability Analyses

Section	Case #	Description	Factor-of-Safety (Static)	Factor-of-Safety (Pseudostatic)
D-D'	1	No leak	1.532	1.105
D-D'	2	Minor leak	1.532	1.105
D-D'	3	Moderate leak	1.448	0.978
D-D'	4	Severe leak	0.822	0.560
E-E'	1	Lower bank - No leak	1.577	1.130
E-E'	2	Lower bank - Minor leak	1.577	1.130
E-E'	3	Lower bank - Moderate leak	0.667	0.429
E-E'	4	Lower bank - Severe leak	0.667	0.429
E-E'	1	Upper bank - No leak	1.642	1.194
E-E'	2	Upper bank - Minor leak	1.642	1.194
E-E'	3	Upper bank - Moderate leak	1.413	1.010
E-E'	4	Upper bank - Severe leak	0.941	0.708


33615-01 Handy Creek Rd. Reservoir Section D-D' - No Leaking - Static
u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_no leaking.pl2 Run By: JH 3/11/2014 10:42AM

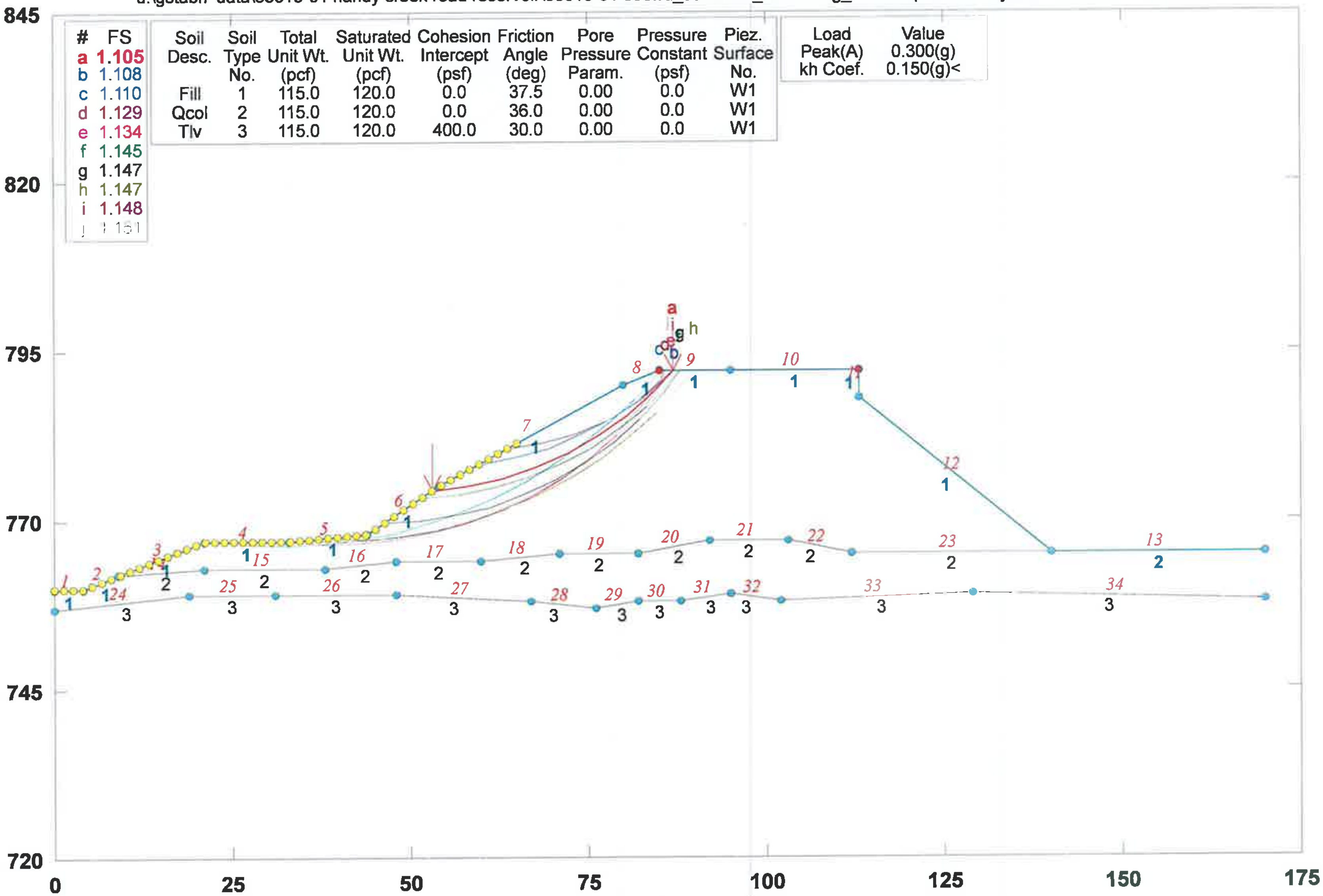
#	FS
a	1.532
b	1.532
c	1.552
d	1.564
e	1.586
f	1.590
g	1.592
h	1.601
i	1.603
j	1.604

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	115.0	120.0	0.0	37.5	0.00	0.0	W1
Qcol	2	115.0	120.0	0.0	36.0	0.00	0.0	W1
Tlv	3	115.0	120.0	400.0	30.0	0.00	0.0	W1

845
820
795
770
745
720
0
25
50
75
100
125
150
175
GSTABL7 v.2 FSmin=1.532
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - No Leaking - Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_no leaking_seismic.pl2 Run By: JH 3/11/2014 10:46AM

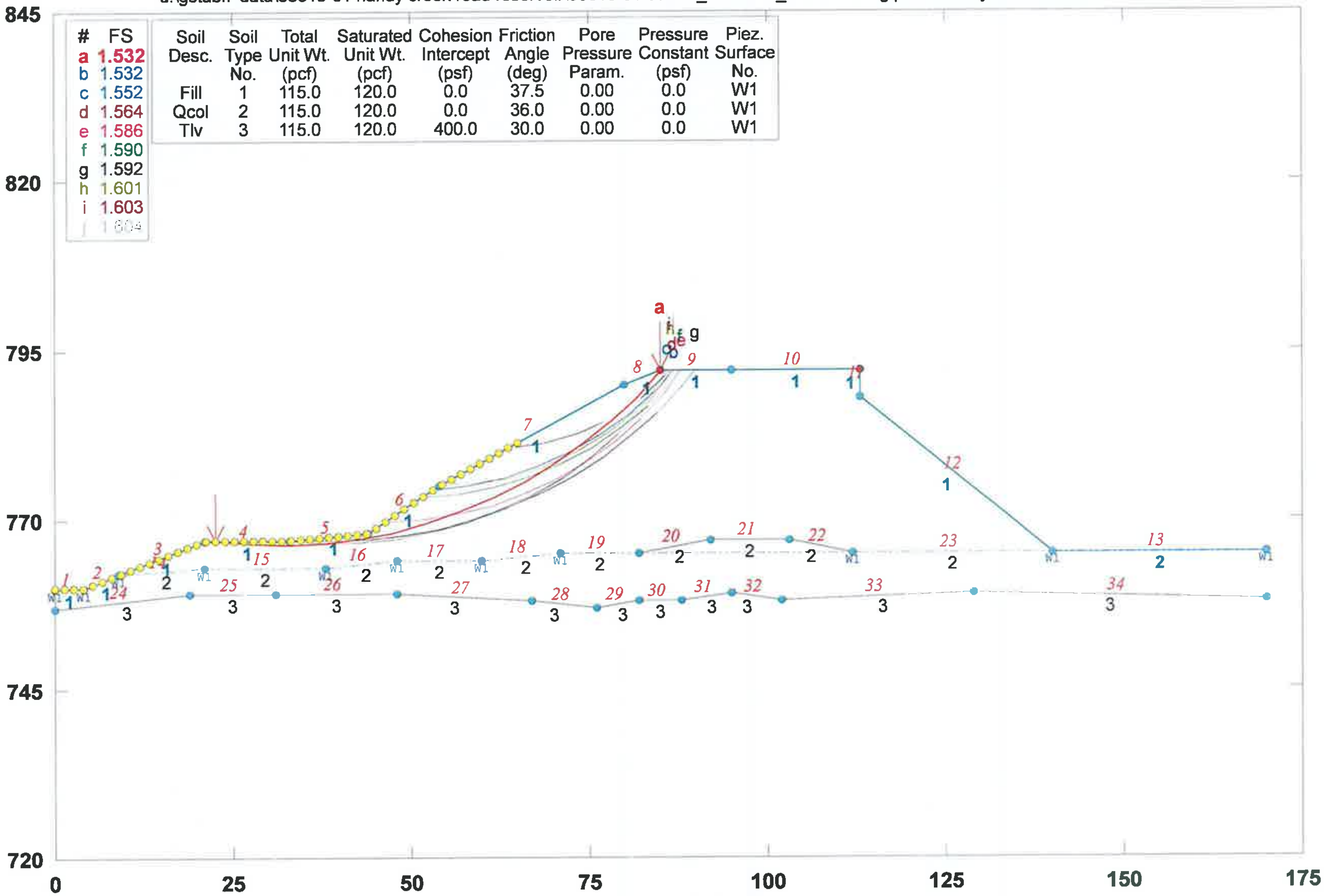


GSTABL7 v.2 FSmin=1.105

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Minor Leaking - Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_minor leaking.pl2 Run By: JH 3/11/2014 10:46AM

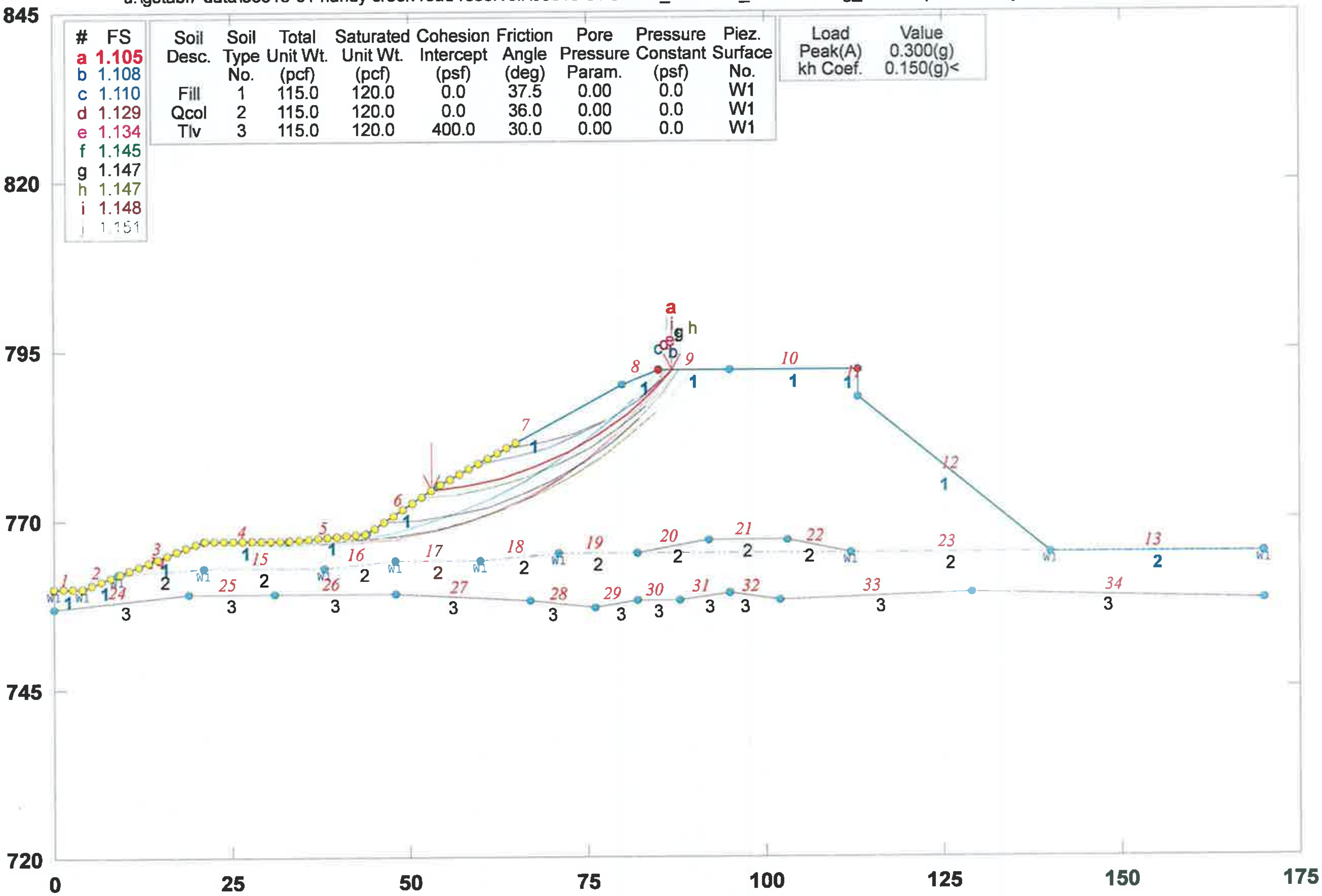


GSTABL7 v.2 FSmin=1.532

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Minor Leaking - Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_minor leaking_seismic.pl2 Run By: JH 3/11/2014 10:47AM

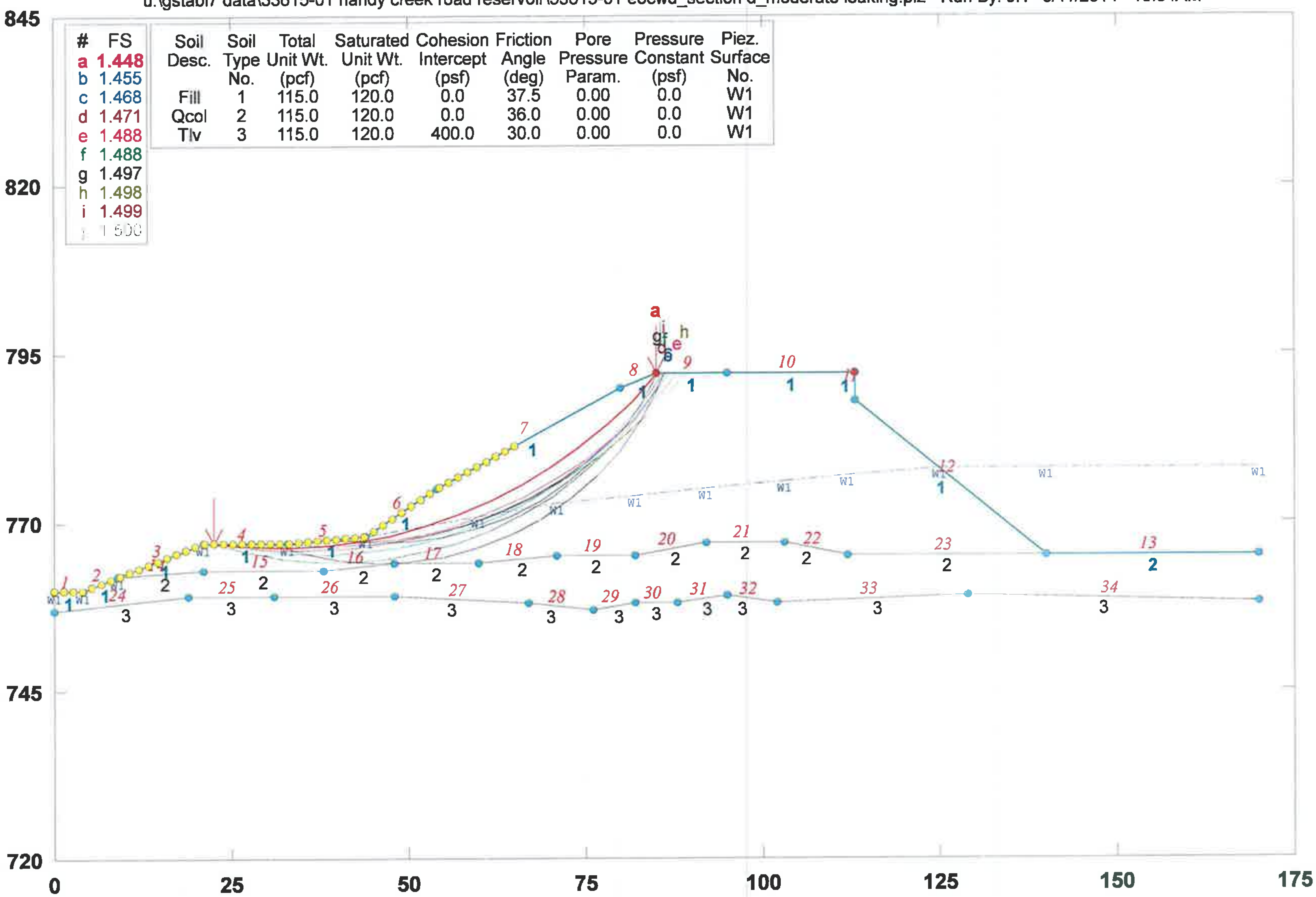


GSTABL7 v.2 FSmin=1.105

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Moderate Leaking - Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_moderate leaking.pl2 Run By: JH 3/11/2014 10:54AM



GSTABL7 v.2 FSmin=1.448

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Moderate Leaking-Seismic

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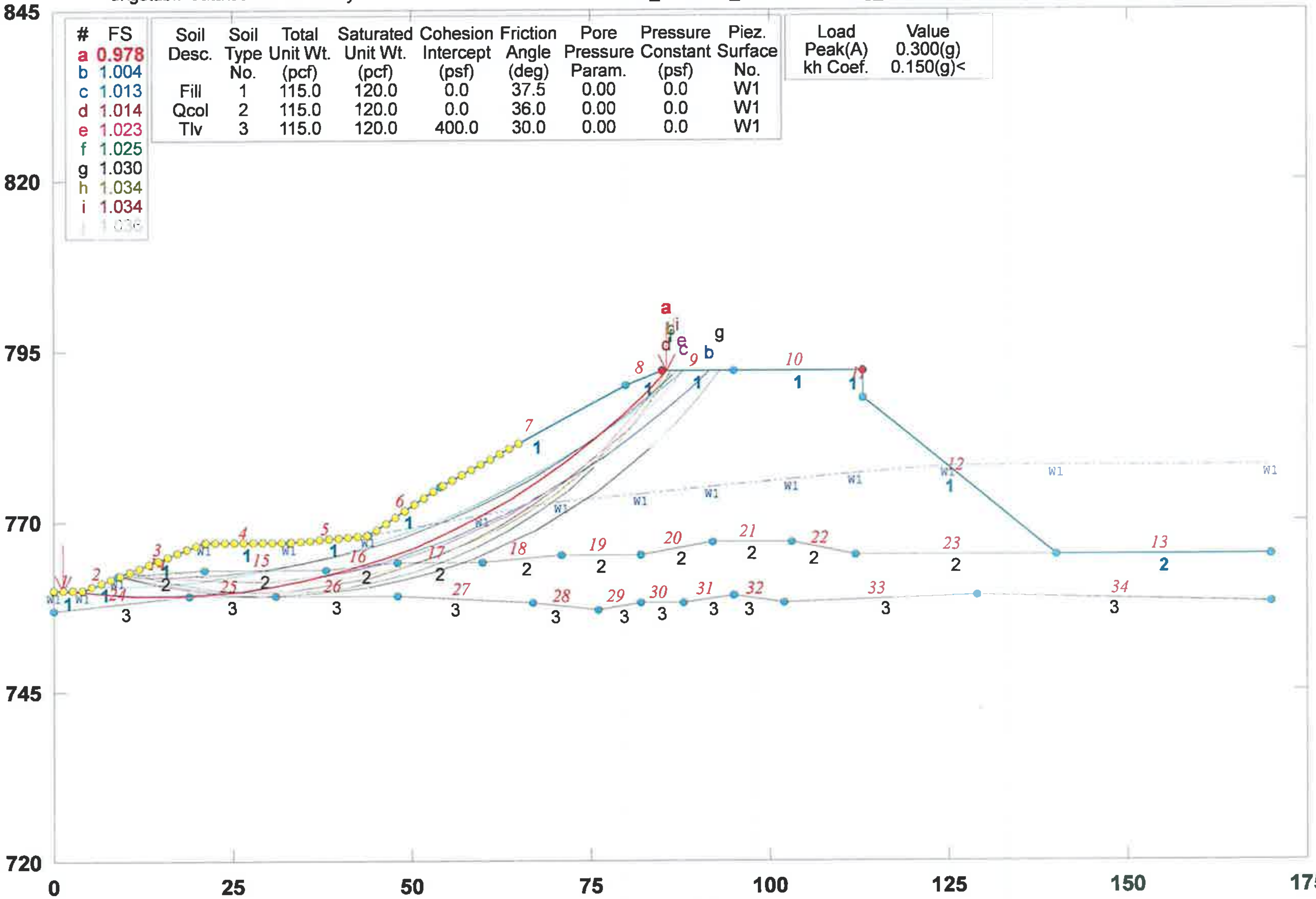
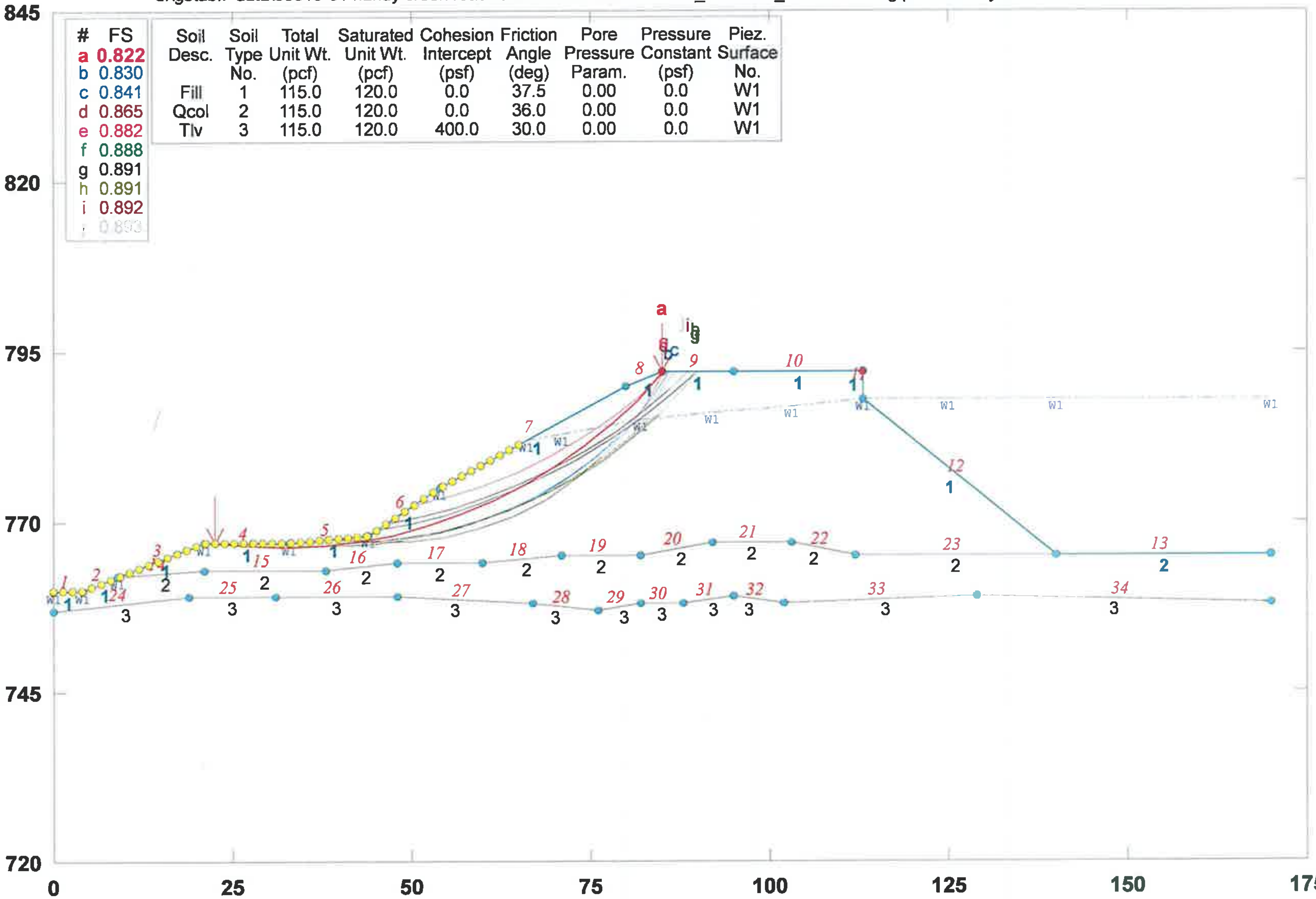


GSTABL7 v.2 FSmin=0.978

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Severe Leaking - Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_severe leaking.pl2 Run By: JH 3/11/2014 10:55AM

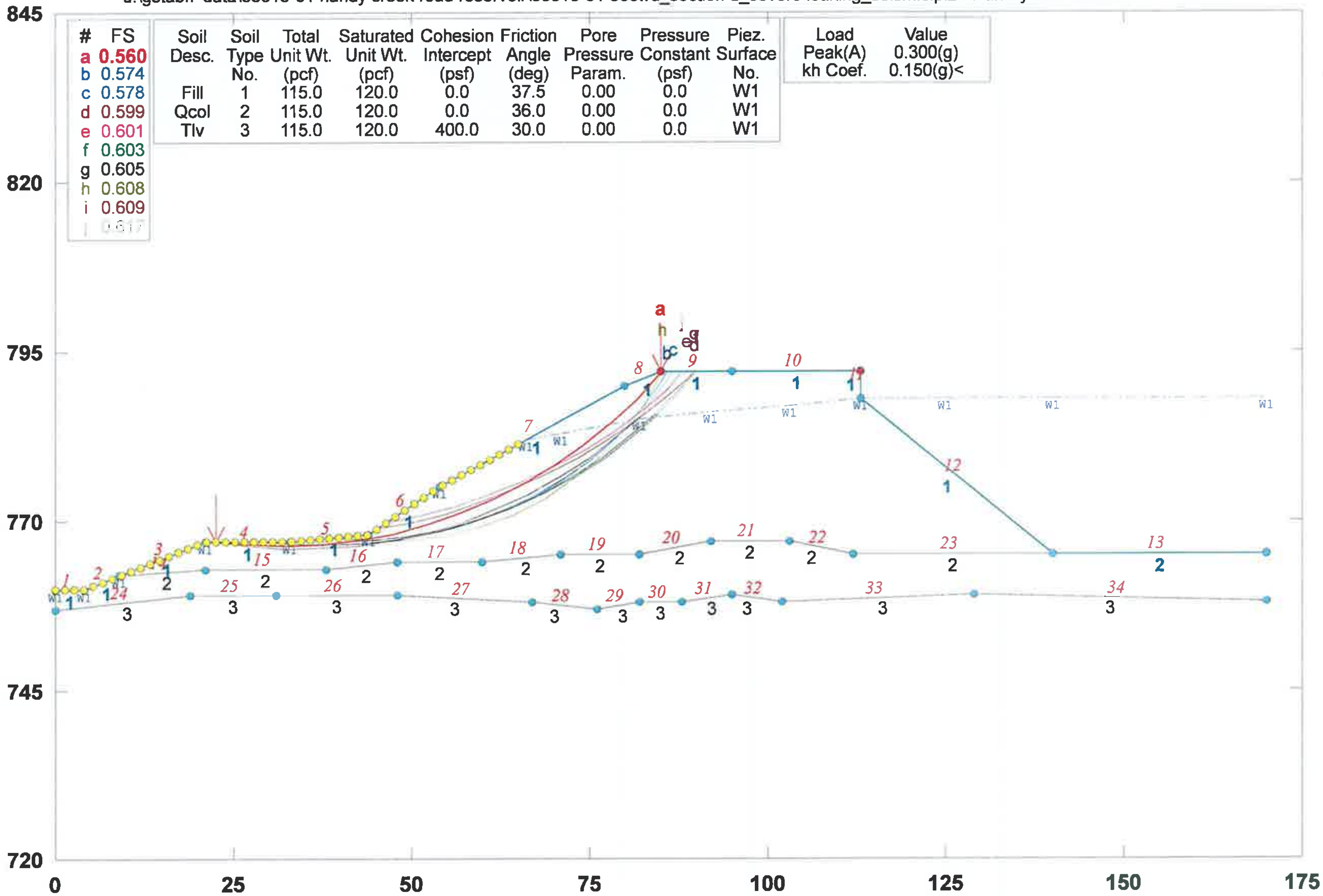


GSTABL7 v.2 FSmin=0.822

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section D-D' - Severe Leaking - Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section d_severe leaking_seismic.pl2 Run By: JH 3/11/2014 10:56AM

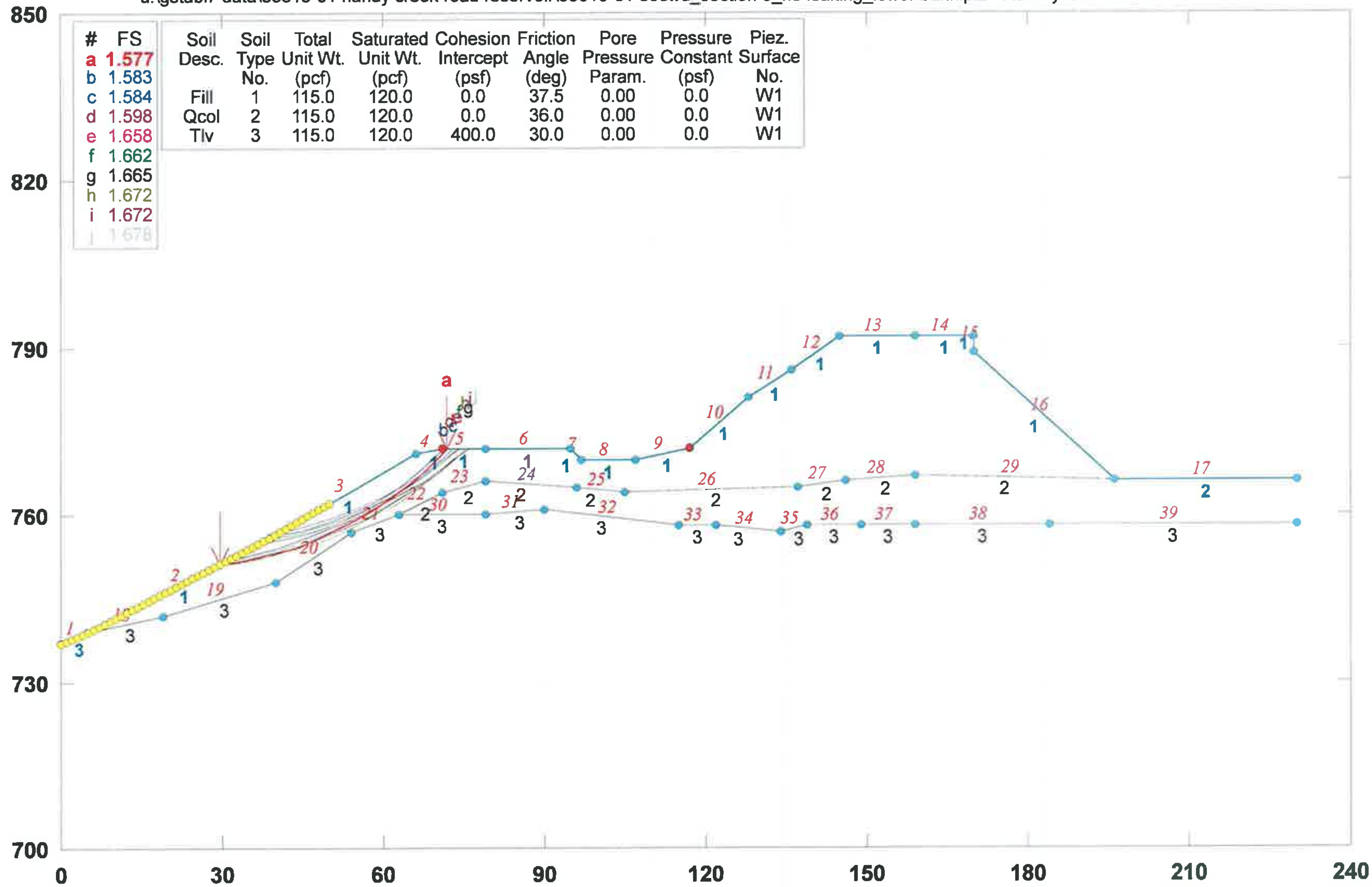


GSTABL7 v.2 FSmin=0.560

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-No Leak_Lower_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_no leaking_lower bank.pl2 Run By: JH 3/11/2014 11:03AM

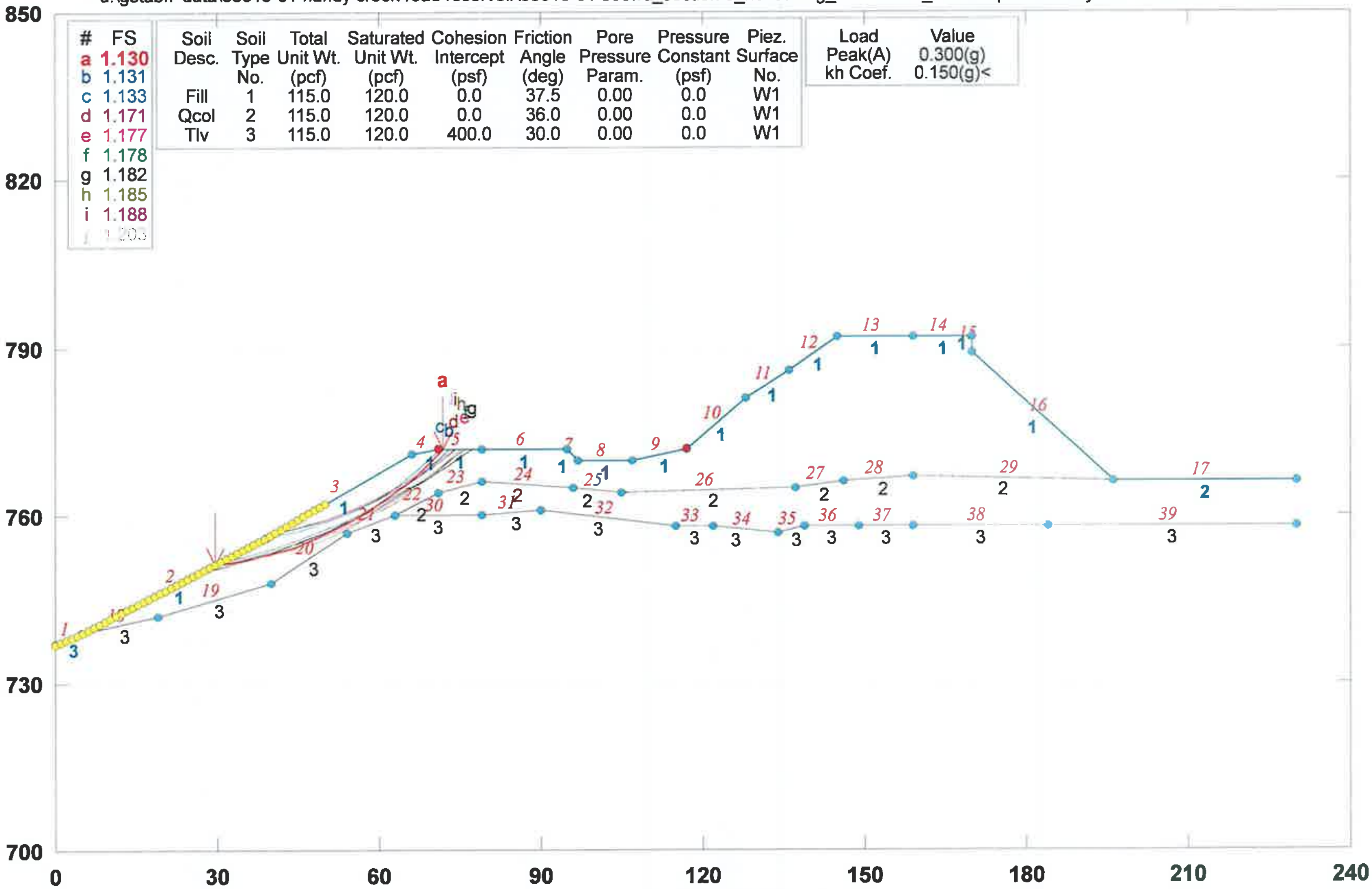


GSTABL7 v.2 FSmin=1.577

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-No Leak_Lower_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_no leaking_lower bank_seismic.pl2 Run By: JH 3/11/2014 11:04AM

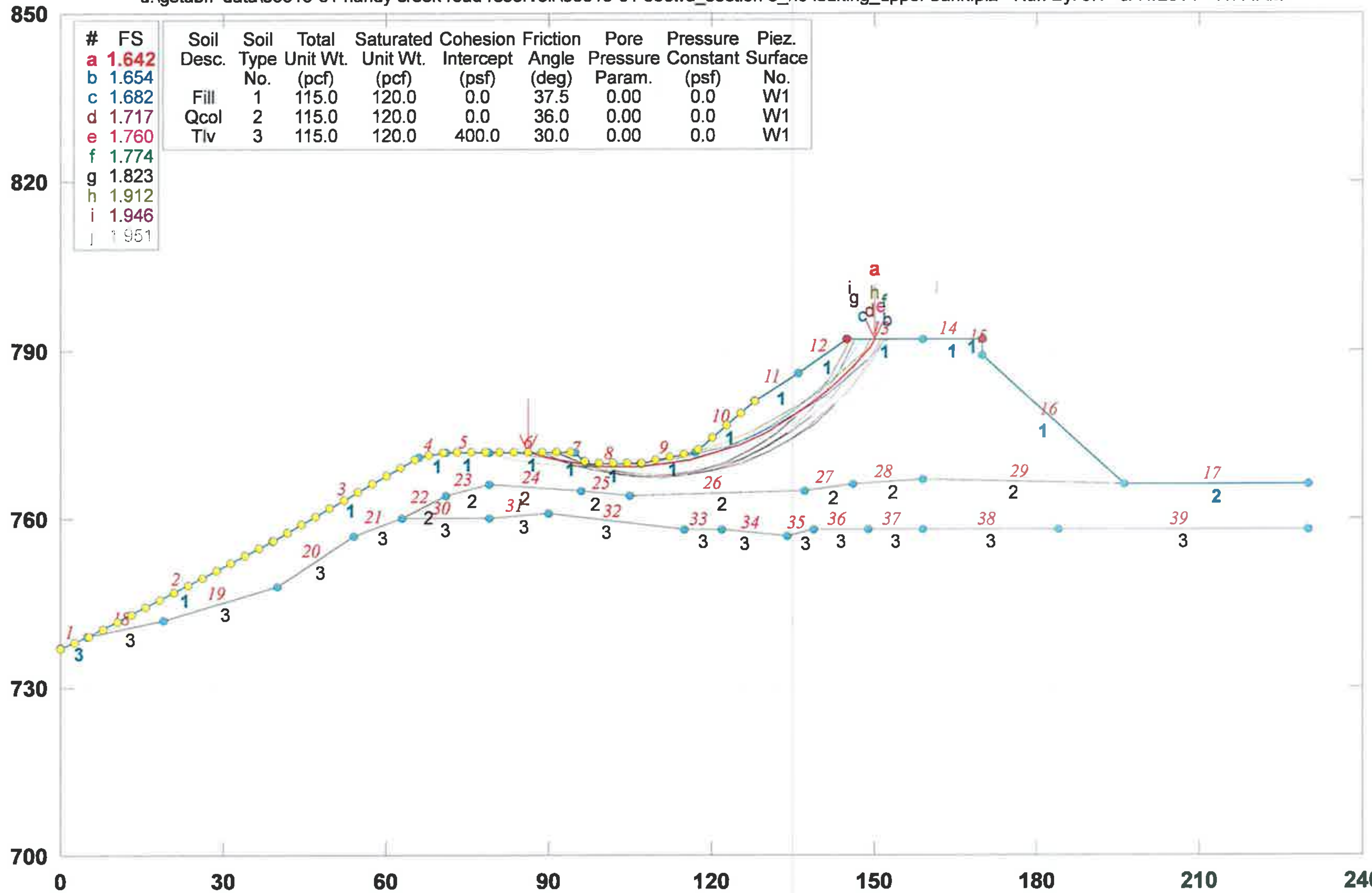


GSTABL7 v.2 FSmin=1.130

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E' - No Leaking_Upper_Static

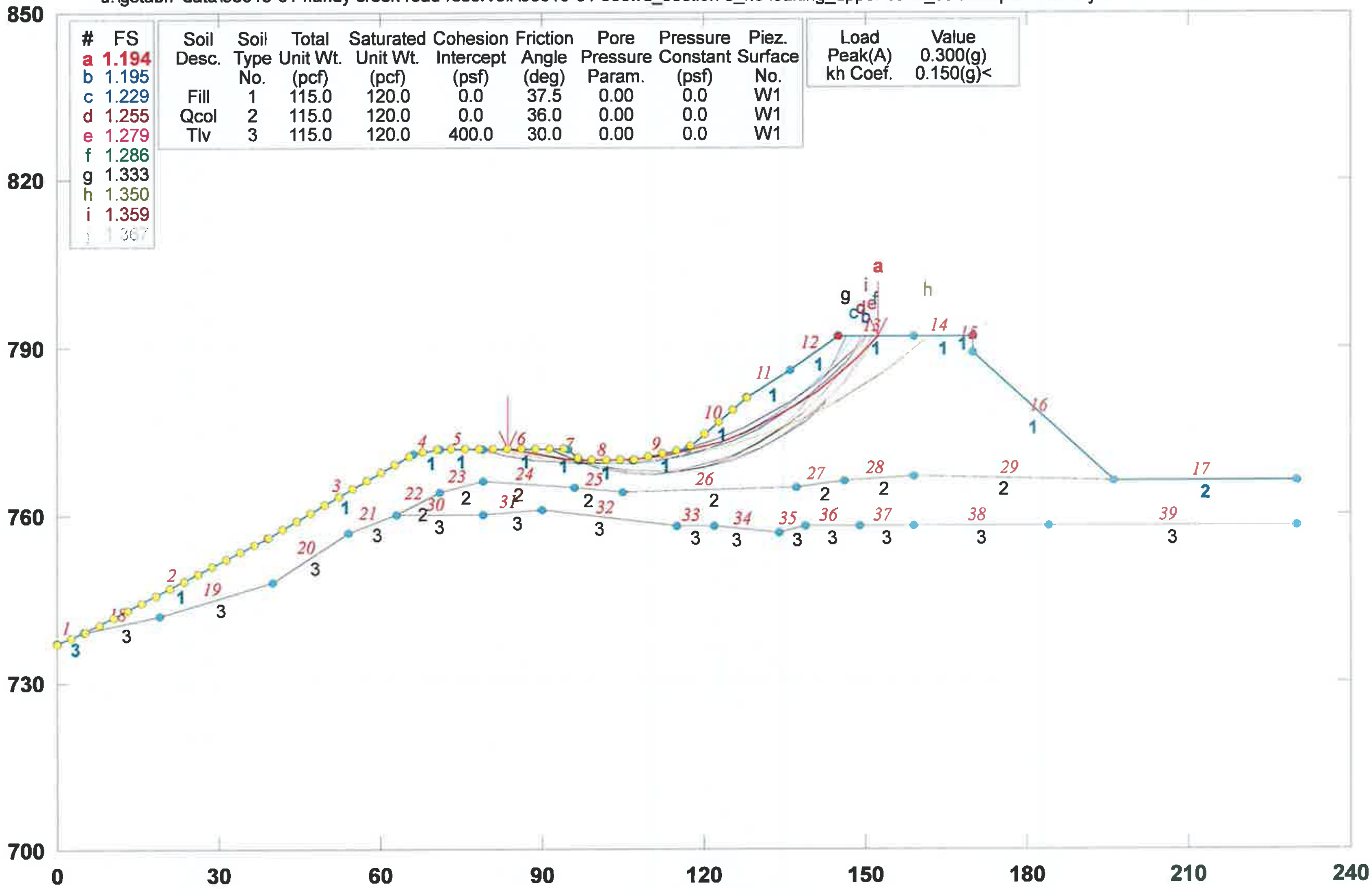
u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_no leaking_upper bank.pl2 Run By: JH 3/11/2014 11:44AM



GSTABL7 v.2 FSmin=1.642
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E' - No Leaking_Upper_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_no leaking_upper bank_seismic.pl2 Run By: JH 3/11/2014 11:44AM

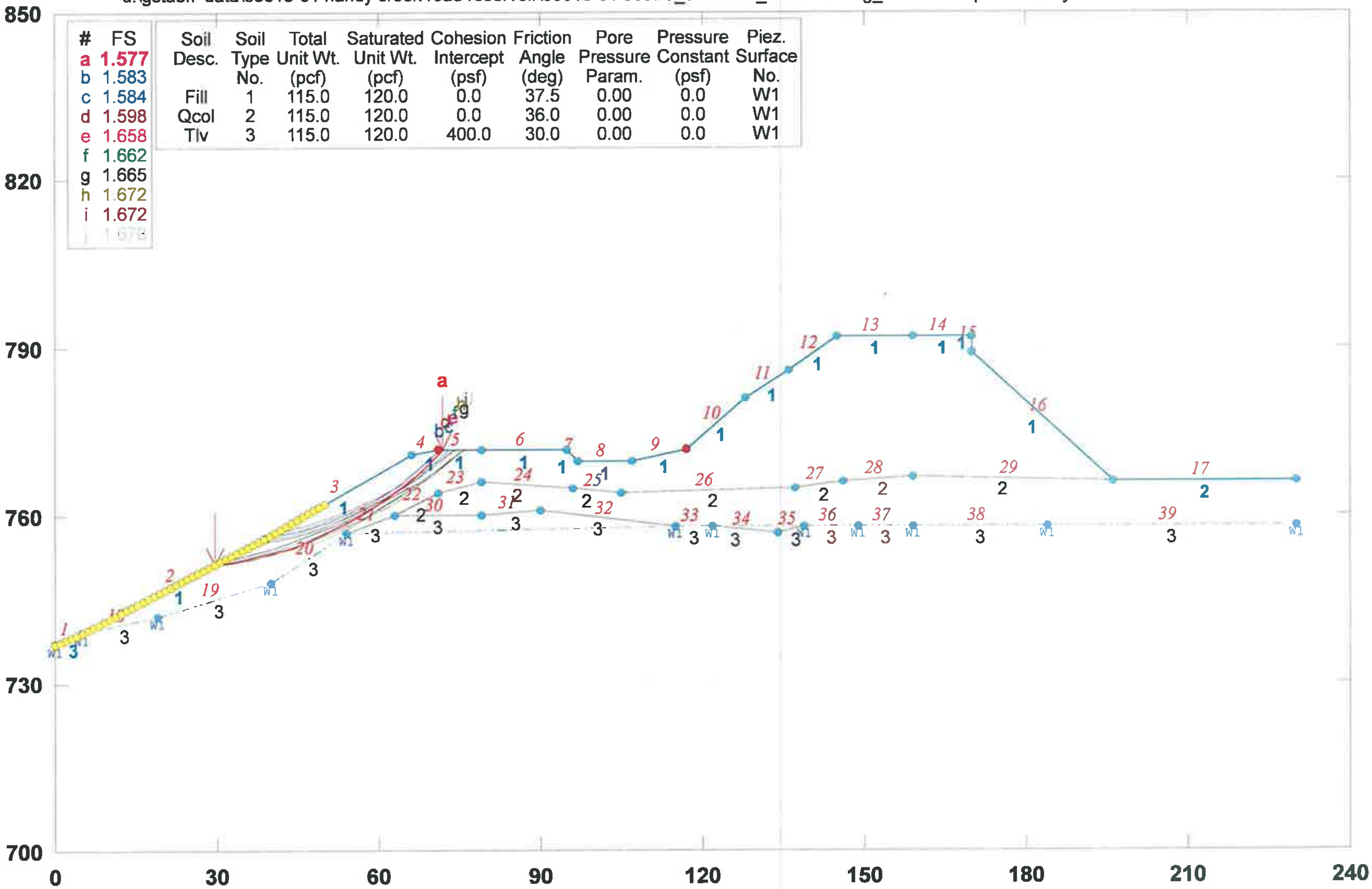


GSTABL7 v.2 FSmin=1.194

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Minor Leaking_Lower_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_minor leaking_lower bank.pl2 Run By: JH 3/11/2014 11:19AM



#	FS
a	1.577
b	1.583
c	1.584
d	1.598
e	1.658
f	1.662
g	1.665
h	1.672
i	1.672

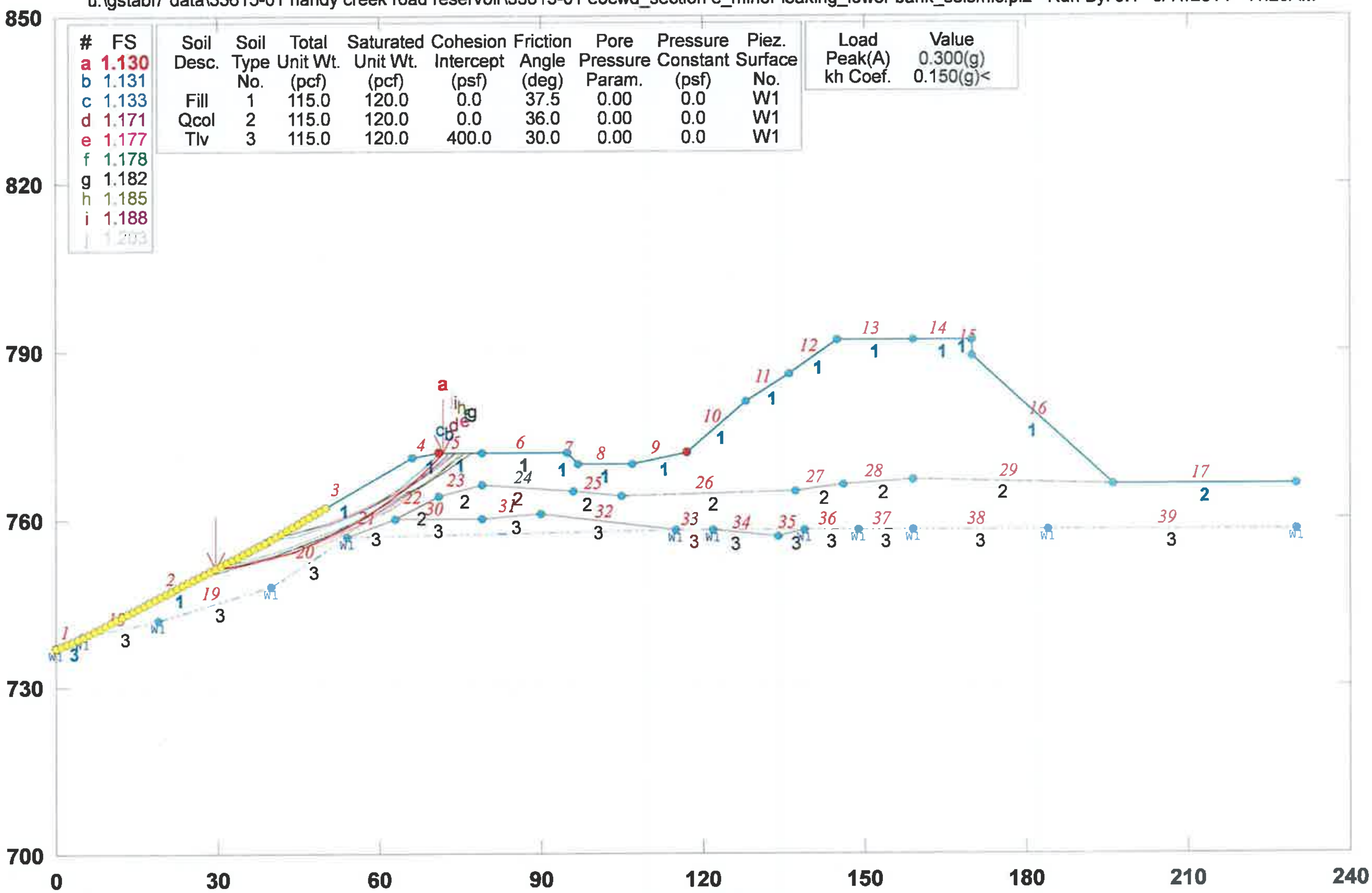
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	115.0	120.0	0.0	37.5	0.00	0.0	W1
Qcol	2	115.0	120.0	0.0	36.0	0.00	0.0	W1
Tlv	3	115.0	120.0	400.0	30.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.577

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Minor Leaking_Lower_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_minor leaking_lower bank_seismic.pl2 Run By: JH 3/11/2014 11:20AM

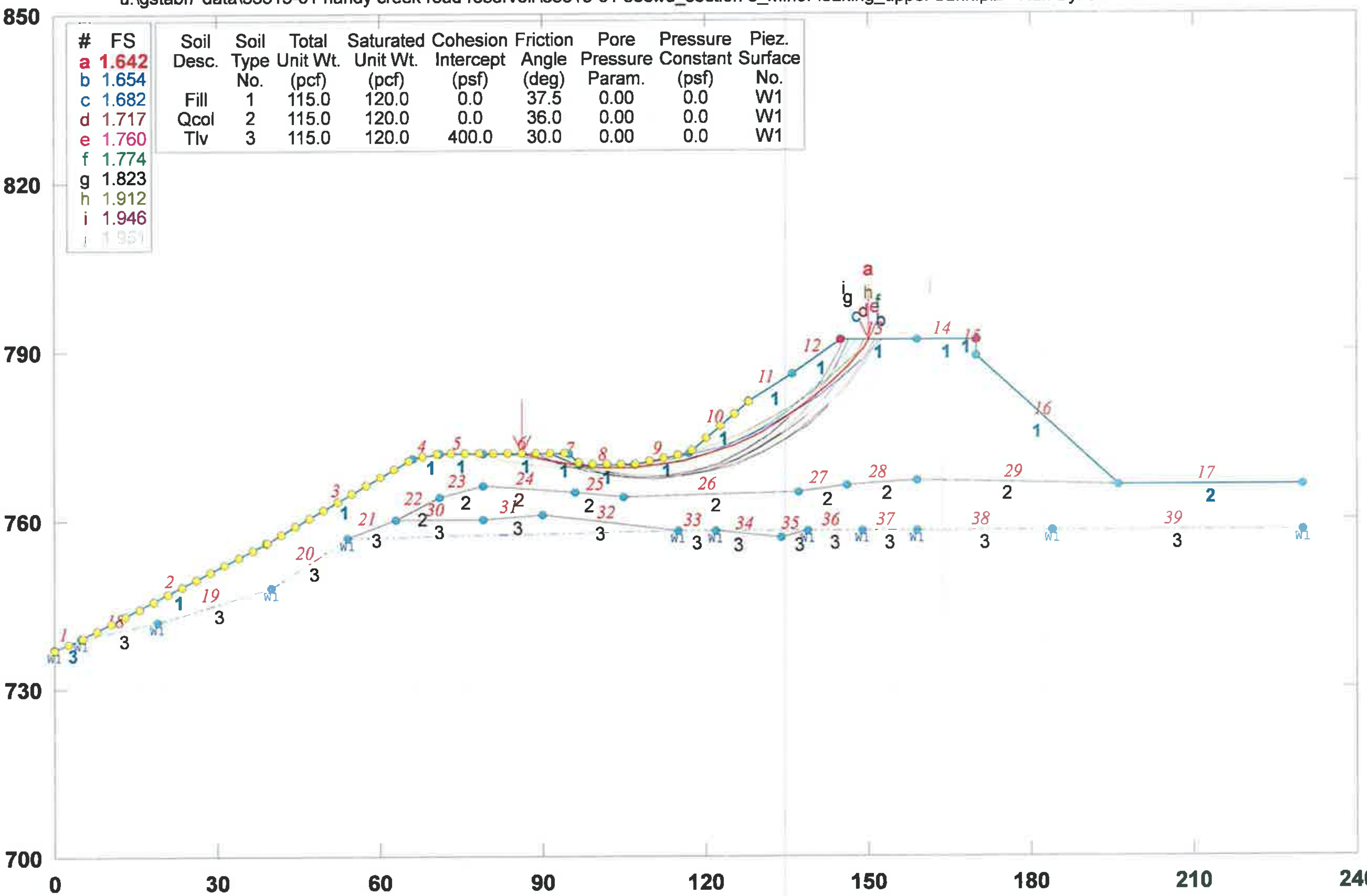


GSTABL7 v.2 FSmin=1.130

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Minor Leaking_Upper_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_minor leaking_upper bank.pl2 Run By: JH 3/11/2014 11:17AM

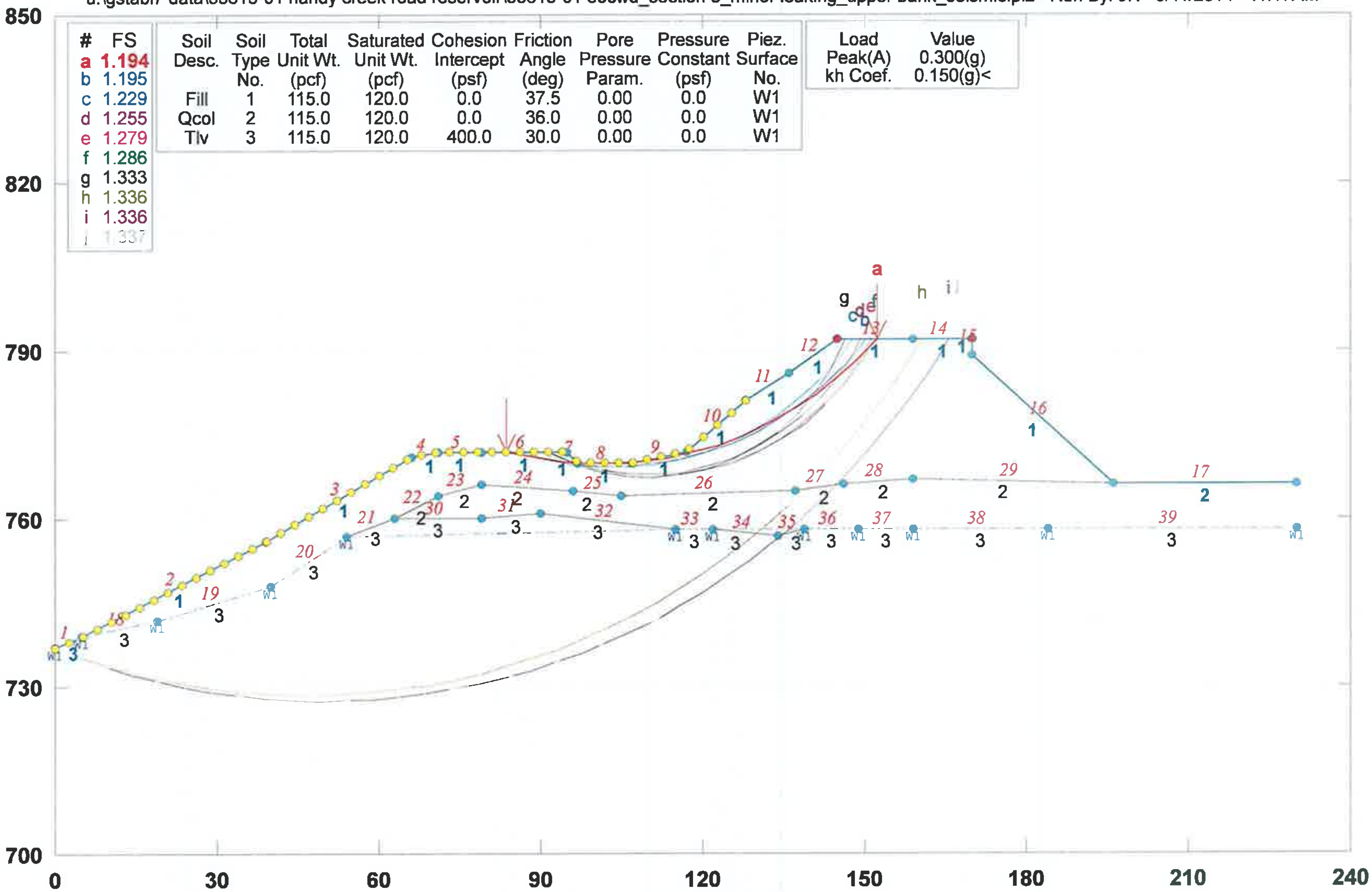


GSTABL7 v.2 FSmin=1.642

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Minor Leaking_Upper_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_minor leaking_upper bank_seismic.pl2 Run By: JH 3/11/2014 11:17AM



#	FS
a	1.194
b	1.195
c	1.229
d	1.255
e	1.279
f	1.286
g	1.333
h	1.336
i	1.336
j	1.337

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	115.0	120.0	0.0	37.5	0.00	0.0	W1
Qcol	2	115.0	120.0	0.0	36.0	0.00	0.0	W1
Tlv	3	115.0	120.0	400.0	30.0	0.00	0.0	W1

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.194

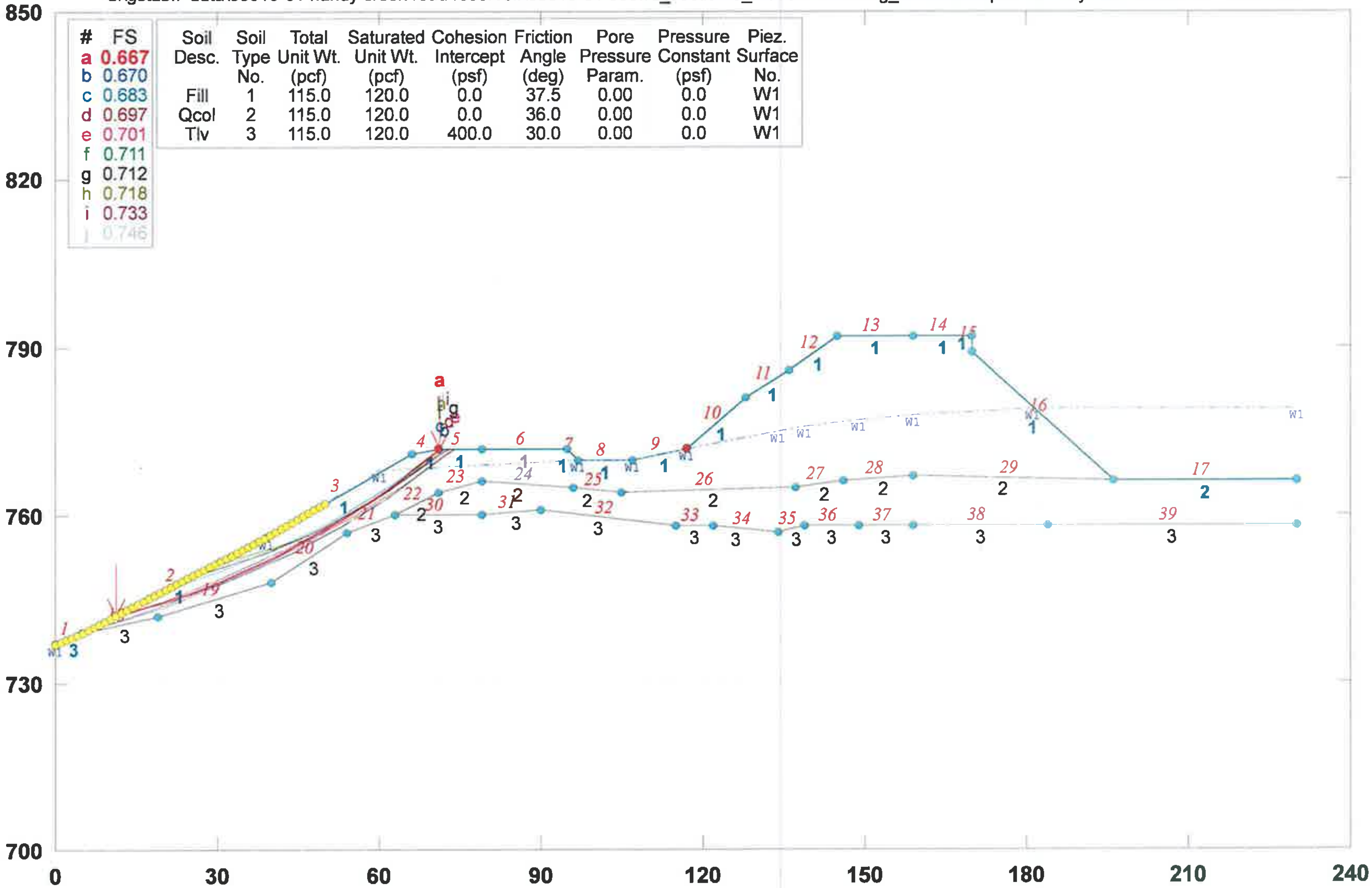
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Moderate Leak_Low_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_moderate leaking_lower bank.pl2 Run By: JH 3/11/2014 11:06AM

#	FS
a	0.667
b	0.670
c	0.683
d	0.697
e	0.701
f	0.711
g	0.712
h	0.718
i	0.733
j	0.746

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	115.0	120.0	0.0	37.5	0.00	0.0	W1
Qcol	2	115.0	120.0	0.0	36.0	0.00	0.0	W1
Tlv	3	115.0	120.0	400.0	30.0	0.00	0.0	W1

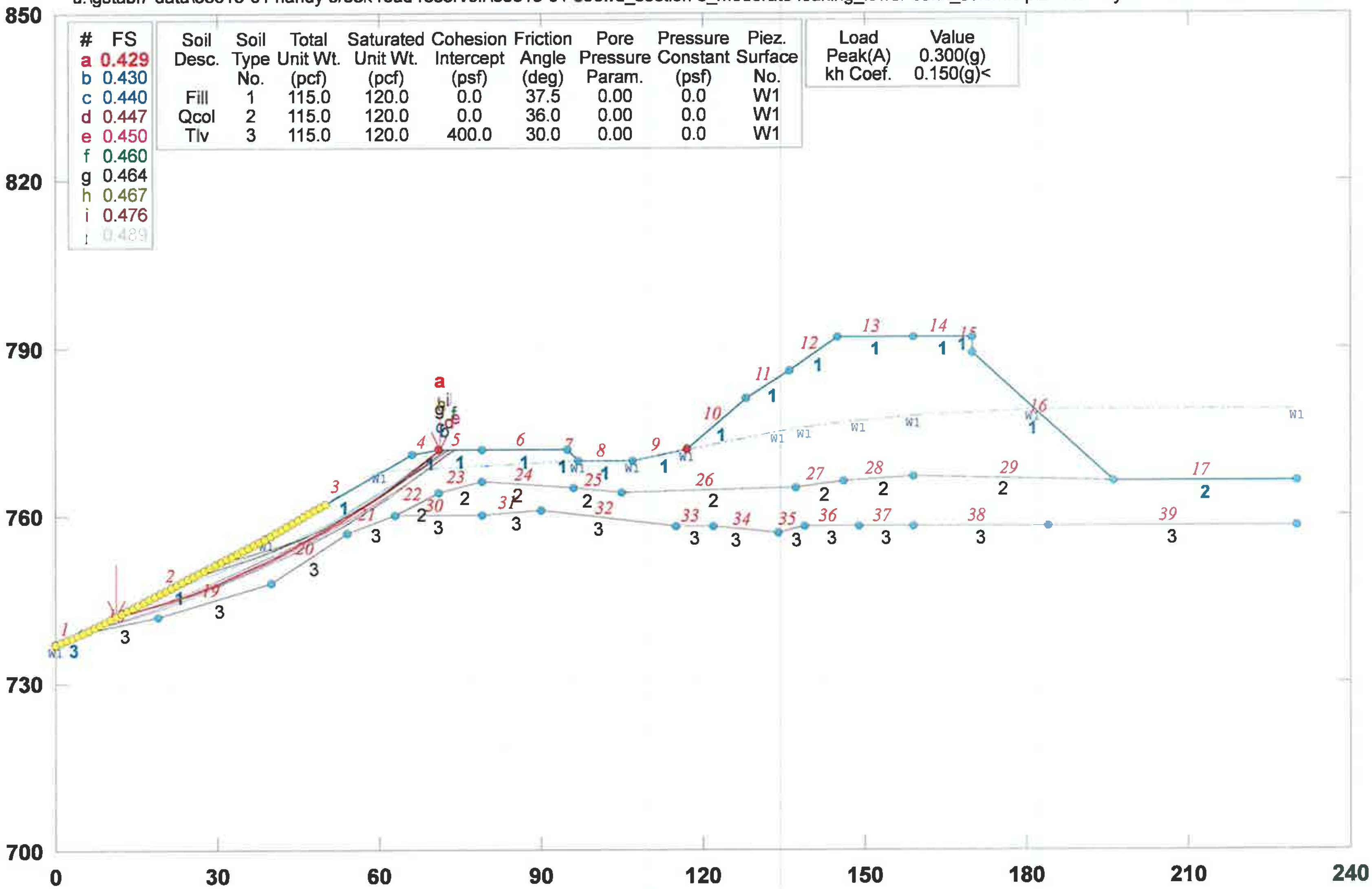


GSTABL7 v.2 FSmin=0.667

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Moderate Leak_Low_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_moderate leaking_lower bank_seismic.pl2 Run By: JH 3/11/2014 11:07AM

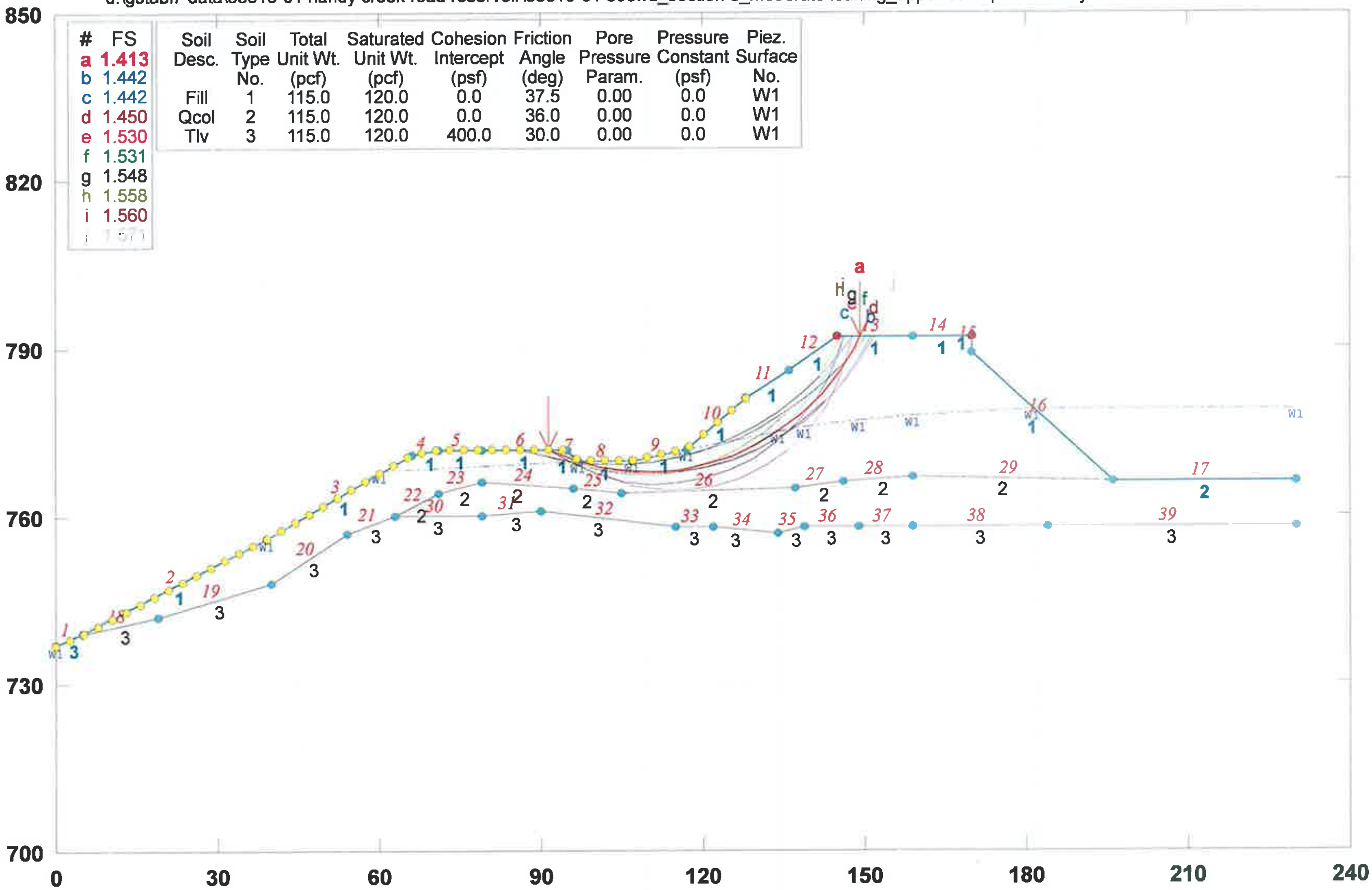


GSTABL7 v.2 FSmin=0.429

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Moderate Leak_Upper_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_moderate leaking_upper bank.pl2 Run By: JH 3/11/2014 11:24AM

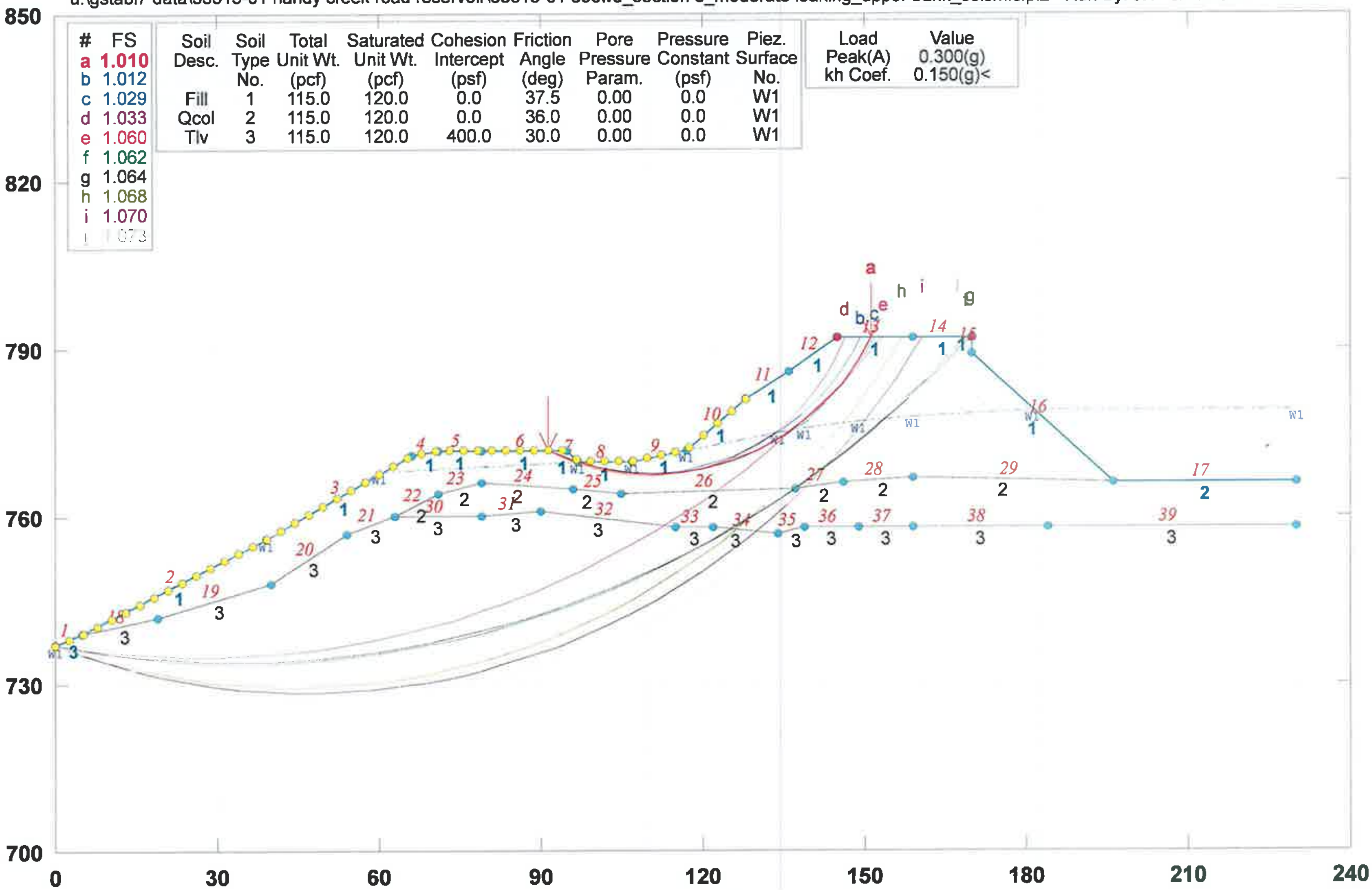


GSTABL7 v.2 FSmin=1.413

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E'-Moderate Leak_Upper_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_moderate leaking_upper bank_seismic.pl2 Run By: JH 3/11/2014 11:25AM

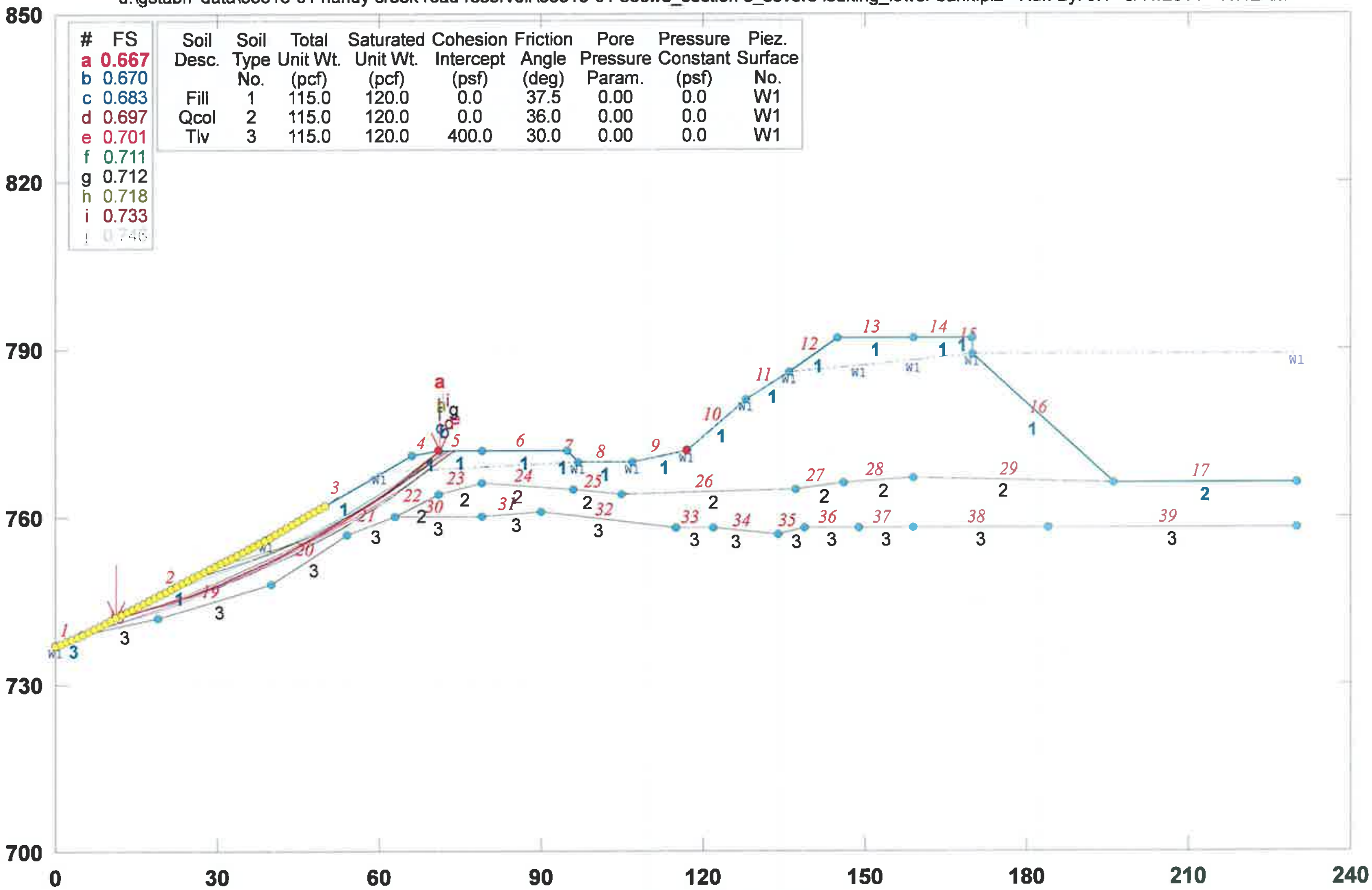


GSTABL7 v.2 FSmin=1.010

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E' - Severe Leaking - Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_severe leaking_lower bank.pl2 Run By: JH 3/11/2014 11:12AM

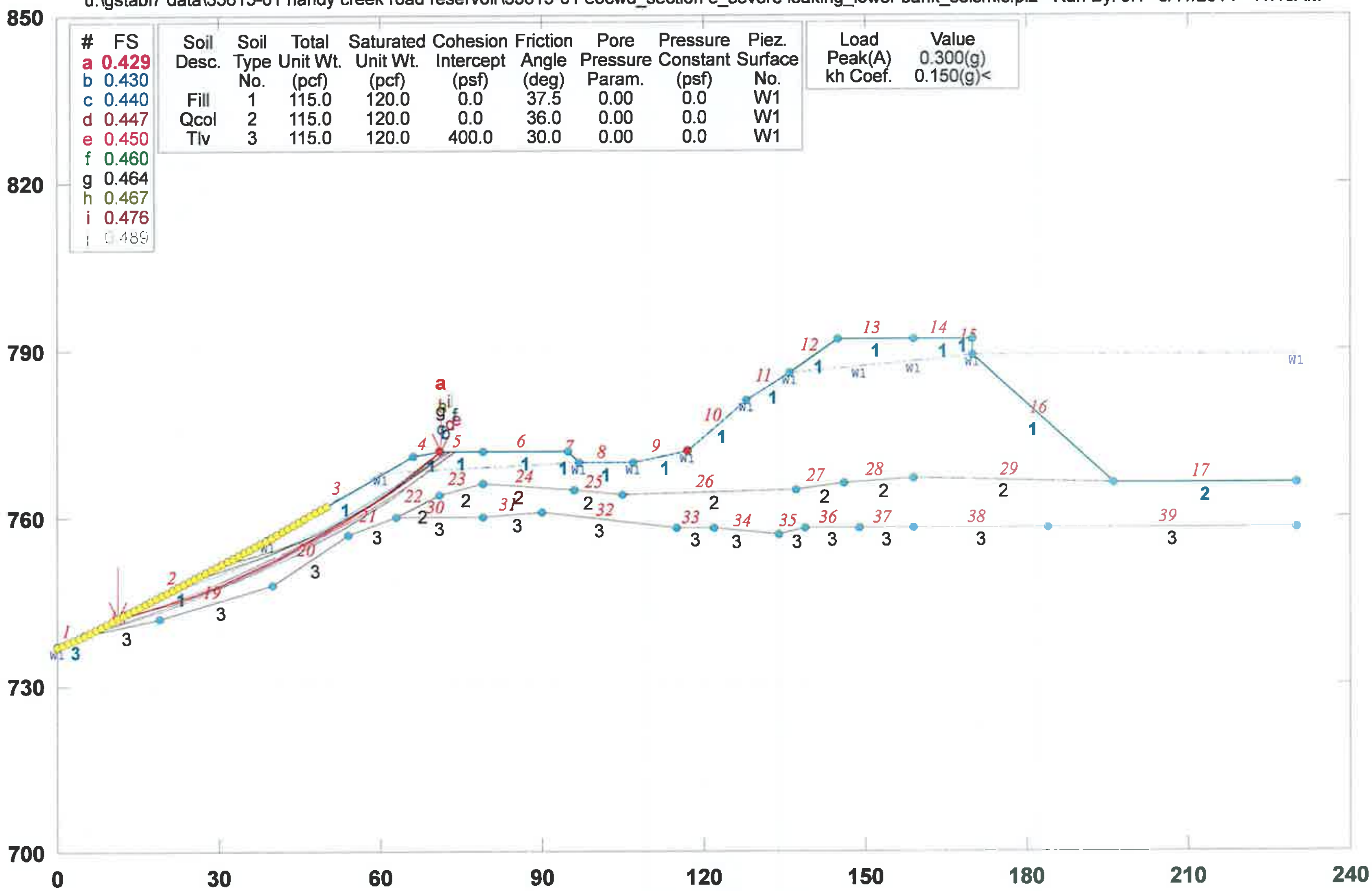


GSTABL7 v.2 FSmin=0.667

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E' - Severe Leaking - Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_severe leaking_lower bank_seismic.pl2 Run By: JH 3/11/2014 11:13AM

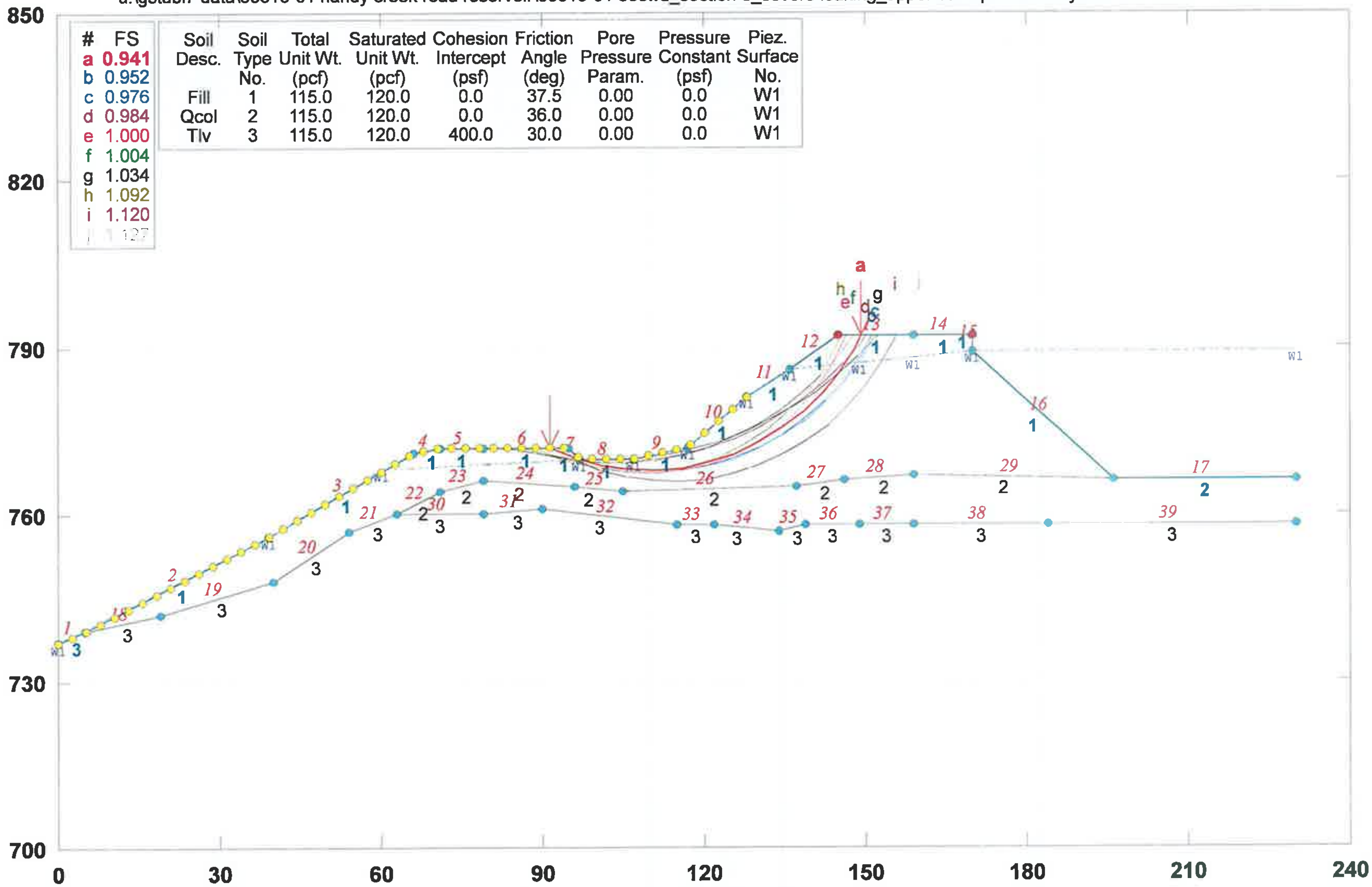


GSTABL7 v.2 FSmin=0.429

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

33615-01 Handy Creek Rd. Reservoir Section E-E' - Severe Leak_Upper_Static

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_severe leaking_upper bank.pl2 Run By: JH 3/11/2014 11:28AM



GSTABL7 v.2 FSmin=0.941

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

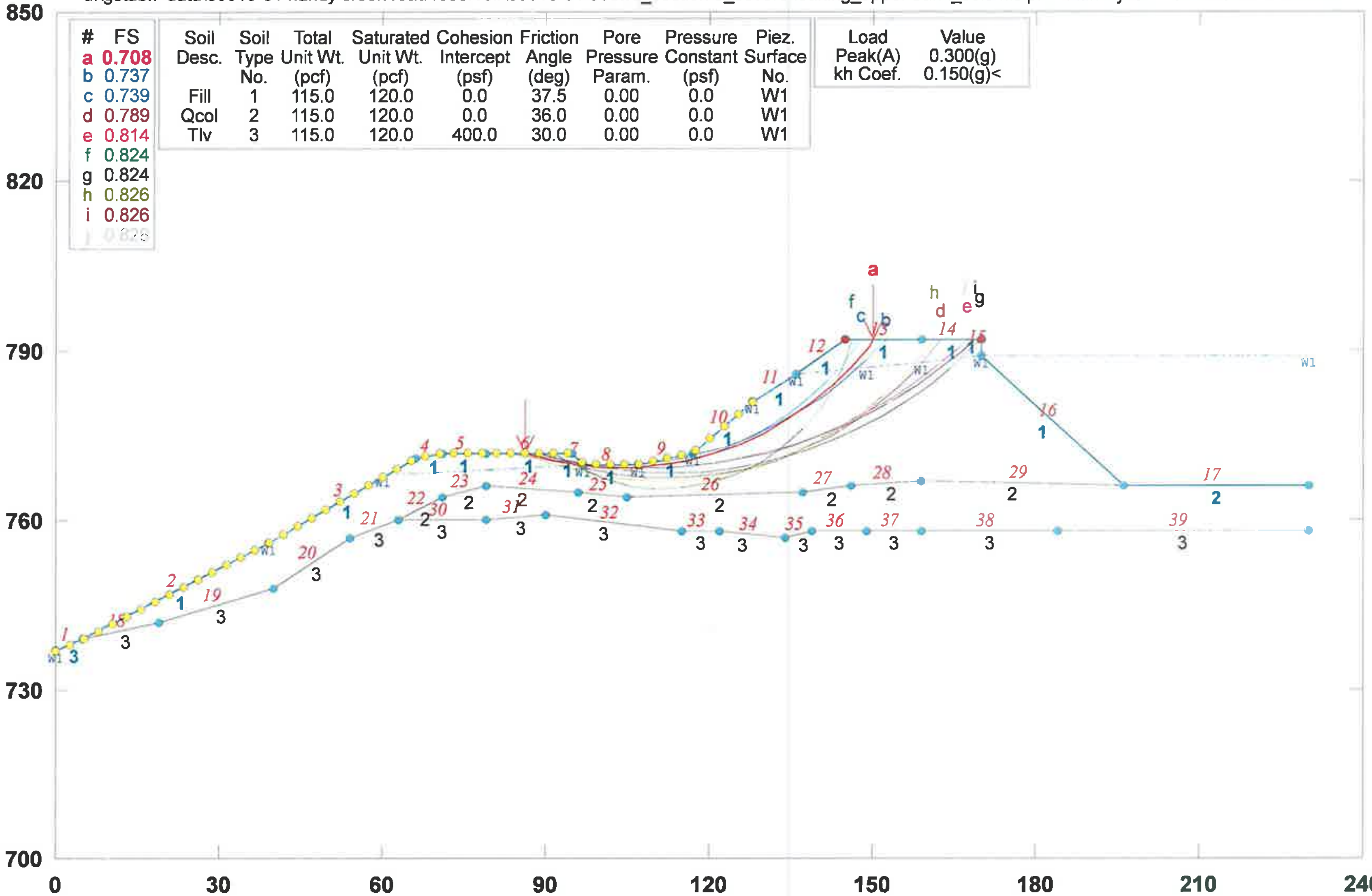
33615-01 Handy Creek Rd. Reservoir Section E-E' - Severe Leak_Upper_Seismic

u:\gstabl7 data\33615-01 handy creek road reservoir\33615-01 eocwd_section e_severe leaking_upper bank_seismic.pl2 Run By: JH 3/11/2014 11:29AM

#	FS
a	0.708
b	0.737
c	0.739
d	0.789
e	0.814
f	0.824
g	0.824
h	0.826
i	0.826

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	115.0	120.0	0.0	37.5	0.00	0.0	W1
Qcol	2	115.0	120.0	0.0	36.0	0.00	0.0	W1
Tlv	3	115.0	120.0	400.0	30.0	0.00	0.0	W1

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=0.708

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

To: **Municipal Water District of Orange County**
P.O.Box 20895
Fountain Valley, CA 92728

Attention: **MR. RICHARD B.BELL**
Principal Engineer
& **MR. LEE A. JACOBI**
Senior Engineer

Subject: **Five Earthquake Scenario Ground Motion Maps for Northern Orange County**

Introduction and Purpose:

At your request, and per your authorization, Earth Consultants International (ECI) has prepared this report summarizing five earthquake scenarios within the Municipal Water District of Orange County's (MWDOC) service area in the northern part of Orange County. The five earthquakes that we modeled were selected in consultation with you, and include: the Peralta Hills fault, the Whittier fault, the Puente Hills fault, the San Joaquin Hills fault, and the Newport-Inglewood fault. The The purpose of our study was to calculate and map the strong ground motions that the five earthquakes would generate during a geologically determined plausible event. This report presents the results of our study.

Scope of Work:

Specific tasks that we completed for this study are listed below:

- We identified the known active faults with highest probability of movement in Orange County.
- We reviewed published papers describing the five faults in the vicinity. These faults include the Peralta Hills Thrust, Puente Hills Thrust, Whittier, San Joaquin Hills thrust, and Newport-Inglewood faults.
- We developed a GIS-based ground motion modeling program based on a methodology, similar to that used by the U.S. Geological Survey (USGS), to calculate the level of shaking caused by a defined earthquake on each fault. These are deterministically generated computer maps of the Peak Ground Acceleration (PGA) and intensity of ground shaking expected in Orange County area from five different potential earthquakes that this area could experience.
- We prepared this report summarizing our results.

Earthquake Scenarios:

Earthquake scenarios describe the expected ground motions and effects of specific hypothetical large earthquakes. To make the calculations, we rely on consensus-based information about the potential behavior of the faults to assume that a particular fault or fault segment will rupture over a certain length. Once the size and location of the hypothetical earthquake are chosen, the ground motions at all locations in the region are estimated. In the five cases used here, both larger and smaller earthquakes are possible, but we have chosen magnitudes that seem to best represent the fault's average behavior.

Please note that these earthquake scenarios are not earthquake predictions. That is, no one knows in advance when a future earthquake will occur, or how large it will be. However, if we make assumptions about the size and location of hypothetical future earthquake, we can make a reasonable prediction of the effects of the assumed earthquake when it occurs.

We calculated the five earthquake scenarios that we consider as the most significant cases for the MWDOC's Orange County service area. The scenarios that we chose are as follows:

1) A magnitude 7.5 earthquake on the Puente Hills thrust;
2) A magnitude 6.6 earthquake on the San Joaquin Hills thrust;
3) A magnitude 6.8 earthquake on the Whittier fault;
4) A magnitude 6.9 earthquake on the Newport-Inglewood fault;
5) A magnitude 6.8 earthquake on the Peralta Hills thrust.

Figure 1 shows the simplified fault map for Orange County. The Whittier and Newport Inglewood faults are thought to be near the end of their earthquake cycles and both faults extend through the study area, with the potential to cause significant damage in the area. The Puente Hills blind thrust is also important, because recent seismologic and geologic studies have revealed the danger of this system which directly beneath Los Angeles (Shaw and Shearer 1999, Dolan et. al 2003). Parameters used for calculating the scenarios are summarized in Table 1.

Fault Name	Fault Type per CGS	Length (km)	Slip Rate (mm/yr)	Maximum Magnitude Earthquake	Approximate Recurrence Interval (yrs)
Puente Hills Blind Thrust	B	44 +/- 4	0.7 +/- 0.4	7.5	2,750
Newport-Inglewood (onshore)	B	75	1 +/- 0.5	6.9	2,200-3,900
Whittier	A	38 +/- 4	2.5 +/- 1	6.8	1,100
Peralta Hills	B	10 to 30	unknown, < 1 mm	6.8	unknown
San Joaquin Hills	B	28 +/- 3	0.5 +/- 0.2	6.6	1,600 - 3,100

Estimating Ground Motions for Scenario Earthquake:

To analyze ground shaking accelerations at these sites due to the selected scenario earthquakes, we employed deterministic analyses, using industry-standard software (EZ-FRISK by Risk Eng. Inc.). Deterministic analyses estimate the Horizontal Peak ground accelerations (PGA) that can be expected at a site due to earthquake rupture of the selected fault, attenuated by distance from the fault and the geologic conditions of the site. Because of the large area, we performed our analysis in 2.5 km grid nodes, then used a contouring program to generate the final maps. The parameters selected for display in the maps are PGA and instrumental intensity, which correlate well with damage and is also a convention that users are familiar with. This deterministic PGA analysis accounts for the effects of earthquake magnitude and distance, and the physical properties of the sediment underlying the each grid location. Our method is similar to that used by the USGS where ground motions are estimated using an empirical attenuation relationship.EZ-FRISK incorporates fault parameters provided by the California Geological Survey (CGS). For creating the scenario maps we used the attenuation relationship of Boore et al. (1997) for calculating peak ground acceleration for bedrock. Local soil characteristics are coming from the WRMS documentation (2000). We then correct the amplitude at each location with the amplitude and frequency-dependent factors determined by Borcherdt (1994). The amplification correction used is based on the Quaternary, Tertiary, Mesozoic ("QTM") geological classification of Park and Ellrick (1998). These categories can be considered to represent soil, soft rock, and hard rock, respectively, and hence they provide a very simple but effective way to assign amplification factors on a large scale. To obtain site amplification factors based on these QTM categories, we used the mean shear-wave velocities assigned to each unit, and then applied the frequency and amplitude-dependent amplification factors determined by Borcherdt (1994) based on these velocities.

For each scenario earthquakes we have prepared two maps, one showing the peak ground acceleration and one showing the shaking intensity away from the fault. On the right hand side in each set of figures, the ocation of the fault is also represented. Calculated ground acceleration results from these scenario earthquakes are shown on Figures 2 through 6. Figures 2a, 3a, etc. show the anticipated seismic intensities as a result of the earthquake scenarios, whereas Figures 2b, 3b, etc. show the peak ground accelerations expected in the area.

The approach used in this study is only approximate in that it shows the average ground motion effect for producing the scenario shake maps. Fault location is accounted for in the present method, but the direction of rupture was not taken into account. The methodology gives average peak ground motion values so it does not account for all the expected variability in motions other than the site amplification variations. Following the USGS earthquake scenario methodology, the way of including the site amplification in the current study is conservative. However, these simple geological classifications are available for the whole Orange County area, and considering the 2.5 km grid size used, are probably accurate enough for the purpose of the analysis. A compilation of more localized (and better constrained) site corrections based on either shallow soil velocity profiles or mainshock / aftershock studies may improve the results. However, such data are not uniformly available.

Actual strong ground motions depend largely on the local site effects, show significant variability for a given distance, magnitude, and site condition and, hence, the scenario ground motions are more uniform than would be expected for an actual earthquake. The true variations are partially attributable to three-dimensional (3D) wave propagation, path effects (such as basin edge amplification and focusing), differences in motions among earthquakes of the same magnitude, and complex site effects not accounted for by the present method. A complete modeling of a scenario earthquake to incorporate all of these factors is a very labor and data intensive effort, and well beyond the stated needs of this project.

We appreciate the opportunity to work with Municipal Water District of Orange County, and we trust that the information provided herein provides you with the information you need at this time. Should you have any questions regarding this report, please do not hesitate to contact either of us.

Respectfully submitted on behalf of
EARTH CONSULTANTS INTERNATIONAL,
Registered Geologists and Certified Engineering Geologists in the State of California

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Project Consultant
(714) 412-0641
gurler@earthconsultants.com

References:

Boore, D. M., W. B. Joyner, and T.E. Fumal (1997). Equations for Estimating Horizontal Response Spectra and Peak Accelerations from Western North American Earthquakes: A Summary of Recent Work,*Seism. Res. Lett.*, **68**, 128-153.

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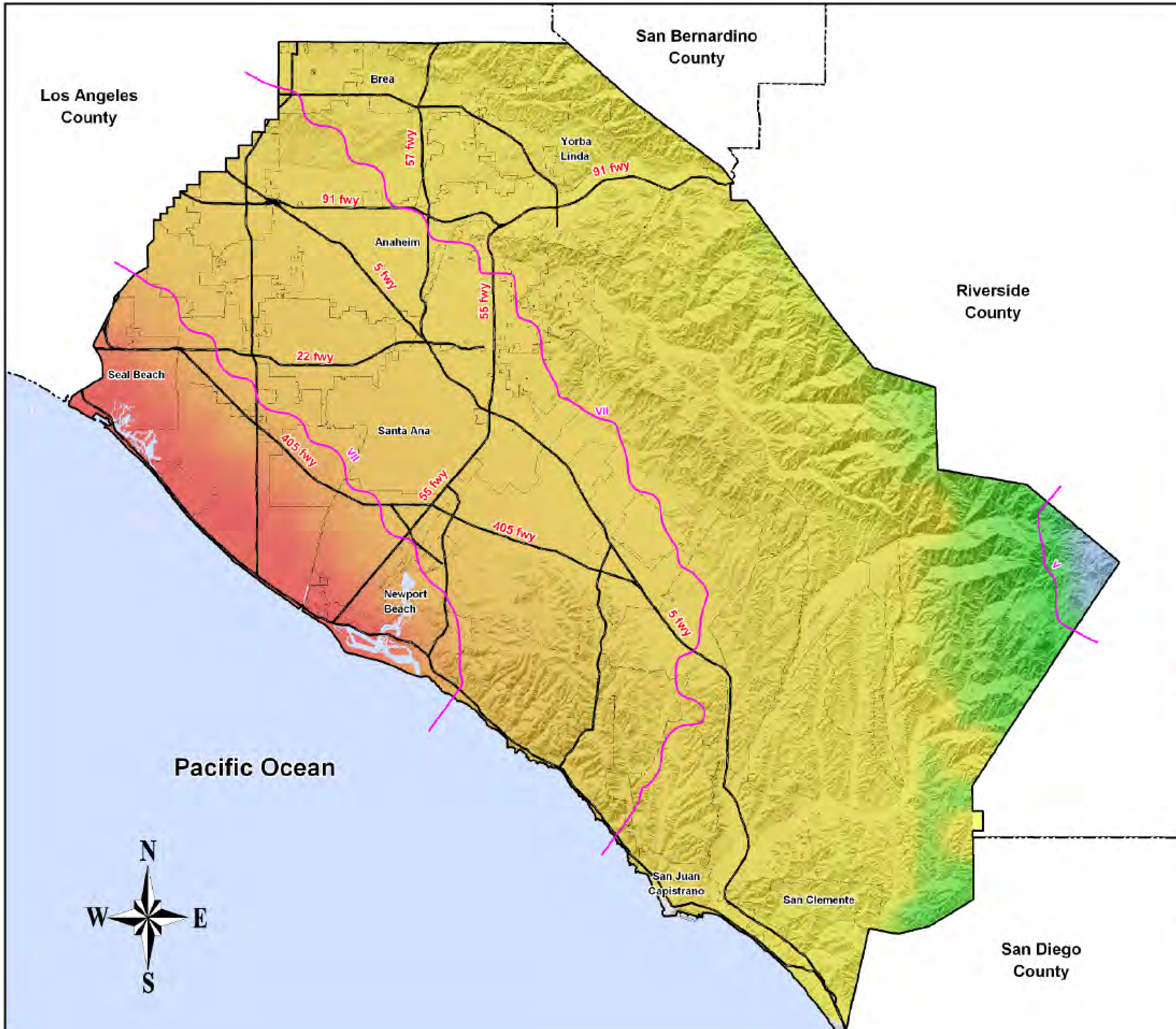
Intensity Map for M6.9 Newport-Inglewood Fault Scenario Orange County, California



Intensity Contour Line

PERCEIVED SHAKING	Not Felt	Weak	Light	Mod	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Mod	Mod/ Heavy	Heavy	Very Heavy
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

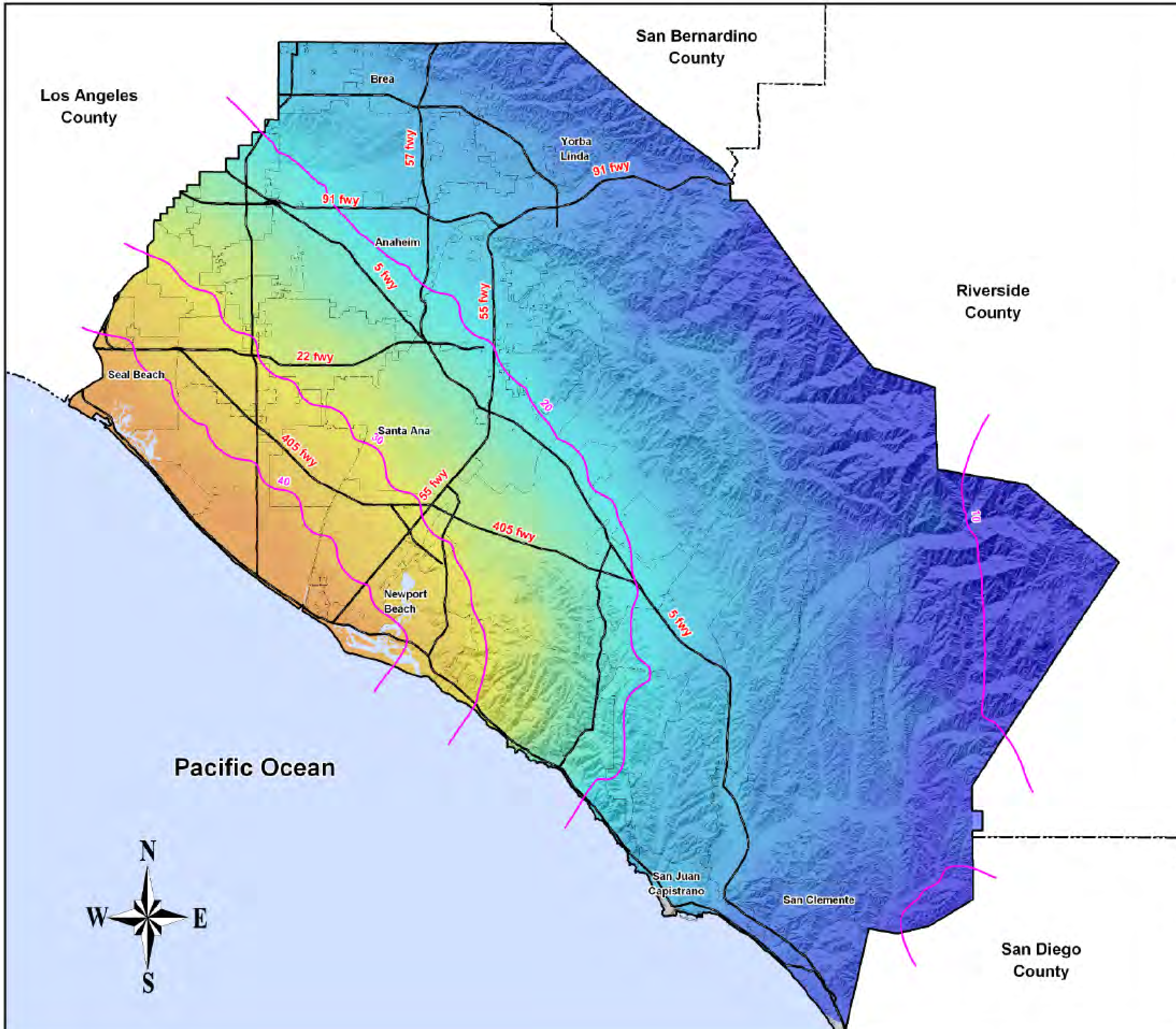
5 0 5 10 15 Kilometers

3 0 3 6 9 Miles

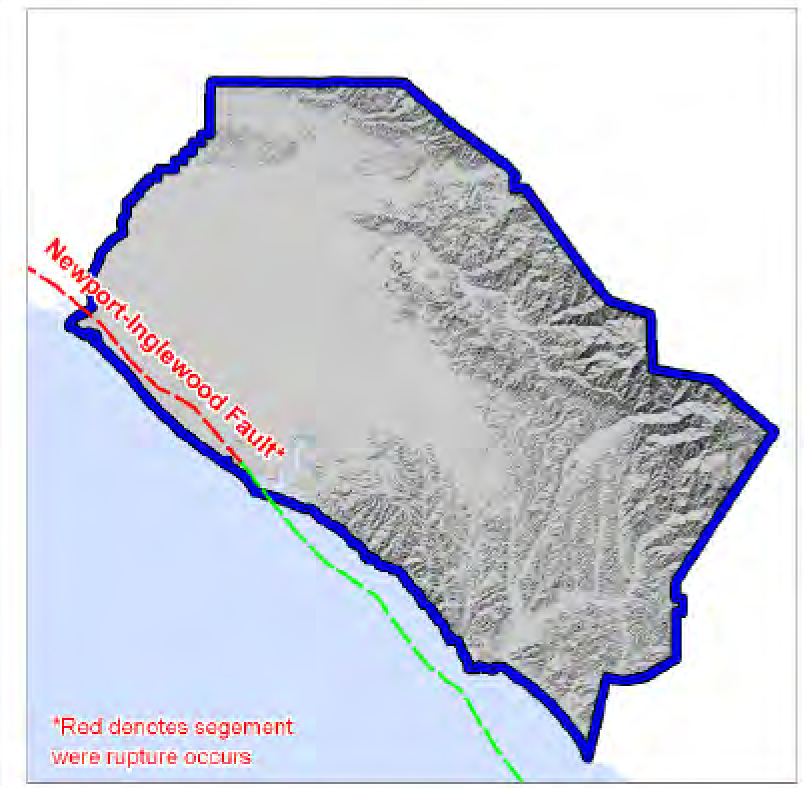
Earth Consultants International

Project Number: 2509
Date: June 2005

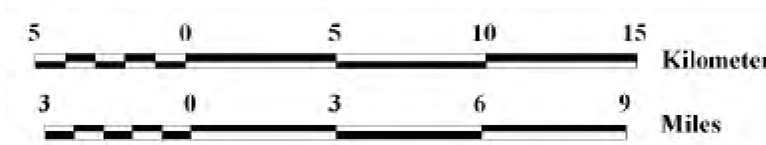
Figure 2a



Peak Ground Acceleration (in %g) for M6.9 Newport-Inglewood Fault Scenario Orange County, California



%g Contour Line



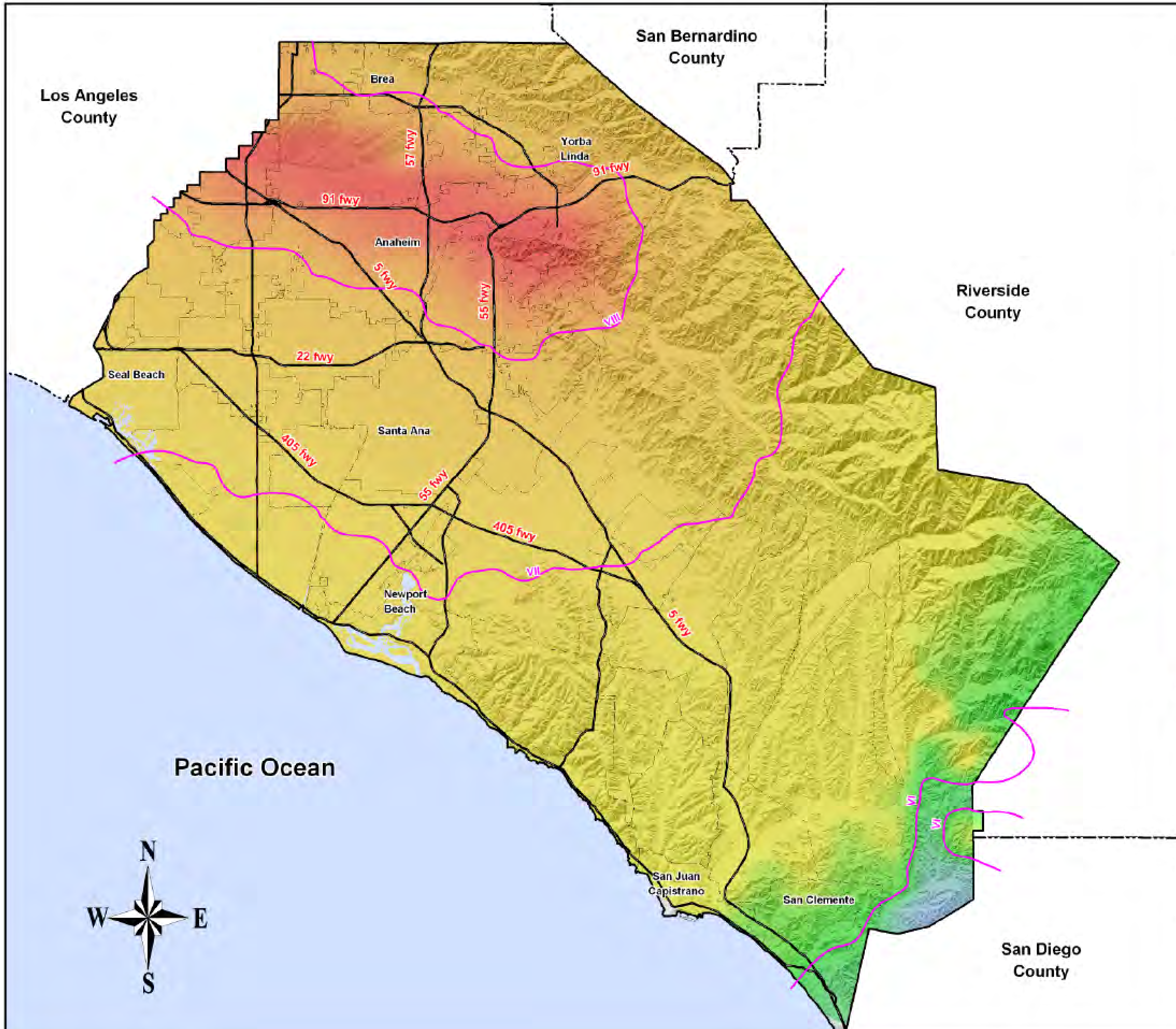
Earth Consultants International

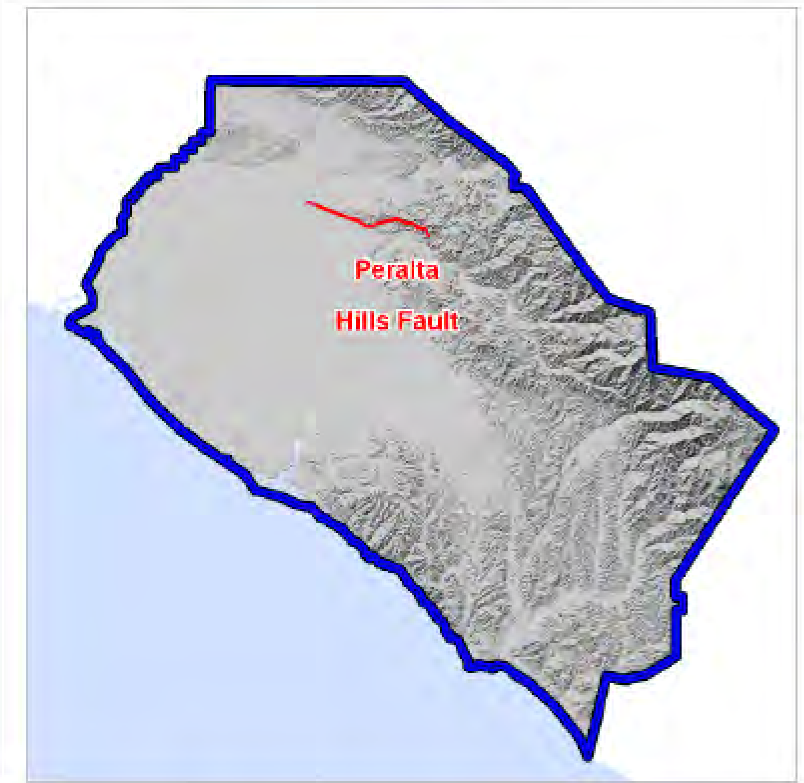
Project Number: 2509
Date: June 2005

Figure 2b

Intensity Map for M6.8 Peralta Hills Fault Scenario

Orange County, California





Intensity Contour Line

PERCEIVED SHAKING	Not Felt	Weak	Light	Mod	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Mod	Mod/Heavy	Heavy	Very Heavy
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

5 0 5 10 15 Kilometers

3 0 3 6 9 Miles

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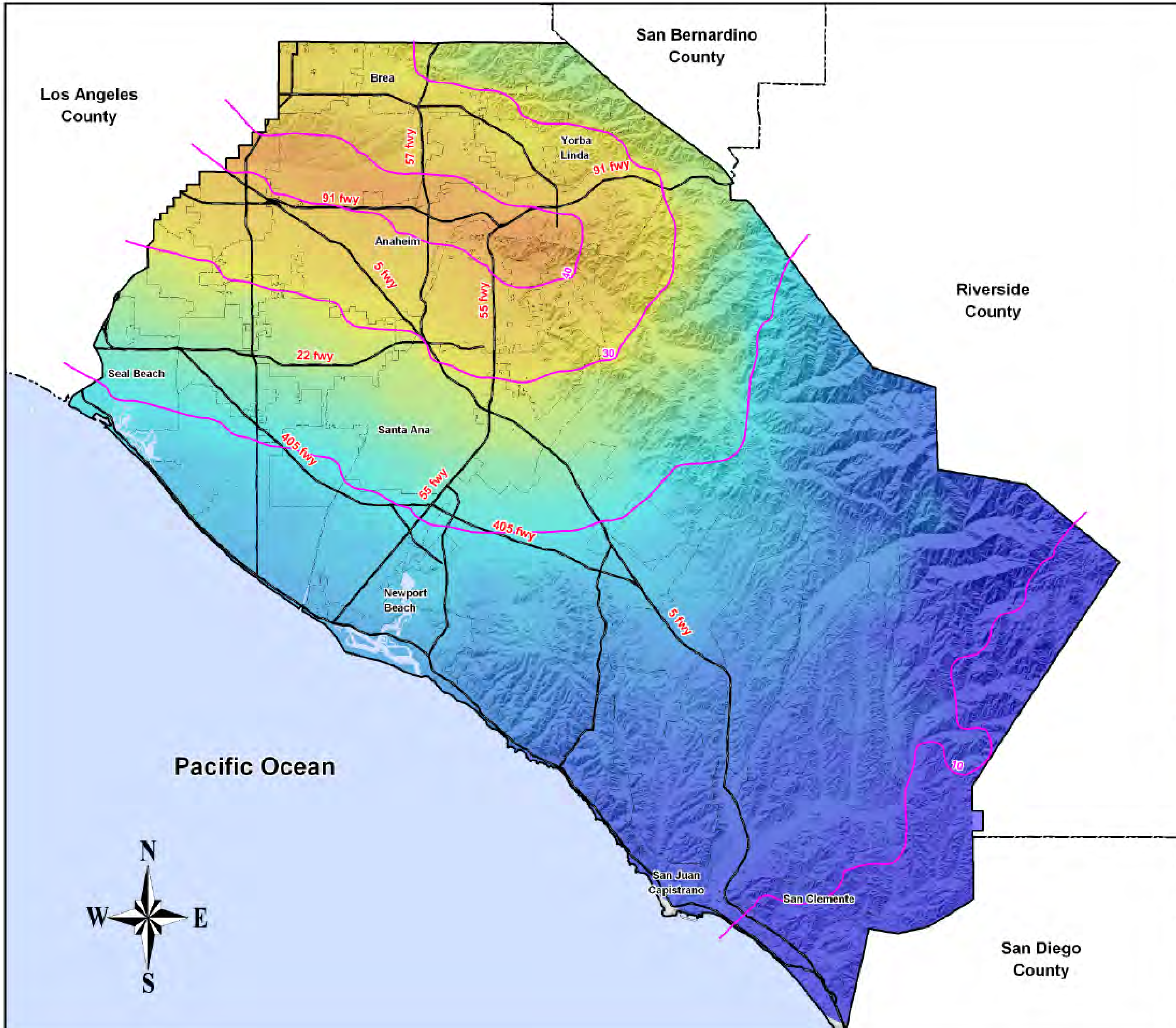
Figure 3a

Peak Ground Acceleration (in %g) for M6.8 Peralta Hills Fault Scenario Orange County, California



%g Contour Line

10 20 30 40 50 60

Z-units:%g

5 0 5 10 15 Kilometers

3 0 3 6 9 Miles

Earth Consultants International

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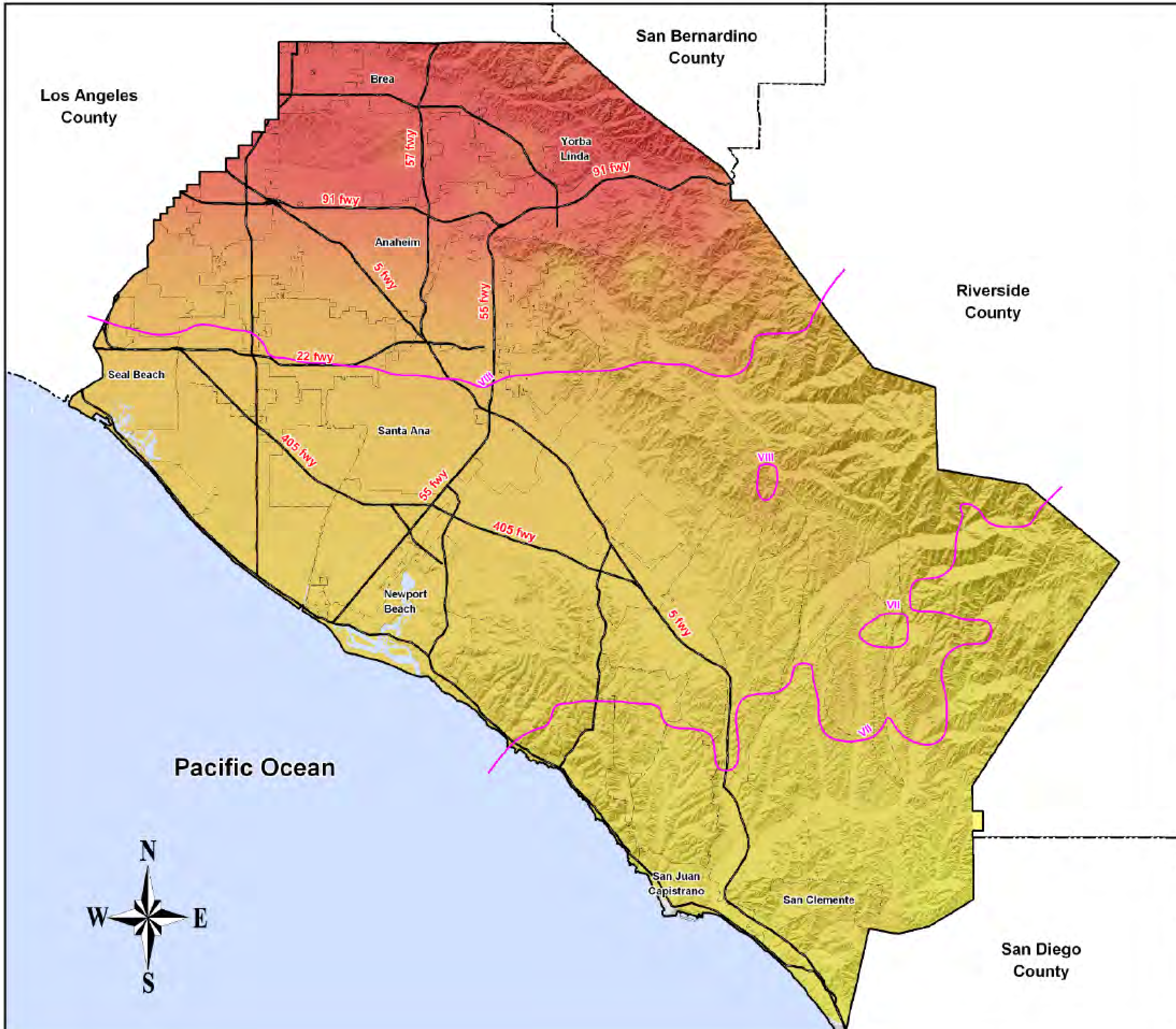
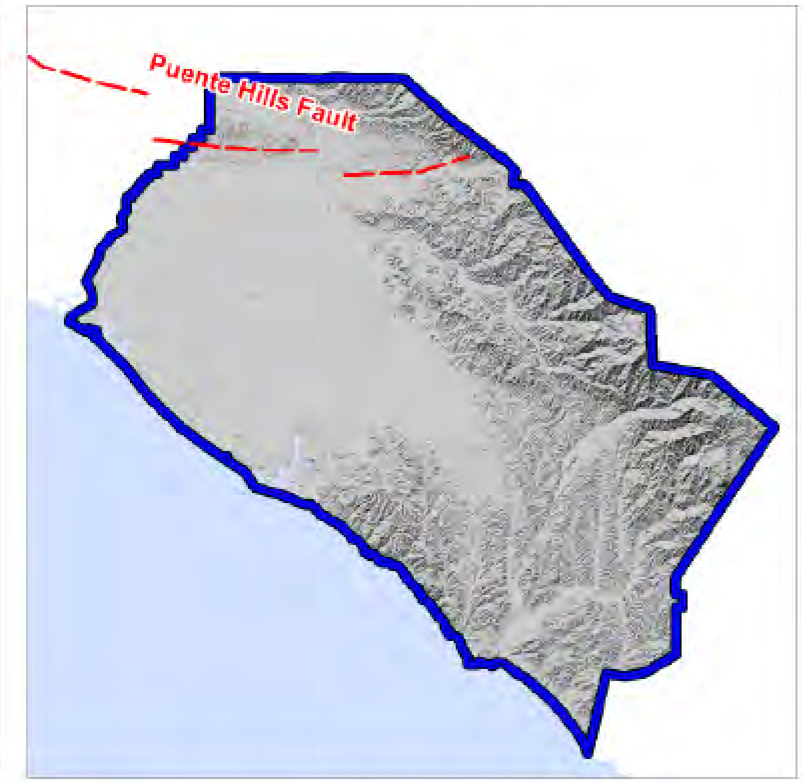
Date: June 2005

Figure 3b

Intensity Map for M7.2 Puente Hills Fault Scenario

Orange County, California





Intensity Contour Line

PERCEIVED SHAKING	Not Felt	Weak	Light	Mod	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Mod	Mod/ Heavy	Heavy	Very Heavy
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

5 0 5 10 15 Kilometers

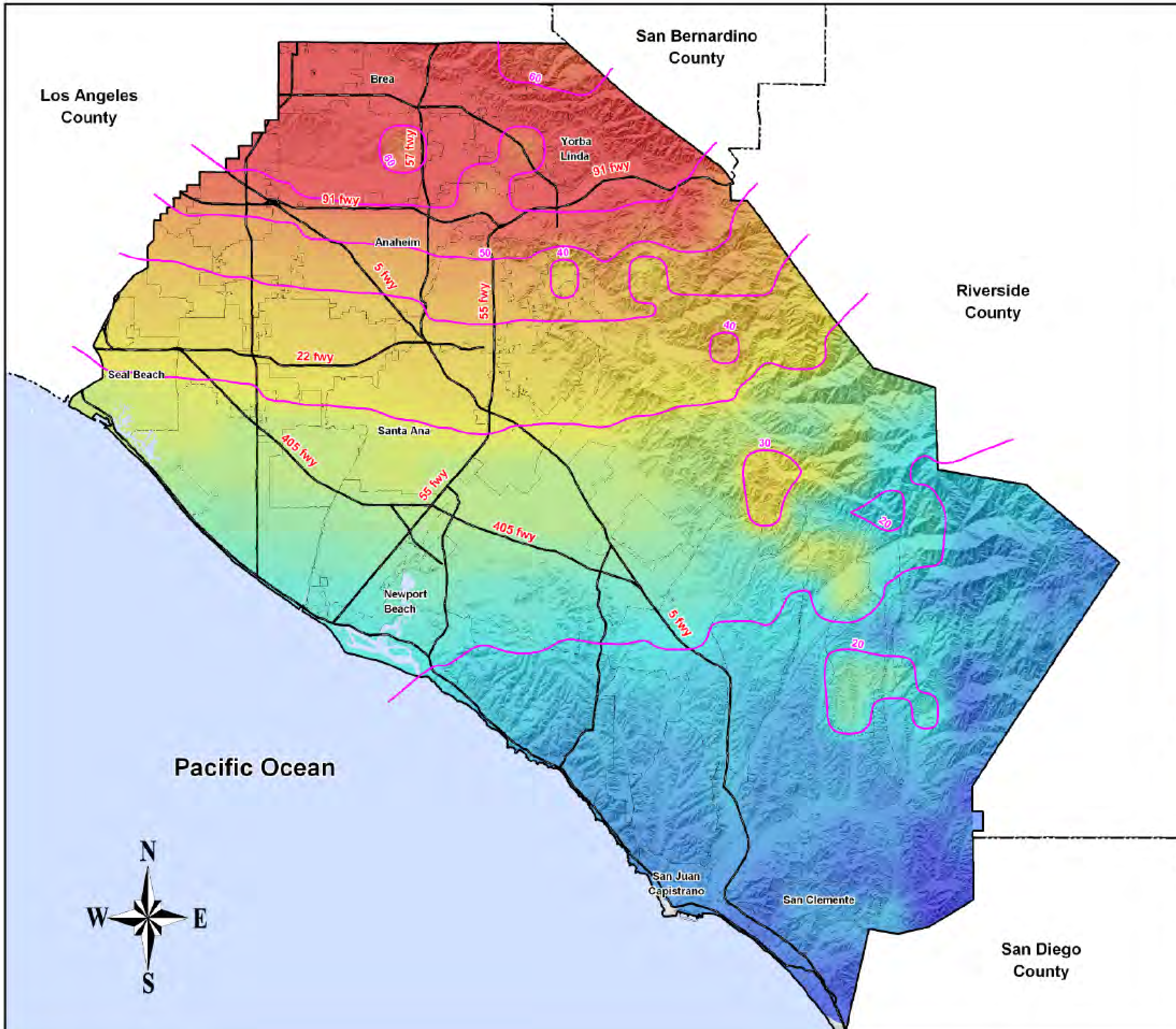
3 0 3 6 9 Miles

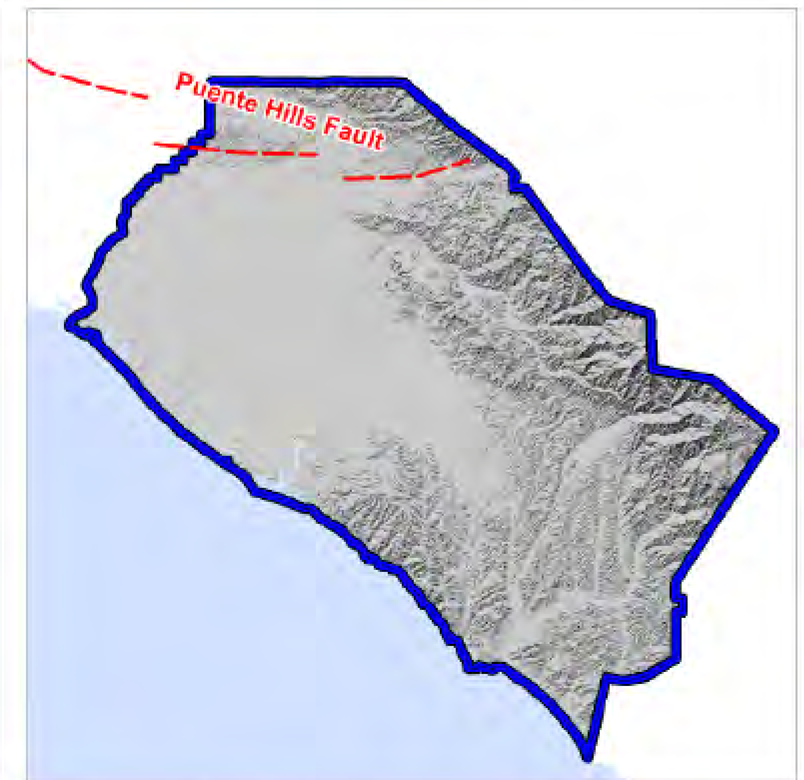
Earth Consultants International

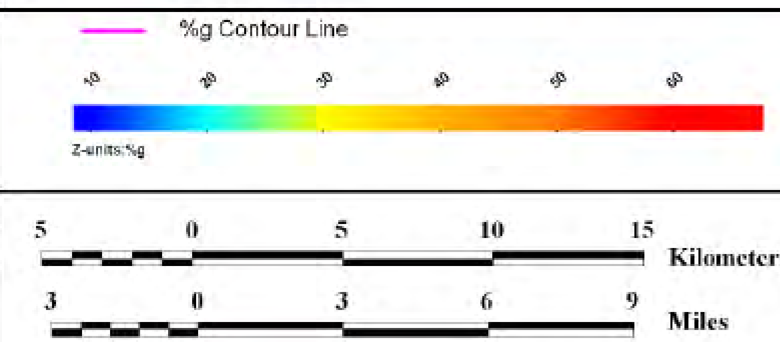
Project Number: 2509
Date: June 2005

Figure 4a

Peak Ground Acceleration (in %g) for M7.2 Puente Hills Fault Scenario Orange County, California







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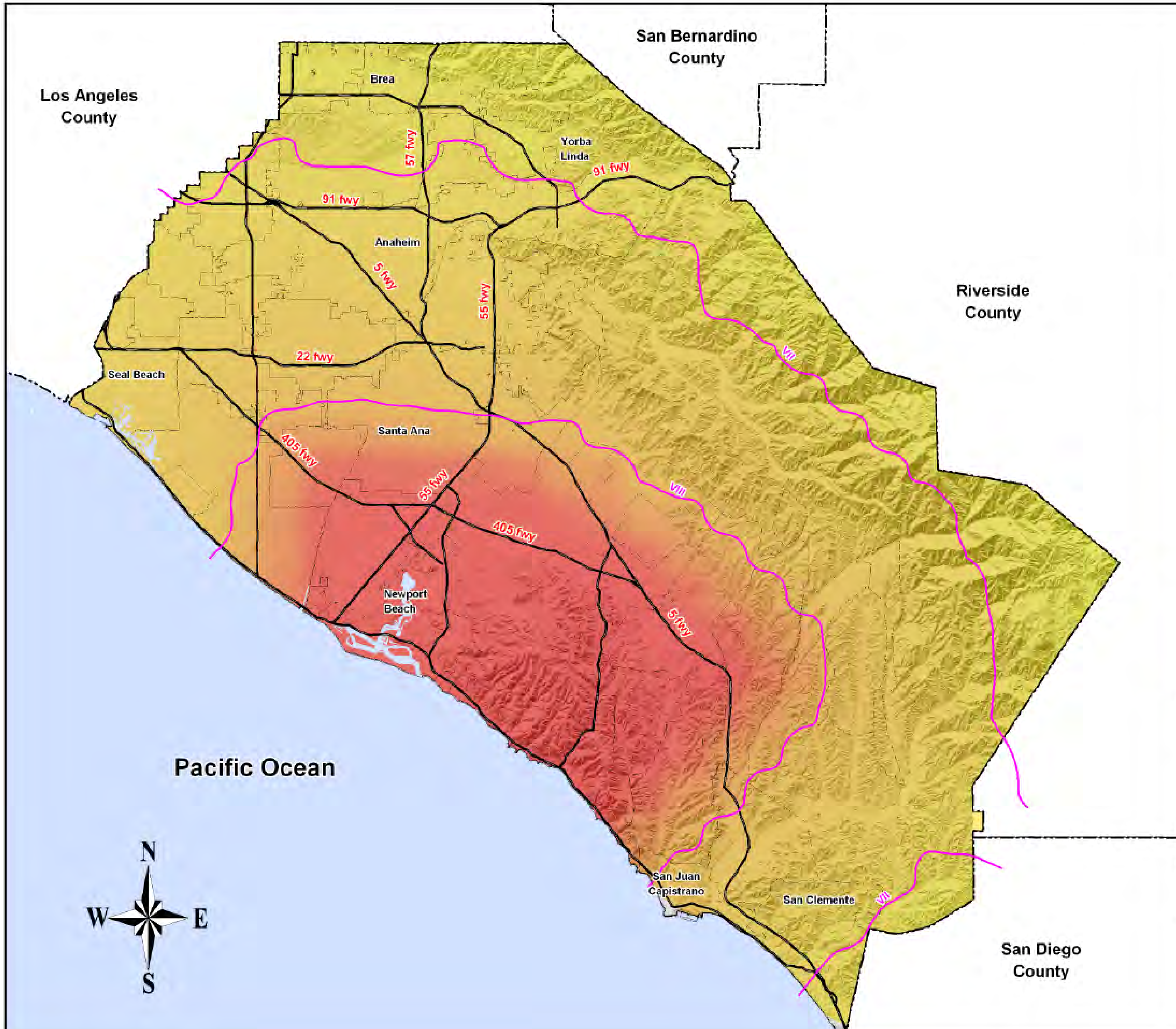
Date: June 2005

Figure 4b

Intensity Map for M6.6 San Joaquin Hills Fault Scenario

Orange County, California



Intensity Contour Line

PERCEIVED SHAKING	Not Felt	Weak	Light	Mod	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Mod	Mod/Heavy	Heavy	Very Heavy
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

5 0 5 10 15 Kilometers

3 0 3 6 9 Miles

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Date: June 2005

Figure 5a

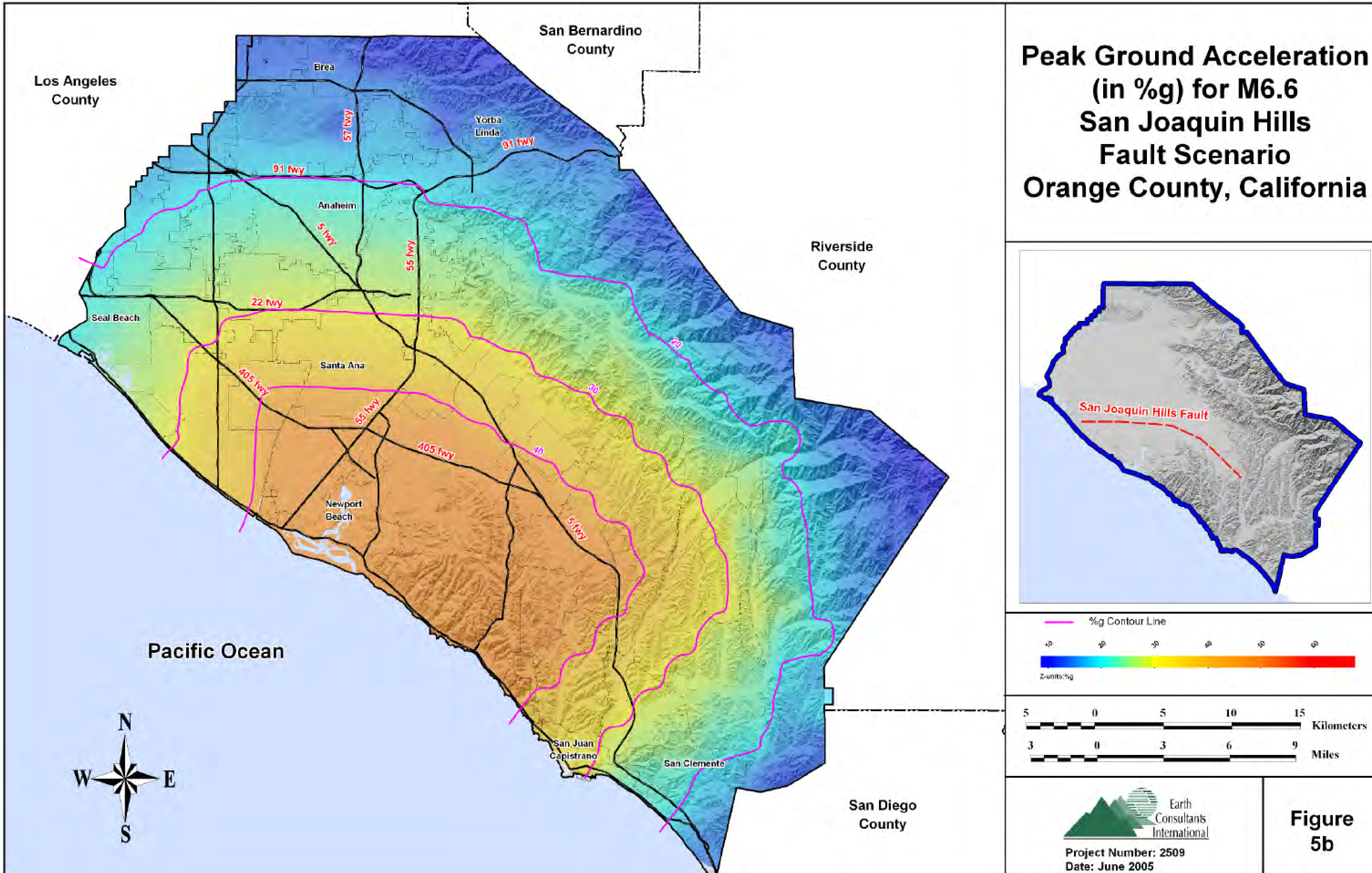
Peak Ground Acceleration (in %g) for M6.6 San Joaquin Hills Fault Scenario Orange County, California
Los Angeles County
San Bernardino County
Riverside County
San Diego County
Pacific Ocean
Brea
Yorba Linda
Anaheim
Seal Beach
Santa Ana
Newport Beach
San Juan Capistrano
San Clemente
91 fwy
57 fwy
5 fwy
55 fwy
22 fwy
405 fwy
20
30
40
San Joaquin Hills Fault
%g Contour Line
Z-units:%g
Kilometers
Miles
N
S
E
W
Earth Consultants International
Project Number: 2509
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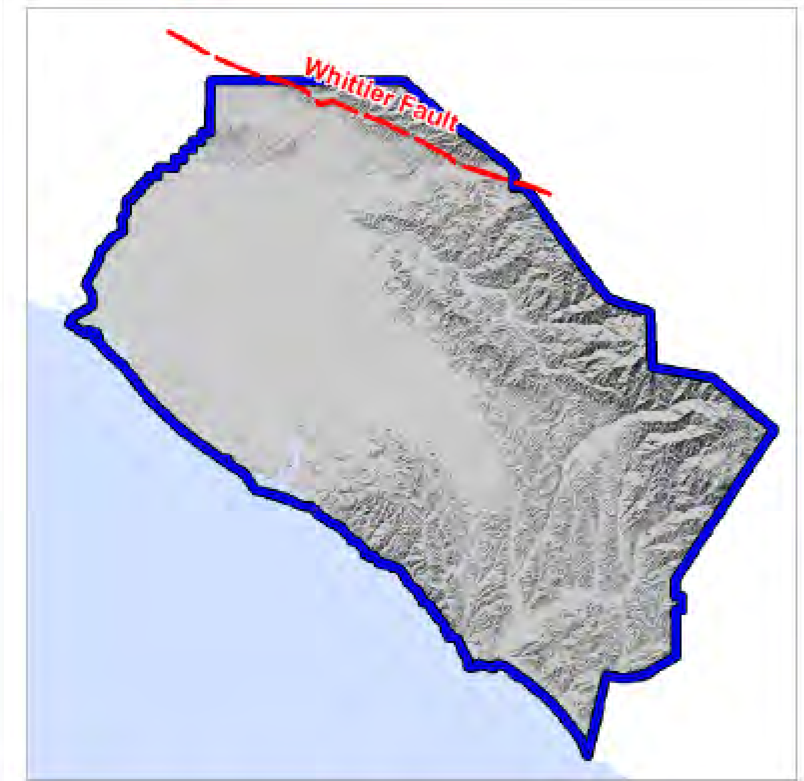
Figure 5b

Intensity Map for M6.8 Whittier Fault Scenario

Orange County, California



Intensity Contour Line

PERCEIVED SHAKING	Not Felt	Weak	Light	Mod	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Mod	Mod/Heavy	Heavy	Very Heavy
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

5 0 5 10 15 Kilometers

3 0 3 6 9 Miles

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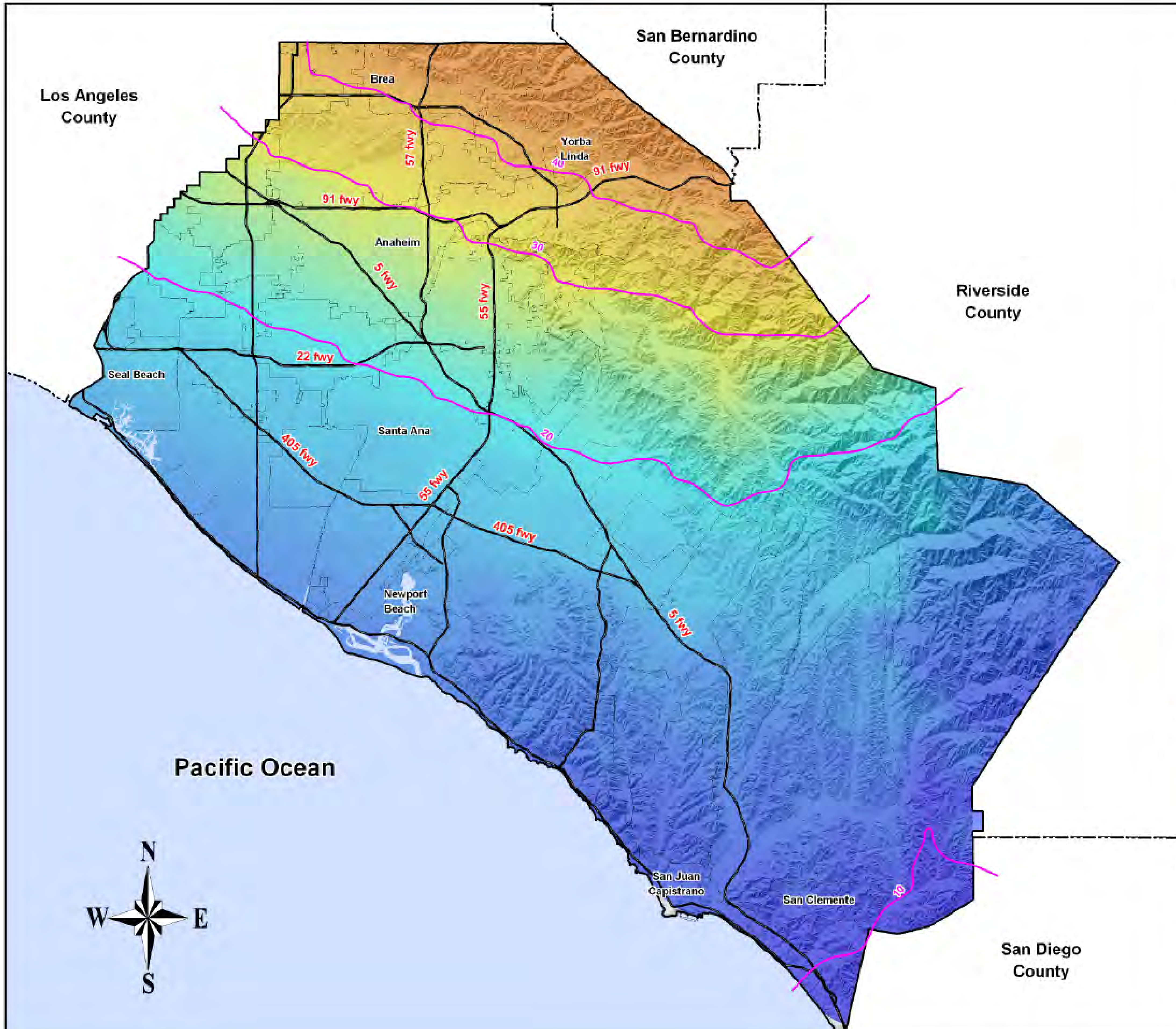
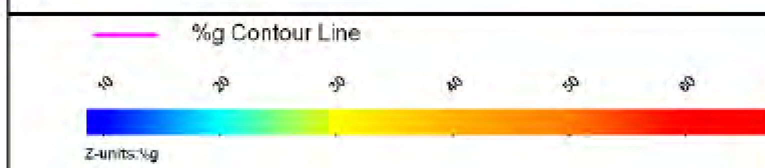
Project Number: 2509
Date: June 2005

Figure 6a



Peak Ground Acceleration (in %g) for M6.8 Whittier Fault Scenario Orange County, California



5	0	5	10	15	Kilometers
3	0	3	6	9	Miles

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Project Number: 2509
Date: June 2005

Figure 6b

The Peralta Hills Fault, a Transverse Ranges Structure in the Northern Peninsular Ranges, Southern California

GEOLOGY AND MINERAL WEALTH OF THE CALIFORNIA TRANSVERSE RANGES

Mark E. Bryant and Donald L. Fife, Converse Consultants, Inc., P.O. Box 6288, Anaheim, California 92806

ABSTRACT

The dominant structural feature of the Peralta Hills, within the transition between the Transverse and the Peninsular Ranges, is the north-dipping reverse left(?)-oblique Peralta Hills fault. Youthfulness of this fault is well demonstrated at several localities along its approximate 5-mile (8km) sinuous trace across the southern Peralta Hills. For the most part, the fault plane is concordant with bedding planes, while certain anomalous near-surface features are similar to conditions reported for other reverse faults throughout the Transverse Ranges. Considering the structural setting of the area, the Peralta Hills fault may be related to flexural slip and not deep-seated movement. There is uncertainty as to whether or not the Peralta Hills fault is capable of producing a large-magnitude earthquake. However, a great deal of evidence suggests that this fault should be considered a potential geologic hazard from both a ground rupture and a ground-shaking viewpoint. For the present, it must be assumed that a potentially destructive earthquake, similar to past seismic activity associated with other reverse faults in the Transverse Ranges, may be possible in the future. Before the Peralta Hills fault is more clearly understood and its implications evaluated, additional research and subsurface exploration are needed in this intensely urbanized area.

INTRODUCTION

Only recently has a significant fault been recognized along the southern edge of the Peralta Hills. This fault, locally known as the Peralta Hills fault, attains a total known length of about 5 miles (8 km) and is conspicuously similar to reverse faults within the Transverse Ranges. The trace of the Peralta Hills fault generally parallels the east-west trend of the Santa Ana River and coincides with the northern topographic termination of crystalline rocks of the Peninsular Ranges along the southern margin of the Los Angeles sedimentary basin. At several localities, youthfulness of the Peralta Hills fault is well demonstrated.

This paper reviews and discusses the fault characteristics, geologic conditions of the area, previous interpretations, recorded seismicity, and analogies to the other active reverse faults. Also, a significant question arises as to whether the Peralta Hills fault is a major tectonic feature capable of producing large-magnitude earthquakes, or simply related to flexural folding within the sedimentary interval and not capable of such potentially damaging earthquakes.

GEOLOGIC SETTING

The Transverse Ranges Province is typically characterized as an east-west trending geomorphic unit, somewhat anomalous to the prominent northwest-southeast structural alignment of California. In contrast to the generally right-lateral strike-slip motion of major faults in other provinces, the Transverse Ranges Province is dominated by folding and reverse faulting as a result of north-south crustal shortening. The Peralta Hills fault, considered here within a transition zone between the northern Peninsular Ranges and the central Transverse Ranges, exemplifies this north-dipping reverse to thrust faulting found throughout the Transverse Ranges Province. Principal faults of the central Transverse Ranges and northern Peninsular Ranges, as well as the Peralta Hills fault, are shown in Figure 1.

The Peralta Hills consist of a north-dipping homoclinal structure with subordinate folds and faults. The exposed Cenozoic sedimentary interval includes the Pliocene Fernando Formation, Upper Miocene Puente Formation, Middle Miocene Topanga Formation, and Upper Eocene to Lower Miocene Vaqueros-Sespe Formation. The combined stratigraphic thickness of these formations is on the order of 7,000 feet (2,100m) in the vicinity of Peralta Hills (Schoellhamer and others, 1981). Quaternary terrace and alluvial deposits surround this elevated bedrock nose on three sides. The dominant structural feature is the Peralta Hills fault which generally trends along the southern edge of the hills (Figure 2).

PERALTA HILLS FAULT

Previous investigations of the Peralta Hills by Richmond (1953), Yerkes (1957), Morton and others (1973), Schoellhamer and others (1954, 1981), and Morton and Miller (1981) revealed a thrust fault approximately one mile (1.6km) in length, on the southern side of the hills. The Middle Miocene Topanga Formation or the Upper Miocene Puente Formation were shown in juxtaposition with Pleistocene terrace deposits. Fife and others (1980), and various geotechnical reports, including unpublished data by Converse Consultants, have disclosed similar fault segments at other nearby localities. Based on this compilation of data, the fault is considered to be a single sinuous trace across the hills, or possibly, several closely-spaced en echelon faults, with a documented total length of more than 5 miles (8km).

decreasing depth to the west; Main ba[...] Near surface at west end of PH.

The Peralta Hills fault ascends from about elevation 300 feet (90m) at the west end to about elevation 900 feet (275m) at the eastern terminus. The westerly half of the fault generally bounds the southern edge of the hills, where it traverses through urbanized land. The easterly half is within natural terrain. For much of its length, bedrock has been thrust southward over terrace deposits. Typically, the fault plane ranges between a 30° and 70° dip to the north, and is locally concordant with the shale, siltstone and sandstone beds. However, it is apparent that the fault does cut across the structural grain of the sedimentary bedrock units on a more regional scale. At one location, the fault was studied in detail by Converse Consultants by way of deep exploratory borings and trenches. Here, the fault was distinguished by nearly

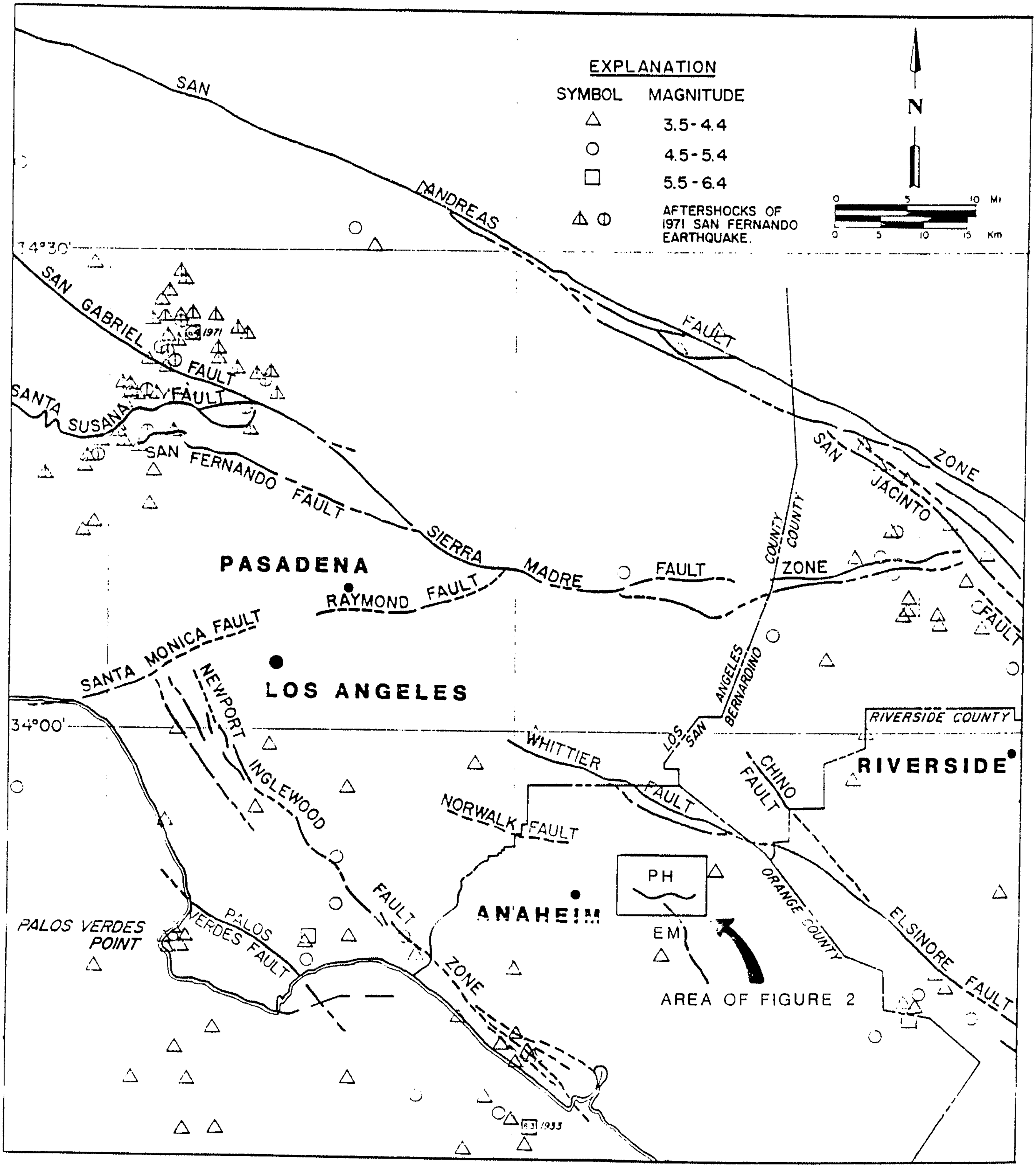


Figure 1. Regional fault map of the central Transverse Ranges and northern Peninsular Ranges. Principal Quaternary-age faults shown and Peralta Hills fault located in southern area. PH = Peralta Hills fault. EM = El Modeno fault.

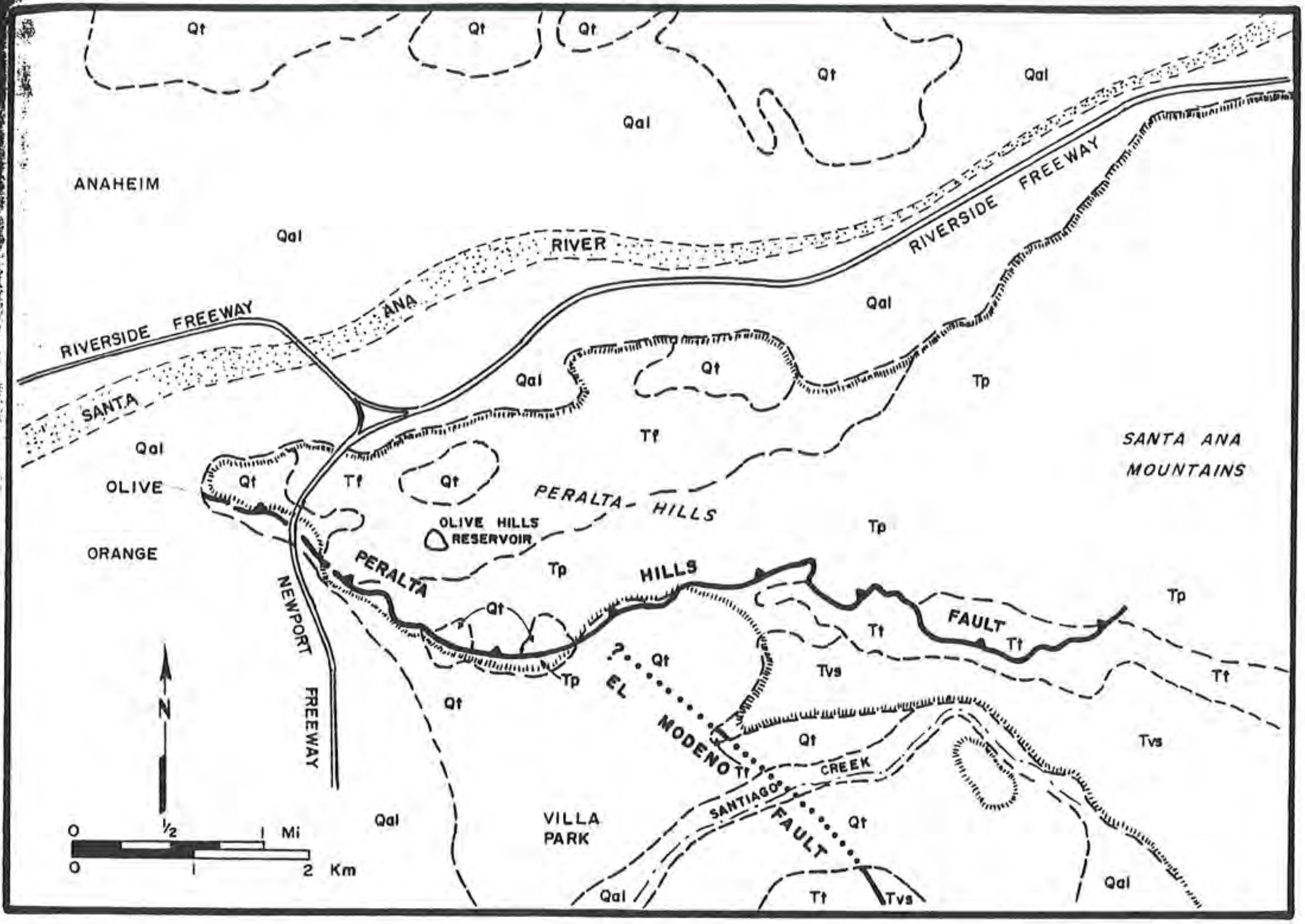


Figure 2. Generalized geologic map of Peralta Hills modified from Morton and Miller)1981). Qal = alluvium, Qt = terrace deposits, Tf = Fernando Formation, Tp = Puente Formation, Tt = Topanga Formation, Tvs = Vaqueros-Sespe Formation, = reverse fault, = normal fault (dotted where burried), = contact, = edge of hills.

parallel-to-bedding displacement at depth and a marked change to a south-dipping (gravity slide) plane near the surface as a continuous arcing shear surface (Photographs 1 and 2). Measured offset was indicated to be at least 250 feet (75m). Similar faulting conditions have been revealed at other localities. At a second location, Pleistocene terrace deposits have been substantially deformed on the downthrown (south) side. The stratification within these Pleistocene river gravels and overlying (Holocene?) colluvial deposits now dips to the south and away from the fault at 40° to 60°. At this same location, the resisting force along a stratum has been overcome by this tilting and (block) failure has resulted in the southerly direction.

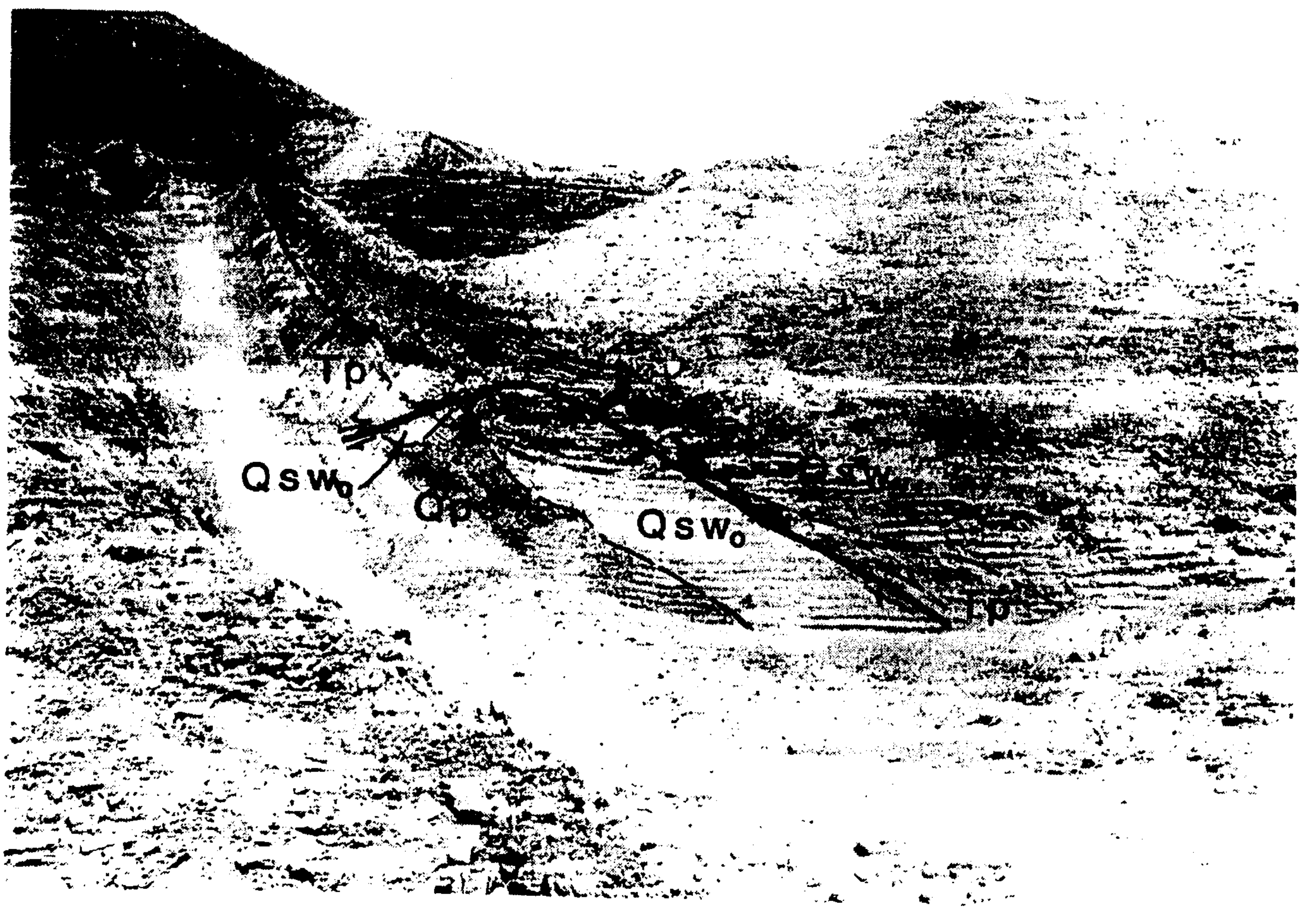
In general, the Peralta Hills fault, as depicted by us, has been unrecognized by previous workers. Erosion and mass wasting have tended to obliterate geomorphic features, while man-made changes to pre-existing surface conditions have further contributed to its obscurity. More recently, roadcuts and large excavations combined with exploratory work along the fault trace have aided us in acquiring valuable near-surface and subsurface data. Youthfulness of the fault was demonstrated at several localities by offsets extending into Pleistocene terrace deposits, and Holocene alluvial and colluvial deposits. Limited C-14 dating of the latter deposits in the vicinity of the aforementioned gravity slide suggested Holocene displacement.

About 10 miles (16km) to the south is another interesting east-west trending feature within the Miocene section consisting mostly of incompetent La Vida Member of the Puente Formation (Tan and others, 1982). This structure trends westward in upper Borrego Canyon for 4 miles (6km) along the north corner of the El Toro quadrangle, just west of the Cristianitos fault. It begins as an asymetrical fold becoming progressively overturned, then to a north-dipping reverse "sheared fold," then apparently into a more conventional north-dipping reverse fault. This feature appears to be an incipient "Peralta Hills fault," although the precise age of deformation has not been determined.

SEISMICITY

Recorded seismic activity for the period 1932-1981 has been rather sparse in the Peralta Hills area as evidenced by the earthquake epicenter plot obtained from the seismological laboratory of the California Institute of Technology earthquake data base. Approximately 60 seismic events occurred in this area, including three significant earthquakes with magnitudes of M3.5, M3.5 and M4.1. The others were less than M3.5 and considered microseismic. Most of the determined focal depths were greater than

Photograph 1. View looking south at east wall of bulldozer trench. Peralta Hills fault at south end. Wall height is about 15 feet (4.6m). Qswy = younger slopewash, Qswo = older slopewash, Qps = older paleosol, Tp = Puente Formation.



16,500 feet (5km), although several were as shallow as 6,500 feet (2km). A cluster of 10 microseismic epicenters were located 1.5 to 2.1 miles (2.4-3.4km) north of the western end of the known Peralta Hills fault. The associated hypocenters are generally consistent with a 60° dip projected from the surface trace, and range between 16,500 and 20,000 feet (5-6km). This focal depth is well within the more competent sedimentary beds, and may be deep enough to be in basement rocks. Of particular interest, the M4.1 earthquake hypocenter can be located on the aforementioned projected fault plane at its determined depth of 14,000 feet (4.3 km). It is possible, however, that the earthquake data may be fortuitous due to the lack of a well established seismicity pattern. Numerous shallow microseismic events may be related to the release of elastic strain energy in the near-surface zone. This is analogous to what takes place in some large landslides within sedimentary bedrock terrane; namely, continuous "slip-stick" movement which occurs along incompetent bedding planes, accompanied by localized shallow ground shaking.

While there is no direct evidence that the Peralta Hills fault generated any major (significant) historic earthquake, it is interesting to note a "short" westward projection would place the fault under the generally accepted location of the 1769 encampment of the Gaspar de Portola expedition (Figure 2). Portola reported the first major California earthquake on July 28, 1769, while camped on the banks of the Santa Ana River near the present site of Olive (Ritcher, 1958). We are not concluding that the Peralta Hills fault ruptured during or generated that earthquake nor are we ruling out that possibility.

DISCUSSION

We propose that the Peralta Hills fault be considered a potential geologic hazard from both a ground rupture and a ground shaking viewpoint. The fault has a known length of about 5 miles (8km), and it may extend for some distance to the east, and especially to the west. Definitive geologic evidence related to depth of faulting and the probability of relatively large-magnitude seismic activity are not known at this time. Future activity may be restricted to only aseismic (creep) movement and associated shallow microseismicity. However, it is conceivable that fault displacement at the surface could accompany a large-magnitude event. Yeats and others (1981), and Yeats (1982) cite examples and provide substantiating evidence for "rootless" reverse faults in the Ventura basin. They differentiate deep-seated faults capable of producing large-magnitude earthquakes from relatively shallow or "low-shake" reverse faults capable of ground rupture, but do not pose a seismic hazard. The latter type consist of either detachment thrusts, flexural-slip faults, or bending-moment faults due to horizontal shortening (Yeats, 1982). Considering these fault models, the Peralta Hills fault most closely resembles a flexural-slip fault.

Characteristic features assigned to Yeats and others' (1981) flexural-slip model include movement exclusively, or nearly, along bedding during folding, and lack of deep microseismicity. Another parameter mentioned is back-tilting of displaced alluvial deposits toward the upthrown block which suggests that the faults flatten with depth, and such faults typically occur as multiple nearly-parallel sets at a synclinal flank. Further, they indicate that surface rupture may result from a large-magnitude earthquake along a nearby "master" fault. The flexural-slip examples described by Yeats and others (1981) are all within approximately 3 miles (5km) of a so-called master fault (i.e. Red Mountain fault, San Cayetano fault, etc.). They point out that flexural-slip faults of Orcutt and Timber Canyons display relative movement opposite to that of the nearby active San Cayetano fault to the north. The interpretation that numerous reverse faults in the Ventura basin are related to flexural slip seems reasonable based on direct geologic evidence.

In the case of the Peralta Hills fault, there are similarities as well as dissimilarities to the prescribed flexural-slip model and associated geologic conditions in the Ventura basin. Among the similarities, the strike of the Peralta Hills fault essentially parallels the bedding planes with both dipping to the north. The Peralta Hills occupy the southern limb of a large broad syncline, where the same formations exposed there crop out 4 to 6 miles (6-10km) to the north, and attain a minimum depth of about 8,000 feet (2500m) beneath the Santa Ana River alluvial valley (Durham and Yerkes, 1965; Schoellhamer and others, 1981; Morton and Miller, 1981). In addition, movement has taken place along bedding, at least near the surface, for much of the Peralta Hills fault; but, displacement has to a limited extent occurred within massive sandstone (Topanga Formation) rather than less competent and stratigraphically higher laminated siltstone and shale beds (Puente Formation). There is no conclusive evidence to relate the recorded microseismicity to either flexural slip or relatively deep crustal displacement based on our cursory review of reported data.

Unlike Yeats and others' (1981) flexural-slip examples, the Quaternary surficial cover has been displaced exclusively along a single fault break although slickensided bedding surfaces were evident within the upthrown block at numerous sites. Vertical offset across the fault plane exceeded 60 feet (18m) at one location. On the Peralta Hills fault, considerable tilting of Quaternary deposits has been recognized at a large excavation. Strata dips southerly as much as 60°, and decreases to 40° for younger material more distant from the north-dipping fault. This sense of deformation is indicative of either drag within the downthrown block, or dominant thrusting effects by the upthrown block. Because of the nearby gravity sliding, we tend to favor the thrusting model. This condition is very similar to the Gould Canyon thrust fault, considered within the Sierra Madre fault zone, and reported by Crook and others (1978). There, the fault dips 30°-65° north into the hillside at depths, and abruptly dips 15°-25° southerly near the surface in one area. Based on mapping and exploration, this "thrust-rooted (gravity) slide" consists locally of 52 feet (16m) of diorite overlying old alluvium (Crook and others, 1978). Another noteworthy example is the Tujunga segment of the San Fernando fault zone where trenches revealed that the north-dipping fault plane was parallel with bedding, but became south-dipping near the surface (Barrows, 1974).

offset along bedding surfaces upthrown fault may be [illegible] against-like [illegible] evidence at SAP and SAR-7.

According to Yeats and others (1981), ground rupture along a flexural-slip fault may occur during a large-magnitude earthquake originating on a nearby "master" fault. The closest active fault is the Whittier fault, located approximately 6 miles (9km) to the north. It is a high-angle northeast-dipping reverse fault of probable Late Pleistocene to Holocene age (Morton and others, 1976). The Whittier fault is commonly considered a northern extension of the Elsinore fault; thus, the entire fault zone would have a total length of about 175 miles (280km). The Elsinore fault is a well-documented active fault with right-lateral separation on the order of 6 to 6.5 miles (9-11km), while the Whittier fault has about 2 miles (3km) of vertical separation and nearly one mile (1-2km) of right-lateral displacement (Kennedy, 1977; Weber, 1977; Yerkes, 1972). McGuire (1981) indicates that two types of motion are represented on the Elsinore fault. He points out that 17 to 19 miles (27-30km) of right-lateral offset took place prior to 4 million years ago, but that the fault exhibited reverse and normal separation during the last 4 million years. No doubt, the Whittier fault could qualify as a principal or "master" fault, particularly if the neotectonic setting is contributory to continued reverse (compressional) displacement; however, it might be too far removed from the Peralta Hills fault to have any local tectonic influence. Although the regional structural pattern may be consistent with the flexural-slip model, movement confined to only one fault surface during flexing seems difficult to explain for such a distance.

Still another possibility exists, namely, if the Peralta Hills fault is indeed a reverse fault, it may continue to the west at least in the subsurface, or faulting may have been translated into tighter folding. It is possible to reinterpret Schoellhamer and others' (1981) structure section E-L (Plate 2) by incorporating a north-dipping (40°-60°) reverse fault, projected from the western end of the mapped Peralta Hills fault, between their Richfield Oil Corporation Hamrick-Olive Well No. 1 and Olive Petroleum Company Well No. 1. Such a fault could be shown with 800 feet (245m) or more of stratigraphic separation by a reduction in the syncline-anticline amplitude, and without altering the subsurface structure based on the respective well logs. This would tend to explain the significant difference in depths of the formational contacts at each well location, and be consistent with the known near-surface structure in the immediate area. Additionally, selected microseismicity data are compatible with this projected fault plane, and the 60° dip previously suggested for such a fault.

The El Modeno fault has commonly been projected from its north-northwest trend to an east-west trend to explain the topography of the Peralta Hills (Yerkes, 1957; Morton and others, 1973). The position of the reverse fault suggests that the El Modeno fault continues its north-northwest trend and is truncated by or passes beneath the Peralta Hills fault (Fife and others, 1980; Ryan and others, 1982; Ryan, 1982). This would tend to support the Peralta Hills fault as an active deep-seated structural feature.

CONCLUSIONS

There is considerable uncertainty as to the significance of the Peralta Hills fault. This fault may be capable of producing a large-magnitude earthquake during rupture since certain aspects favor the deep-seated fault model interpretation. The Peralta Hills fault may be somehow related to the Transverse Ranges compressional stresses. Future seismic activity may be similar to other reverse faults within this region (i.e. San Fernando fault). Since the available data are still inconclusive, there is a need for more detailed research, including possibly the installation of a microseismicity network, review of oil and water well data, conducting geophysical surveys, additional trenching, core drilling, etc. If we assume that the currently known fault would undergo surface rupture along its entire length, the associated earthquake would be in the range of M6.0-M6.5 according to Greensfelder (1973). Considering this potentiality and the intense urbanization, more definitive information is required to determine the seismic risk of the area.

ACKNOWLEDGMENTS

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ENGINEERING EVALUATION OF THE

PETERS CANYON SIX MILLION GALLON RESERVOIR

Prepared for:

East Orange County Water District

Job No. EOCWD1.001.001

July 2012

Richard Brady & Associates
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Engineering Evaluation of the

Peters Canyon Six Million Gallon Reservoir

1. SCOPE

Originally constructed in 1963 the Peters Canyon Reservoir has a prominent distinction as the highest storage reservoir in the distribution system managed by the East Orange County Water District. In 1996 the reservoir suffered massive roof damage requiring removal and replacement of the entire roof structure. More recently a number of small issues have developed raising the question of the overall health of the reservoir and its future viability. In order to address these issues, the following are included in this report.

- Findings on the weather conditions that caused the original roof failure and an analysis of the current roof condition with respect to similar failure.
- Results of an internal inspection of the reservoir
- An evaluation of the existing structure for seismic and structural integrity, for compliance with the current building code.
- Results of a reservoir leakage test performed using a hook gauge.
- Results of attempts to locate the reservoir underdrains.
- Evaluation of the current venting scheme and the associated masonry wall spalling.
- An evaluation for mitigating destructive wind forces.
- Recommended short term improvements to deal with nuisance/cosmetic issues.
- Recommended long term improvements to extend the reservoir life for 50 more years.
- Cost estimates and schedules for the recommended improvements.
- Specifications for recommended short and long term improvements.

In addition to the text of this report, appendices included as a part of this report are as follows:

Appendix A	Facility Drawings
Appendix B	Photographs
Appendix C	Structural Calculations
Appendix D	Sketches of Recommended Structural Improvements
Appendix E	Cost Estimate for Improvements

2. BACKGROUND

The Peters Canyon Six Million Gallon Reservoir was originally designed and constructed circa 1963. The reservoir, situated on a hillside in East Orange County, Is located just south of Jamboree

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Road. The site can be reached by driving south on Handy Creek Road; a private access road which intersects with Jamboree Road between Tustin Ranch Road and East Chapman Avenue.

The basin consists of a 4” concrete slab with 6” thickened edges 1’ in width. Trapezoidal in cross section, the reservoir is 280’-4” in length and 170’-4” wide. The reservoir floor has a bottom elevation of 765’ and a slightly sloping roof with an effective depth of approximately 28 feet. A four feet high 8” masonry wall sits atop a continuous concrete footing, backfilled to reveal only 2’ above the finished grade. Reinforced concrete piers occur at 21’ on center along the north and south walls providing the primary structural support for the main roof beams. The roof is framed from Glu-Lam and conventional timbers with the original framing being constructed from redwood, and subsequent repairs using pressure treated lumber. The roofing is Zip-Rib aluminum roofing. There is no structural roof diaphragm. Excerpts of original construction drawings are provided in Appendix A.

A series of repairs and renovations took place in the 90’s starting with the installation of a liner in 1994. In 1996 portions of roofing were significantly damaged from high winds resulting in roughly 1/3 of the roofing requiring replacement. The roofing repairs were executed in 1997 with the entire roof being removed and reattached with improved hardware. A seismic assessment was conducted in 1999 which resulted in a seismic retrofit in 2000. The retrofit consisted of adding a rock anchors to the base of each concrete pier in order to provide increased resistance to seismic overturning perpendicular to the north and south walls.

3. INVESTIGATIONS AND OBSERVATIONS

a. Severe Weather Event and Roof Failure

According to record documents, the damaged reservoir roof was discovered upon the return of workers following the Thanksgiving holiday and a damage report was filed by Boyle Engineering on 4 December 1996. According to NOAA data archives a high wind event occurred in the week preceding the discovery. This event (reference number 251315) was of sufficient magnitude to blow down numerous trees and power lines, as well as dislodge a 70 feet steel beam from an orange county construction site. Sustained winds were documented at 35 – 45 mph, however a maximum recorded wind speed of 98 mph was observed during the event.

The original roof was found to have been assembled with carbon steel screws that have a high potential difference with the aluminum roof structure. This potential difference caused the retaining screws to rapidly corrode, resulting in a loss of strength. It is this loss of strength combined with a strong wind condition that led to the roof failure of 1996

For the purpose of load calculations a wind speed of 100 mph was assumed to have caused the damage to the roof. Using the wind force guidelines of ASCE 7-05 (the basis of the current building code) the resultant uplift at the corner of the structure was calculated at 36.1 psf. This is

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well within the allowable uplift for ZIP RIB style roofing. Current specifications for ZIP RIB roofing have a rating of UL-90 which corresponds to the ability to resist an uplift pressure of up to 90 psf. The available documentation for the existing roof does not contain any specification for the uplift resistance at the time of construction; however it is reasonable to assume that the performance of ZIP RIB roofing systems has not changed significantly over the subsequent 16 years.

The construction documents are very specific given the nature of the original construction error in that stainless steel screws were used to install the retaining clips for the entire roof system following the failure. Visual inspection of these screws is not possible without removing a roof panel, but the connections appear to be sound and in good condition. The only point of concern regarding the roofing system is the irregular condition of the ZIP RIB standing seams. The system is based on two adjoining pieces coming together at a standing retaining bracket. The mechanical retention of this bracket is solely dependent on the quality of the crimp applied to the roof panels. The crimps observed at the Peters Canyon Reservoir were bent and irregular resulting in the potential for mechanically weak connections. It should be noted that there are major concerns with the lack of a structural roof diaphragm, however see the structural evaluation below.

b. Reservoir Inspection

On June 5, 2012 an internal inspection of the reservoir was conducted by Lee Biggers, S.E., and Dan Cavanaugh, E.I.T., of Richard Brady & Associates. The photographs taken during this investigation are found in Appendix B.

Items of importance observed discovered during this inspection include the following:

- Corrosion on interior joist hangers and other strong tie style connectors.
- Condensation on the underside of the roof, with visible staining on beams and joists.
- Side wall vents in evidence not indicated in original construction drawings.
- A broken corner atop one of the concrete columns. (Grid B-13)
- Column top mounting nut, backed off and corroded (not galvanized). (Grid C-4)
- Corrosion on Frame Brace connecting bolts (not galvanized).
- Corrosion on hardware of lift out roof panel
- Drag Strut hardware not installed according to original plan (sheet 20 detail 3)
- Multiple direct to sunlight holes in the roof, obvious sources of water intrusion.

The general opinion of the inspection with respect to the physical condition of the structure framing was favorable with the above items noted.

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c. Reservoir Leakage Test

A hook gage was installed at the reservoir access door on June 5 2012, while the reservoir was isolated to perform a leakage test of the facility. Forty-five hours later the gage was removed and the reservoir was returned to service. A difference in level of 3/16" was noted at that time. A 3/16" change in water level in the reservoir corresponds to a loss of 560 cubic feet (cf) of water or 299 cf/day. Two un-isolable loads were identified during the test: the trailer located on site and the COX transmission facility just outside the property. The cox facility has an average metered usage of 1800 cf each month, or 60 cf/day. The trailer is un-metered, but serves as the residence for two adult individuals. The American Water Works Association (AWWA) estimates the average water usage by an individual to be 10 cf/day, for a total usage by the occupants of the trailer of 20 cf/day. After accounting for these loads the total net loss due to leakage and evaporation is reduced to 219 cf/day or 1638 gallons. AWWA guidelines for reservoir leakage restrict the allowable daily leakage for *new* concrete reservoirs to 0.05% of the total volume. In the case of the Peters Canyon Reservoir the allowable daily leakage is 3000 gallons with an observed leakage of 1638 gallons.

d. Underdrains

Due to unstable slope materials and reports of rattlesnakes in the area the search for the underdrains was limited to probing the suspected areas of the hillside. By physical observation it has been confirmed that the mulch lining the hillside is at least 30 inches in depth. It is suspected that the drains are buried near the surface under this mulch. No surface water, wetness, or abnormally lush foliage was found in the area of either the north or south underdrain. Based on this investigation it has been determined that the leakage from the reservoir is reasonably low and likely to be near the areas of the outlet or absorbed directly into the ground beneath.

e. Reservoir Vents

The reservoir is equipped with multiple vents both in the walls and the top of the roof ridgeline. Generally, a reservoir is only required to possess enough vents to prevent a siphon lock during rapid draining of the reservoir. As originally designed the reservoir was provided with 278 square feet of vent screening. This air exchange area was later increased by adding 45 square feet of venting that is not shown on existing plans. No additional vents are necessary to achieve necessary air exchange; in fact fewer may be beneficial in protecting water quality from insect and pollen intrusion.

Internal inspection resulted in the discovery that all internal masonry wall spalling was located at the position of the added vents. It is apparent that the cause of the spalling degradation of the perimeter masonry walls around the vents is related to the construction process used to install

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the new vents into the existing structure. The direction of spalling indicates it is careless construction that weakened the surfaces surrounding the vents as the original concrete masonry units had to be knocked out in order to create a space to install each vent screen.

4. STRUCTURAL EVALUATION

The objective herein is to review the condition of the subject reservoir with respect to compliance with the current building code (The 2010 California Building Code (CBC)). The observations made during the recent internal investigation (by boat) of the existing structural frame revealed that the physical condition, in general, of all structural elements is good. Minor corrosion of metal framing connectors was noted. Some minor spalling of concrete was noted.

The condition of the reservoir with respect to resisting earthquake induced (seismic) loading as well as wind loading is deficient with respect to the CBC. Actually, the existing structure has never been compliant with any current building code during its entire existence due to the absence of a functioning roof diaphragm. The metal roofing now in place and throughout the life of the structure has little or no diaphragm shear capacity and is not designed or tested for this capability. Specifically, this deficiency relates to the lack of a means to transfer seismic inertial loads (as well as wind loading) from the tributary roof mass as well as that tributary from interior concrete columns and perimeter masonry walls to vertical force resisting elements which include the perimeter masonry shear walls and the interior concrete braced frames. The above described deficiency condition was aggravated by the opening up of the roof framing along the roof ridge to accommodate the installation of a roof centerline venting system. This alteration eliminated the ability of the concrete braced frames along the roof centerline to provide any lateral resistance to seismic or wind loads.

An analysis is provided in Appendix C to determine the viability of adding a functional roof diaphragm without adding to or altering the existing structural frame, i.e. timber framing, concrete columns and masonry perimeter walls. The major change will consist of removal of the existing metal roofing and adding wood structural panel sheathing (exterior grade plywood) to the entire roof. Steel horizontal truss members will be added along the open roof centerline to provide for diaphragm continuity. These proposed improvements are indicated by the sketches in Appendix D.

During the installation of a new roof diaphragm, corroded framing connectors can be replaced; spalled concrete columns can be repaired as well as making other observed minor structural repairs. A system of adhesive anchors will need to be installed to provide for shear transfer from the roof diaphragm to the perimeter masonry shear walls. A sprayed polyurethane foam (SPF) roofing system will be applied to the wood panel substrate which will eliminate the vulnerability of a metal roofing system to corrosion, deterioration, leakage and wind uplift. This system needs no fasteners, insulation joints, counter flashings or membrane seams.

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5. CONCLUSIONS

Based on the observations and findings discussed above and a general comparison with current building code requirements relating to seismic resistant design, the following conclusions have been reached regarding the property condition of the Peters Canyon Six Million Gallon Reservoir.

a. Roofing

Condition: Fair
The aluminum cladding is not likely to suffer a repeat of the wind event that occurred in 1996, however multiple leaks are evident and the overall system fails to provide the necessary structural diaphragm that the structure requires. Roofing repairs are necessary to exclude water entry into the reservoir.

b. Structural

Material Condition: Good
The existing structural members are in generally good condition and can be expected to remain sound as long as current general conditions persist. Inspections should be conducted annually to detect any rot or damage at the ends of each beam and at the interior hinge connections mid span.

Regulatory Compliance: Poor
The structure does not meet current building code standards for seismic analysis. The addition of a structural diaphragm to the existing structure will bring the structure into code compliance. Some connections included in the construction drawings are not incorporated into the existing structure.

c. Reservoir

Condition: Good
The observed leakage from the reservoir is within allowable limits. Some intrusion of debris and pollen was observed but this could be reduced to a minimum with simple maintenance and repairs.

6. RECOMMENDATIONS

a. Immediate Improvements

Structural Diaphragm Installation: A diaphragm of 23/32" wood panel sheathing should be installed over the entire roof surface to meet current seismic code requirements.

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Structural Upkeep: Bolt tightening and replacement of non-conforming hardware will greatly contribute to extending the longevity of the reservoir system.

Structural connectors: It is recommended that the drag strut joists along every bent line be reattached with connectors as specified in the original construction drawings.

Venting and weather sealing: Re-screening the reservoir vents with a tighter mesh screen material may help alleviate some of these problems over time. Much of the existing weather seal also appeared dry and cracked allowing insect and foreign matter intrusion.

b. Long Term Recommendations

Roof insulation: It is possible to insulate the roof with a spray on polyurethane foam (SPF) to provide multiple benefits to the reservoir system. First, the spray foam will act as a monolithic barrier to water intrusion at all roof seams and penetrations. Second, the foam will insulate the structure lowering the internal temperature, thus lowering amount of internal condensation forming on wood and metal structures inside the reservoir and reduce overall evaporation.

7. QUALIFICATIONS

The opinions expressed in this report are based on limited visual observations, a review of available as-build construction documents, inquiries with Water District staff and the application of engineering experience and judgment. No physical testing was performed to determine the adequacy of the reservoir's structural elements or their compliance with current building code requirements. Structural calculations and recommended improvements are of a preliminary nature and in no way represent a final, complete design. There is no claim, either stated or implied, that all conditions were observed. Neither is there any warranty expressed or implied as to the condition of the structure.

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Respectfully Submitted:

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R. L. Biggers, SE 1825

Dan Cavanaugh, EIT 141205

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APPENDIX A

FACILITY DRAWINGS

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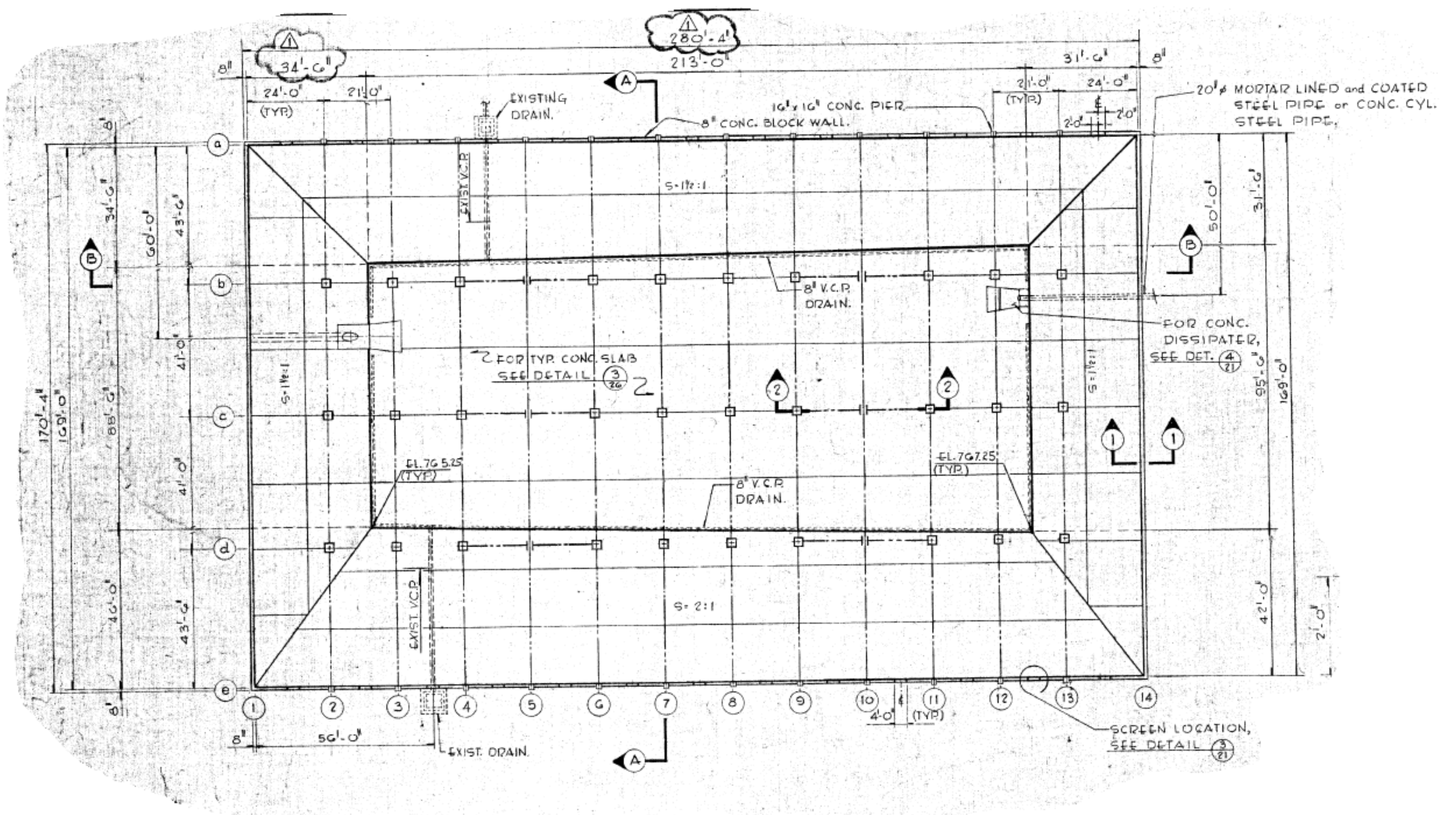
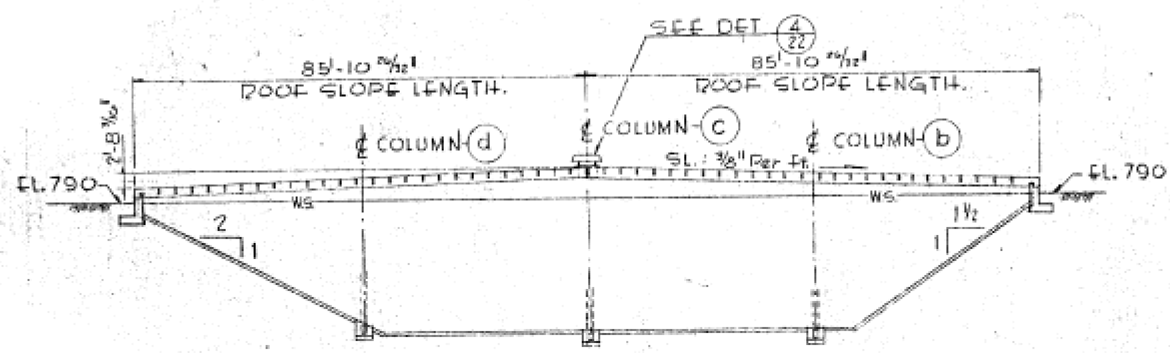


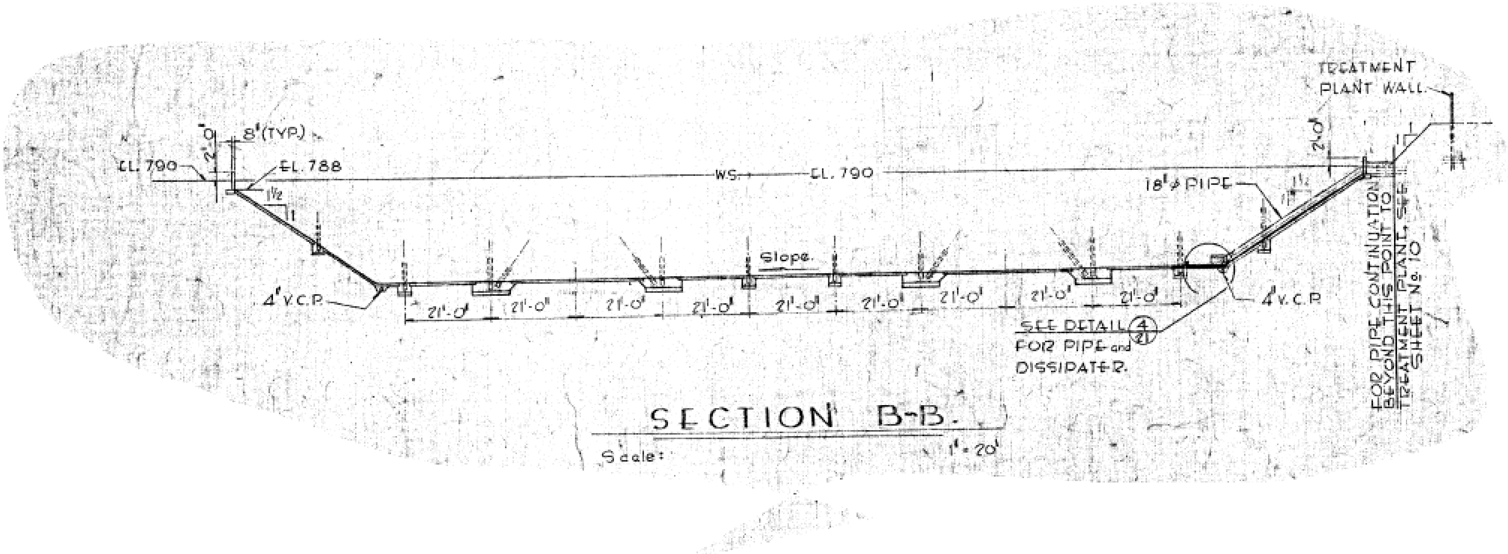
PLAN

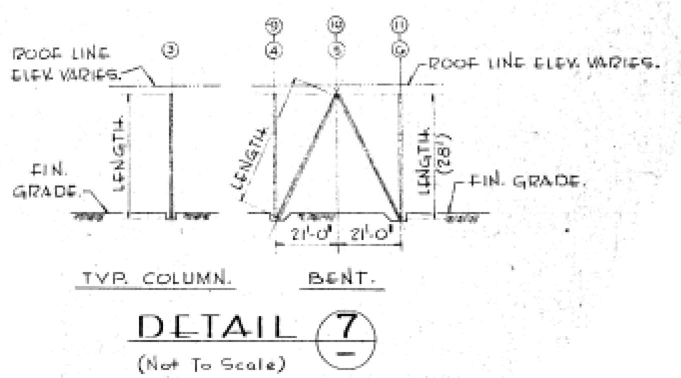
Scale: 1" = 20'

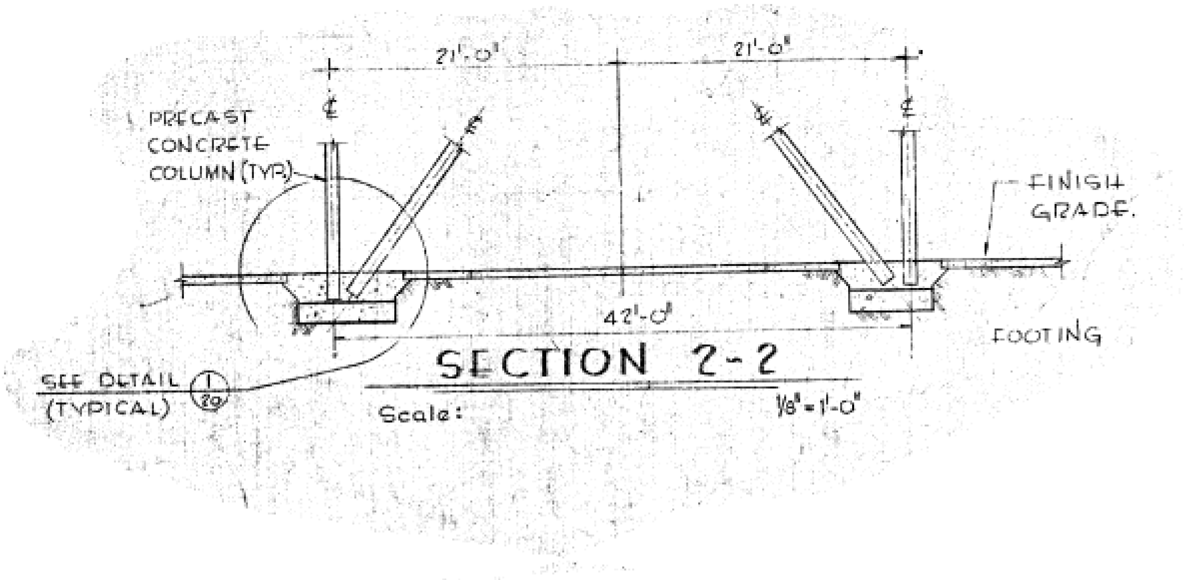


SECTION A-A

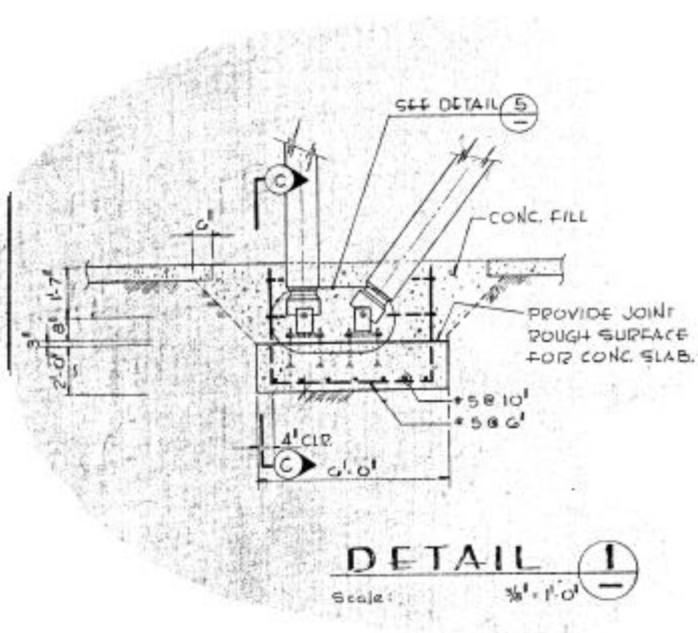
Scale: 1" = 20'


TREATMENT PLANT WALL
2'-0"
8" (TYP.)
EL. 790
EL. 788
WS
EL. 790
18" ϕ PIPE
Slope
4" V.C.P.
21'-0" 21'-0" 21'-0" 21'-0" 21'-0" 21'-0" 21'-0" 21'-0" 21'-0"
4" V.C.P.
SEE DETAIL 4/21 FOR PIPE and DISSIPATER.
FOR PIPE CONTINUATION BEYOND THIS POINT TO TREATMENT PLANT, SEE SHEET No. 10
SECTION B-B.
Scale: 1" = 20'

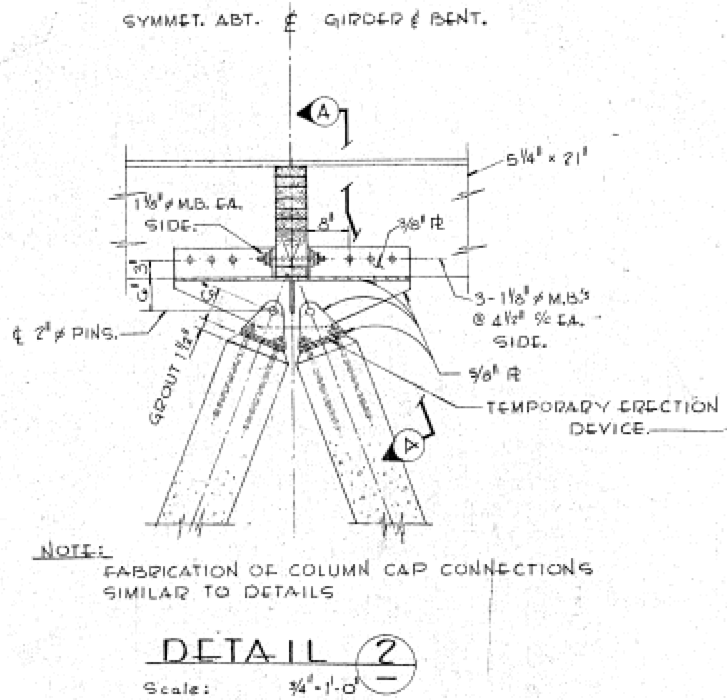

ROOF LINE ELEV. VARIES.
ROOF LINE ELEV. VARIES.
LENGTH
LENGTH
LENGTH (28')
FIN. GRADE.
FIN. GRADE.
21'-0" 21'-0"
TYP. COLUMN.
BENT.
DETAIL 7
(Not To Scale)

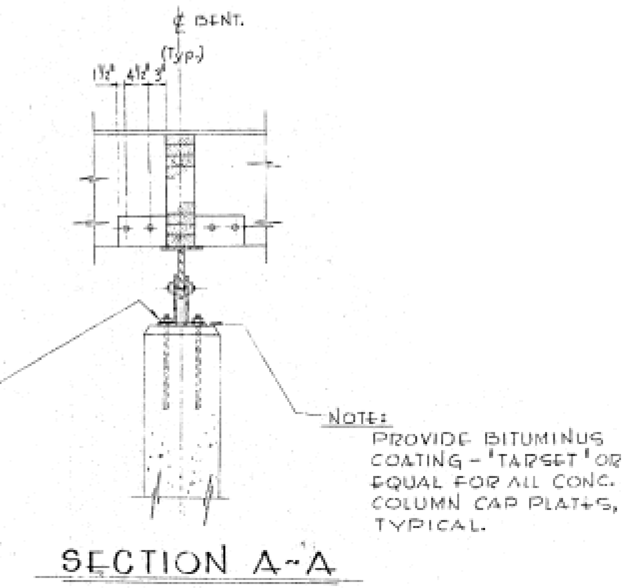


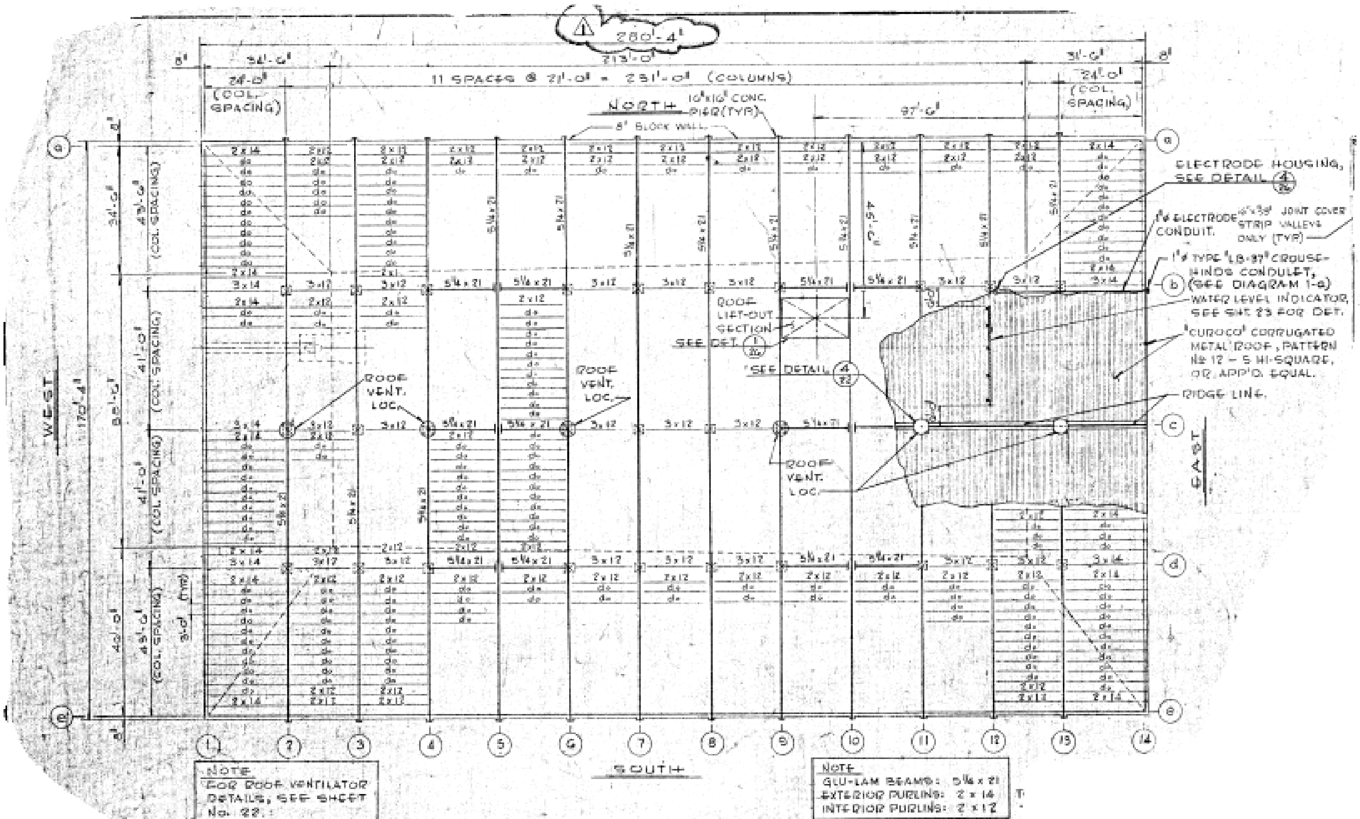
21'-0"
21'-0"
PRECAST
CONCRETE
COLUMN (TYP.)
FINISH
GRADE.
42'-0"
FOOTING
SEE DETAIL
(TYPICAL)
SECTION 2-2
Scale:
1/8" = 1'-0"



SEE DETAIL
CONC. FILL
PROVIDE JOINT
ROUGH SURFACE
FOR CONC. SLAB.
#5 @ 10"
#5 @ 6"
4" CLR
DETAIL
Scale:


SYMMET. ABT. ℄ GIRDER & BENT.
5 1/4" × 21"
1 1/8" ∅ M.B. EA. SIDE.
8"
3/8" PL
3"
6"
3 - 1 1/8" ∅ M.B.'S @ 4 1/2" C/C EA. SIDE.
℄ 2" ∅ PINS.
GROUT 1 1/2"
5/8" PL
TEMPORARY ERECTION DEVICE.
NOTE:
FABRICATION OF COLUMN CAP CONNECTIONS SIMILAR TO DETAILS
DETAIL 2
Scale: 3/4" = 1'-0"

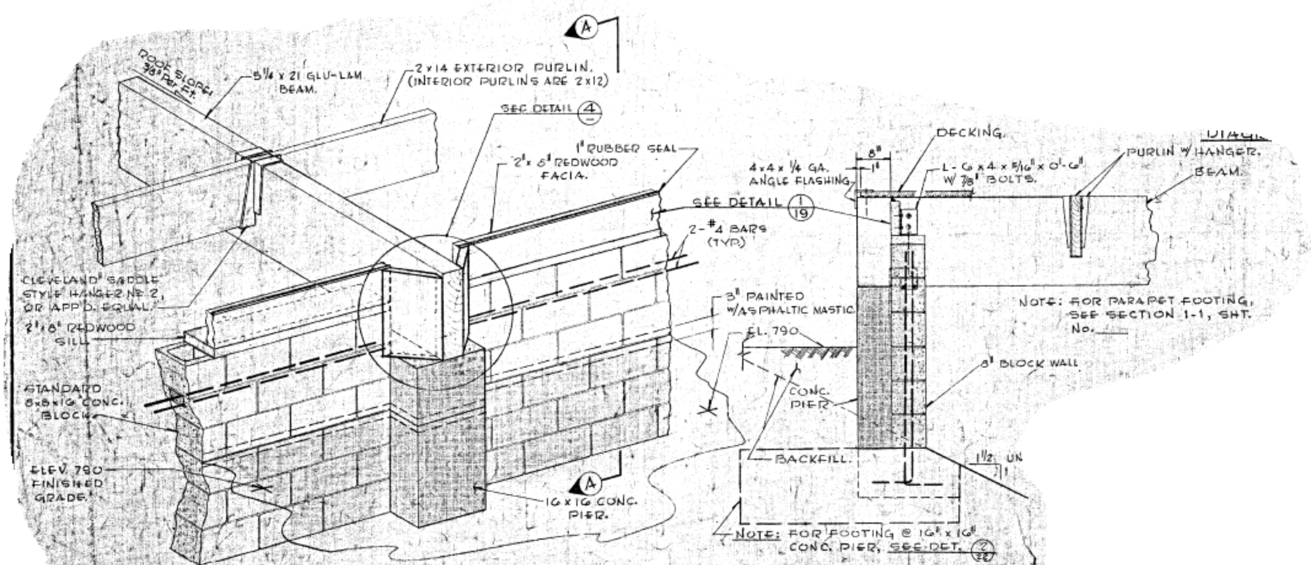

℄ BENT.
(TYP.)
1 1/2" 4 1/2" 3"
NOTE:
PROVIDE BITUMINUS COATING - "TARSET" OR EQUAL FOR ALL CONC. COLUMN CAP PLATES, TYPICAL.
SECTION A-A



ROOF FRAMING PLAN

DETAIL 1

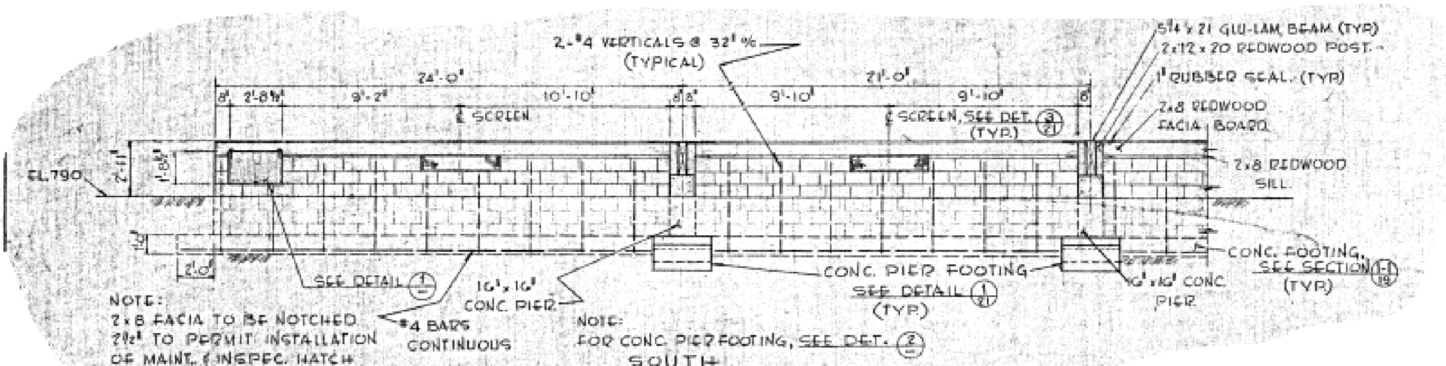
Scale: 1" = 20'-0"



RESERVOIR PARAPET & ROOF TIMBER ILLUSTRATED SECTION.

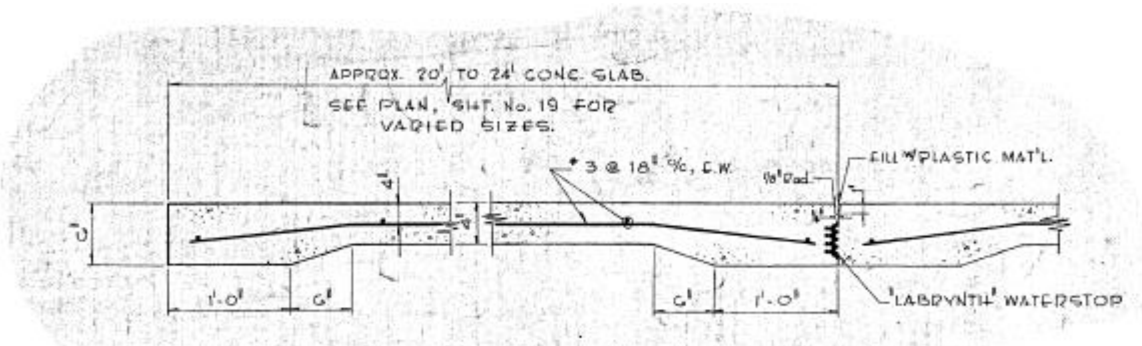
SECTION A-A

Scale: 1/4" = 1'-0"



SOUTH
ELEVATION

Scale: ¼" = 1'-0"



TYPICAL RESERVOIR SLAB and CONSTRUCTION JOINT.

DETAIL 3

Scale: 1½" = 1'-0"

APPENDIX B

PHOTOGRAPHS

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EOCWD PETERS CANYON RESERVOIR

NORTH WALL

EAST WALL

SOUTH WALL 1

SOUTH WALL 2

WEST WALL

OVERALL ROOF LOOKING WEST

TYPICAL ROOFING / GIRDER PILASTER

TYPICAL ROOFING

THERMAL SHRINKAGE EFFECTS ON SOUTH WALL 1

THERMAL SHRINKAGE EFFECTS ON SOUTH WALL 2

UNDERSIDE OF ROOFING

ROOF FRAMING AT CENTERLINE OF RESERVOIR

POST CONSTRUCTION INSTALLED VENT

TYPICAL GLULAM AND LINER

TYPICAL PRECAST CONCRETE COLUMN

TYPICAL EAST AND WEST WALL CONSTRUCTION

CONCRETE COLUMN WITH BROKEN CORNER

TYPICAL EAST-WEST BRACED FRAME

COROSION ON LIFT OUT ROOF PORTAL

TYPICAL E/W WALL FRAMING

ROOF VENT FRAMING

COLUMN WITH LOOSE NUT (NON-GALVANIZED)

ROOF VENT FRAMING

TYPICAL BRACED FRAME CONNECTION
(CORRODED NUTS AND BOLTS)

RESERVOIR WATER LEVEL CHANGE OVER 45 HOUR PERIOD

APPENDIX C

STRUCTURAL CALCULATIONS

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BRADY

Project EOCWD 6 MG RESERVOIR Job No. EOCWD1.001.001

Description

LOADS

GRAVITY

ROOF:

ROOFING	-	1.0	PSF
INSTALL 23/32" P.I. 48/24 SHEATHING	-	2.1	
2x14/2x12 PURLINS @ 36" O.C.	-	1.7	
5 1/4 x 21 GLB @ 41'	-	0.9	
5 1/4 x 21 GLB @ 21'	-	1.8	
SUB-TOTAL TO ROOF FRAMING	-	7.5	PSF

TRIBUTARY CONC. COL'S:

$$\frac{14' \times 1^{\square\prime} \times 150\ PCF \times 30\ EA.}{280' \times 170'} = 1.3\ PSF$$

TRIBUTARY 8" CMU WALLS:

$$\frac{4' \times 83\ PSF}{170'} = 2.0\ PSF$$

TRIBUTARY MISCELLANEOUS = 0.2 PSF

TOTAL WEIGH TRIBUTARY TO ROOF = 11.0 PSF

By RLB Date 07/12 Checked Date Sheet C1/9

BRADY

Project EOCWD 6 MG RESERVOIR Job No. EOCWD1.001.001

Description

LOADS

SEISMIC

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.1 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

By RLB Date 07/12 Checked Date

Sheet C2 / 9

LOADS

SEISMIC

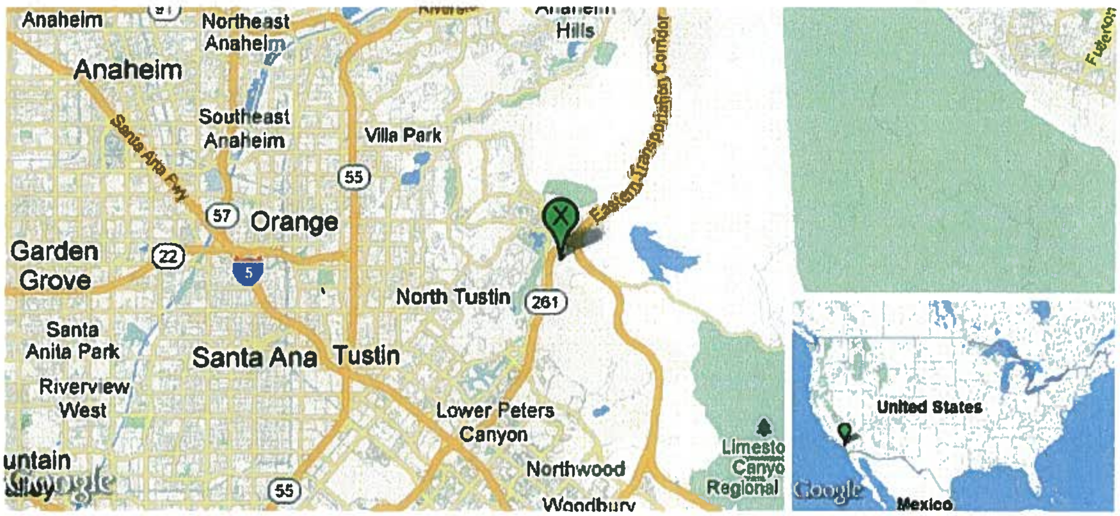
Report Title EOCWD 6 MG Reservoir
Fri July 20, 2012 20:16:31 UTC

Building Code Reference Document ASCE 7-10 Standard
(which makes use of 2008 USGS hazard data)

Site Coordinates 33.7756°N, 117.7542°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category IV (e.g. essential facilities)



USGS-Provided Output

S_S = 1.500 g	S_{MS} = 1.500 g	S_{DS} = 1.000 g
S_1 = 0.581 g	S_{M1} = 0.871 g	S_{D1} = 0.581 g

FORCE RESISTING SYSTEMS:

LONGITUDINAL DIRECTION (EAST-WEST) - ORDINARY CONCRETE BRACED FRAME, R = 3 (GOVERNS)
- SPECIAL REINFORCED MASONRY SHEAR WALLS, R = 5

$$C_{s_1} = \frac{S_{DS}}{R/I} = \frac{(1.00)(1.5)}{3} = 0.50$$

$$C_{s_2} = \frac{(1.00)(1.5)}{5} = 0.30$$

TRANSVERSE DIRECTION (NORTH-SOUTH) - SPECIAL REINFORCED MASONRY SHEAR WALLS, R = 5

$$C_s = \frac{(1.00)(1.5)}{5} = 0.30$$

SEISMIC ANALYSIS

GENERAL OVERVIEW

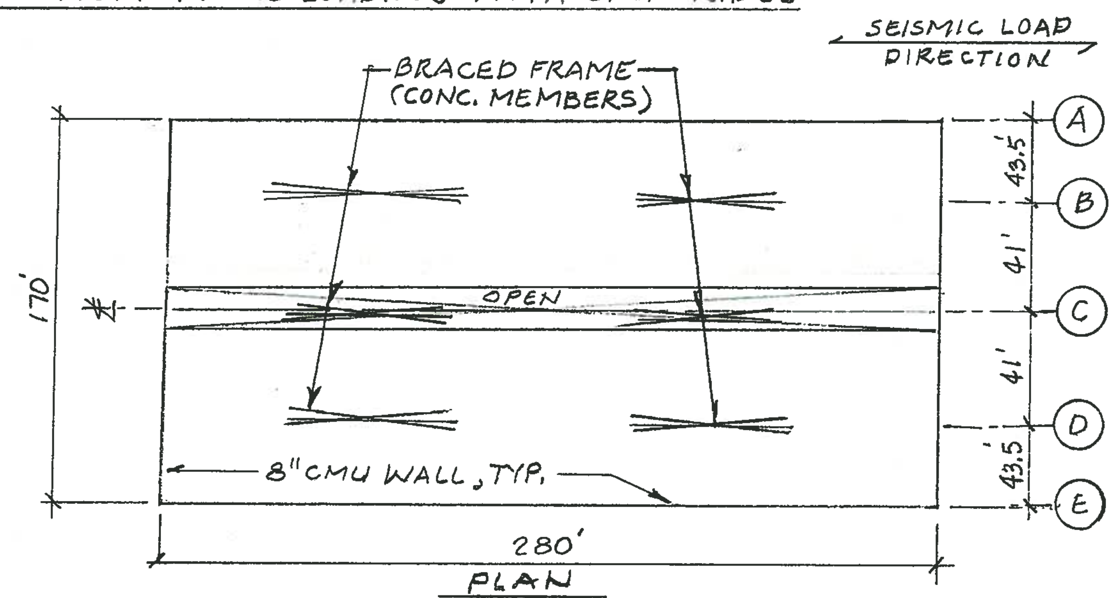
1. An obvious immediate conclusion is that the subject structure is completely deficient in that it has no viable functional roof diaphragm or other horizontal lateral force transfer system. There is no structural system for delivering lateral loads to the vertical lateral force resisting elements (masonry shear walls, braced frames, etc.).

2. If a flexible roof diaphragm is installed (plywood sheathing) leaving the roof in its current configuration with continuous open venting along the ridge (as indicated on pages B12 and B13 of Appendix B) effectively separating the reservoir into two separate structures, the overall structure will remain significantly deficient as is analytically indicated below.

3. An analytical check will be made below to determine the viability of a continuous sheathed roof diaphragm which utilizes a horizontal steel truss to transfer loads in the open vent area along the ridge.

SEISMIC ANALYSIS

CHECK LONGITUDINAL LOADING WITH OPEN RIDGE



PLAN

NOTE: THE FRAMES ALONG GRID C ARE OF NO VALUE SINCE THERE IS NO MEANS TO TRANSFER SEISMIC LOADS TO THEM.

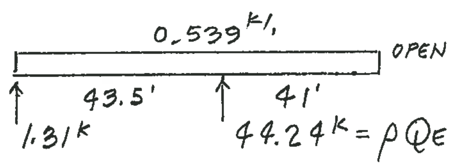
LOAD TO TYPICAL FRAME ALONG GRIDS B AND D:

$V = C_s W = (0.50)(11.0\ PSF) = 5.5\ PSF$

ALLOWABLE STRESS DESIGN GOVERNING LOAD COMBINATION:

$(0.6 - 0.14 S_{DS})D + 0.7\rho Q_E \quad , \quad \rho = 1.0$

LOAD TO FRAME $= 0.7\rho Q_E = (0.7)(1.0)(140')(5.5\ PSF) = 0.539^{k/\prime}$

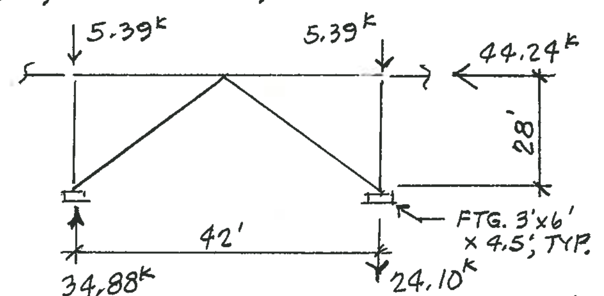


TRIBUTARY COL. DL:

$P_{DL} = (1.5)(21')(0.5)(43.5' + 41')(8.8\ PSF)$

$= 11.71^k = D$

$(0.6 - [0.14][1.0])D = 5.39^k$



FTG. WT. $= 3' \times 6' \times 4.5' \times 0.15\ KCF = 12.15^k$
$\ll 24.10^k$

BY INSPECTION, FRAMES WILL OVERTURN

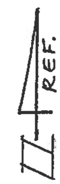
BRADY

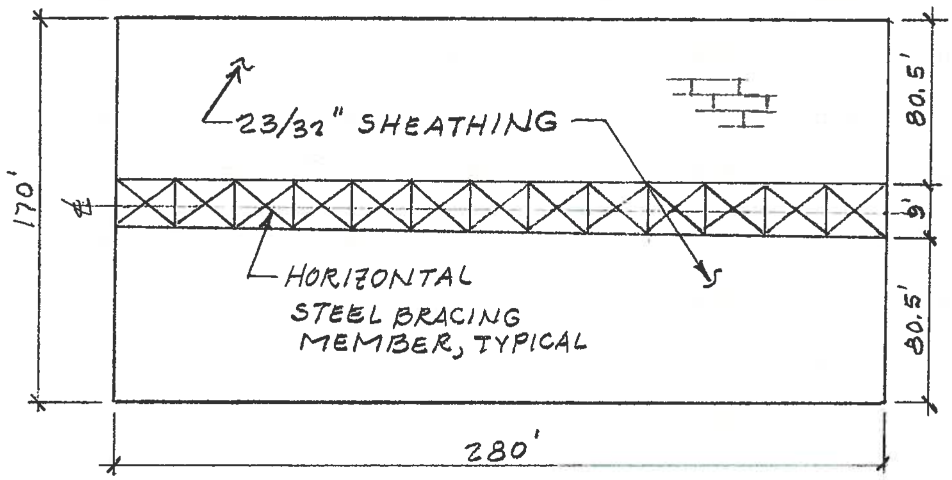
Project EOCWD 6MG RESERVOIR Job No. EOCWD1.001.001

Description

SEISMIC ANALYSIS

CONTINUOUS ROOF DIAPHRAGM





PLAN

DIAPHRAGM SHEATHING: REF. 2010 CBC TABLE 2306.2.1(1)

UTILIZE ONLY PERIMETER MASONRY SHEAR WALLS AND DESIGN AS A FLEXIBLE DIAPHRAGM.

$C_S = 0.30$ IN BOTH DIRECTIONS

$$\Sigma F_{i\,ASD} = (170')(280')(11.0\,PSF)(0.30)(0.7) = 109.956^k$$

$$F_{px} = \frac{\Sigma F_i}{\Sigma w_i} w_{px} = 109.956^k$$

MAX. LONGITUDINAL SHEAR = $v_{E\text{-}W} = \frac{(0.5)(109.956^k)}{280'} = 0.196^{k/\prime} < 0.215^{k/\prime}$ CASE 3 UNBLOCKED

MAX. TRANSVERSE SHEAR = $v_{N\text{-}S} = \frac{(0.5)(109.956^k)}{161'} = 0.341^{k/\prime} < 0.425^{k/\prime}$ CASE 1 BLOCKED

CHECK, TRANSVERSE SHEAR @ 24' FROM EDGE:

$$v_{N\text{-}S} = \left(\frac{116'}{140'}\right)(0.341^{k/\prime}) = 0.283^{k/\prime} < 0.285^{k/\prime}$$ DISCONTINUE BLOCKING

MAX. DIAPHRAGM CHORD FORCE = $\frac{(109.956^k)(280')}{(8)(170')} = 22.64^k$

REQ'D STEEL STRAP AREA = $\frac{22.64^k \times 1.67}{36\,KSI} = 1.05^{\square''}$

NO OF 7/8" SPLICE BOLTS REQ'D IN 3/8" STRAP = $\frac{22.64^k}{8.11^k} = 2.79 \rightarrow$ 3 EA. SIDE

By RLB Date 07/12 Checked Date

Sheet C6, 9

SEISMIC ANALYSIS

CONTINUOUS ROOF DIAPHRAGM

CHORD STRAP:

$$A_{g\,MIN.} = \frac{1.05^{\square''}}{0.85} = 1.24^{\square''}$$

$$\text{HOLE AREA} = (1'')(0.375^{\square''})$$

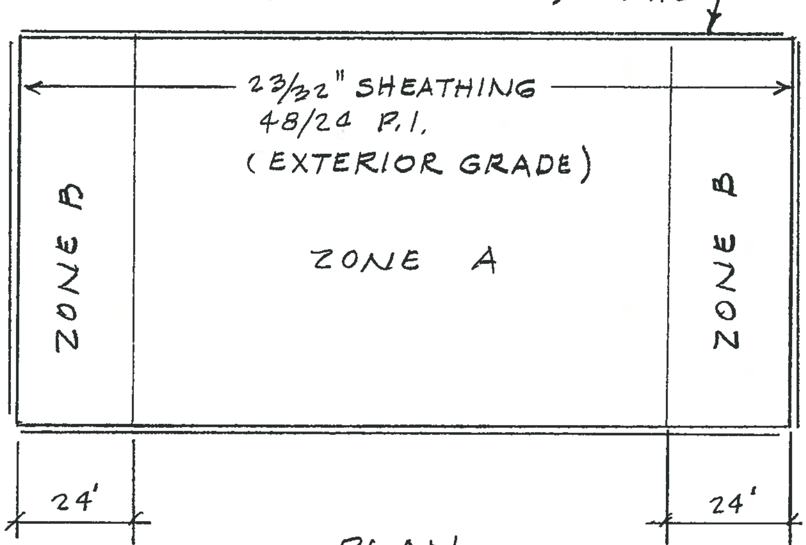
$$\therefore A_{g\,REQ'D.} = 1.05^{\square''} + 0.375^{\square''} = 1.425'' \longrightarrow \text{PL } 3/8'' \times 4'' \quad A_g = 1.50^{\square''}$$

$$A_e = A_n U = (1.50^{\square''} - 0.38^{\square''})(1.0) = 1.12^{\square''}$$

$$\text{YIELDING} \quad \frac{P_n}{\Omega_t} = \frac{(36\text{KSI})(1.50^{\square''})}{1.67} = 32.34^k > 22.64^k, \text{ O.K.}$$

$$\text{RUPTURE} \quad P_n = \frac{(58\text{KSI})(1.12^{\square''})}{2.00} = 32.48^k > 22.64^k, \text{ O.K.}$$

STRAP 3/8" x 4" SPLICED W/ 3-7/8" A307 BOLTS
EA. SIDE OF ADJOINING PIECES, TYPICAL



PLAN

SHEATHING NAILING:

ZONE A – 10d (3" x 0.148") @ 6" MAX. AT
SUPPORTED EDGES

ZONE B – 10d (3" x 0.148") @ 4" O.C. ALONG
CONTINUOUS EAST-WEST EDGES
@ 6" ALONG NORTH-SOUTH EDGES
ALL UNSUPPORTED EDGES BLOCKED

SEISMIC ANALYSIS

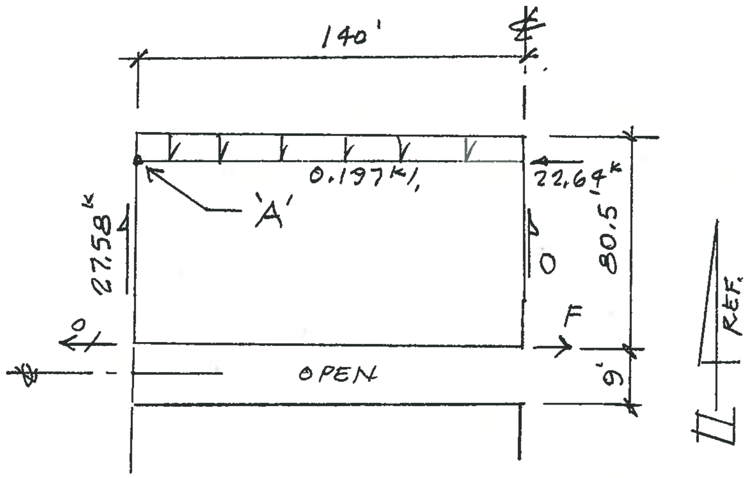
CONTINUOUS ROOF DIAPHRAGM

DIAPHRAGM OPENING:

$\Sigma M @ 'A' = 0$

$$F = \frac{(0.197^{k/'})(140')^2(0.5)}{80.5'}$$

$= 23.98^k \rightarrow 24.0^k$



PARTIAL PLAN

HORIZONTAL TRUSS:

TRUSS CHORD IN TENSION AND COMPRESSION;

$KL_{MAX.} = (1)(21'-3') = 18'$

$\angle 6 \times 6 \times 1/2$, $KL = 18'$, $\frac{P_n}{\Omega_c} = 25.9^k > 24.0^k$, O.K.

USE $\angle 6 \times 6 \times 1/2$ CHORDS

TRUSS DIAGONAL IN TENSION ONLY;

$$P_{MAX.} = \frac{\sqrt{(42')^2 + (9')^2}}{42} \times 24.0^k = 24.5^k$$

$$A_{NET\ REQ'D.} = \frac{P \times \Omega}{F_u} = \frac{24.5^k \times 2.00}{58\ ksi} = 0.84^{\square''}$$

$\angle 4 \times 4 \times 3/8$ W/CLIPPED LEG $= (3.5'')(0.375'') - (0.375'' \times 1'') = 0.94^{\square''} > 0.84^{\square''}$ O.K.

USE $\angle 4 \times 4 \times 3/8$ DIAGONALS

BOLTS; A325 7/8", 2 EA. CONN.

SHEAR – 2 EA. x $16.2^k = 32.4^k > 24.5^k$, O.K.

BEARING – 2 EA. x $60.9^k \times 0.375 = 45.7^k > 24.5^k$, O.K.

TENSION – 2 EA. x $27.1^k = 54.2^k > 24.5^k$, O.K.

USE 7/8" A325 BOLTS

BRADY

Project EOCWD 6MG RESERVOIR Job No. EOCWD1.001.001
Description

SEISMIC ANALYSIS

CONTINUOUS ROOF DIAPHRAGM

8" MASONRY SHEAR WALLS:

MAX. CONDITION AT EAST AND WEST (TRANSVERSE) WALLS

$V_{MAX.} = (1.5)(109.956^k)(0.5) = 82.467^k$ (54.978^k FOR OVERTURNING)

$f_{v\,MAX.} = \dfrac{82.467^k}{(7.63'')(167.33')(12''/')} = 0.0054\text{ KSI} = 5.4\text{ PSI}$

$F_v = \frac{1}{2}\left[4 - \left(\frac{M}{Vd}\right)\right]\sqrt{f'_m} = 0.5\left[4 - \dfrac{5.33'V}{167.33'V}\right]\sqrt{1500\text{ PSI}} = 76.8\text{ PSI} >> f_v$ O.K.

OVERTURNING ON MASONRY:

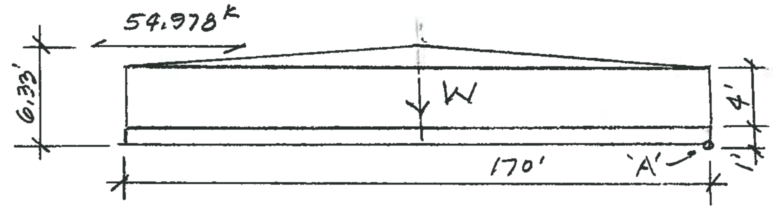
$M_{AVG.} = (5.33')(54.978^k) = 293.0'^k$

$n\rho = \dfrac{(21.48)(0.80^{\square''})}{(7.63'')(167.33')(12''/')} = 0.0011$, $2/kj = 46.4$ $j = 0.98$

$f_s = \dfrac{M}{A_s j d} = \dfrac{(293.0'^k)}{(0.8^{\square''})(0.98)(167.33')} = 2.23\text{ KSI} < 20\text{ KSI}$, O.K.

$f_m = \dfrac{M}{bd^2}\,2/kj = \dfrac{(293.0'^k)(46.4)}{(7.63'')(167.33')^2(12''/')} = 0.0053\text{ KSI} = 5.3\text{ PSI} << f'_m/3$, O.K.

OVERTURNING ON FOOTING:



$O.T.M.\ @\ 'A' = (6.33')(54.978^k) = 348'^k$

$W = \dfrac{(4')(170')(83\text{ PSF}) + (1')(2')(170')(150\text{ PCF})}{1000} = 107.4^k$

$(0.6 - 0.14S_{DS})D = (0.46)(107.4^k) = 49.4^k$

$R.M.\ @\ 'A' = 49.4^k \times 170' \times 0.5 = 4199'^k >> O.T.M. = 348'^k$, O.K.

8" MASONRY PERIMETER SHEAR WALLS ARE O.K.

By RLB Date 07/12 Checked Date

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APPENDIX D

SKETCHES OF RECOMMENDED

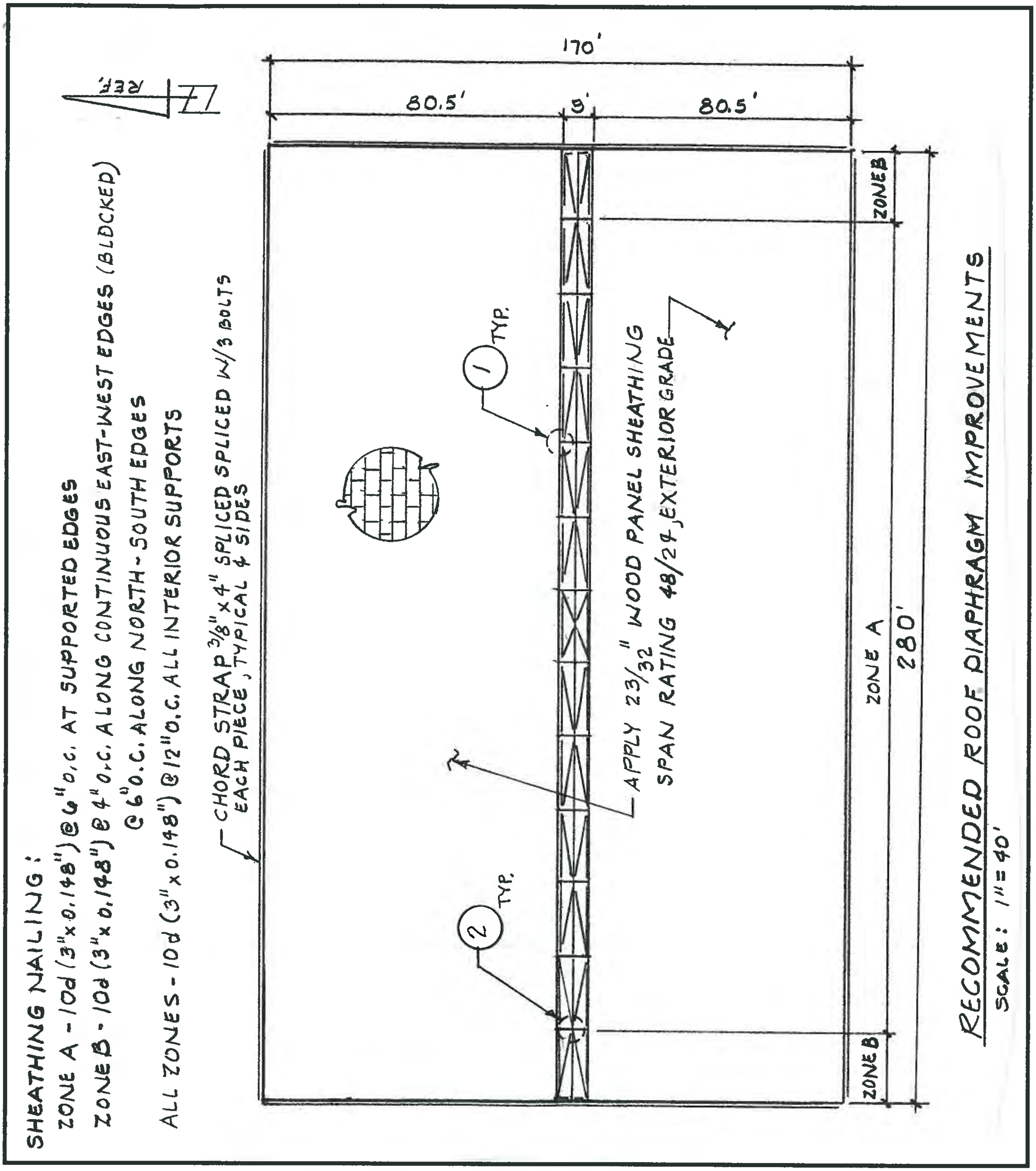
STRUCTURAL IMPROVEMENTS

3710 Ruffin Road
San Diego CA, 92123
Tel: 858.496.0500
Fax: 858.496.0505
www.richardbrady.com

BRADY

Project EOCWD 6MG RESERVOIR Job No. EOCWD.001.001

Description
ROOF DIAPHRAGM



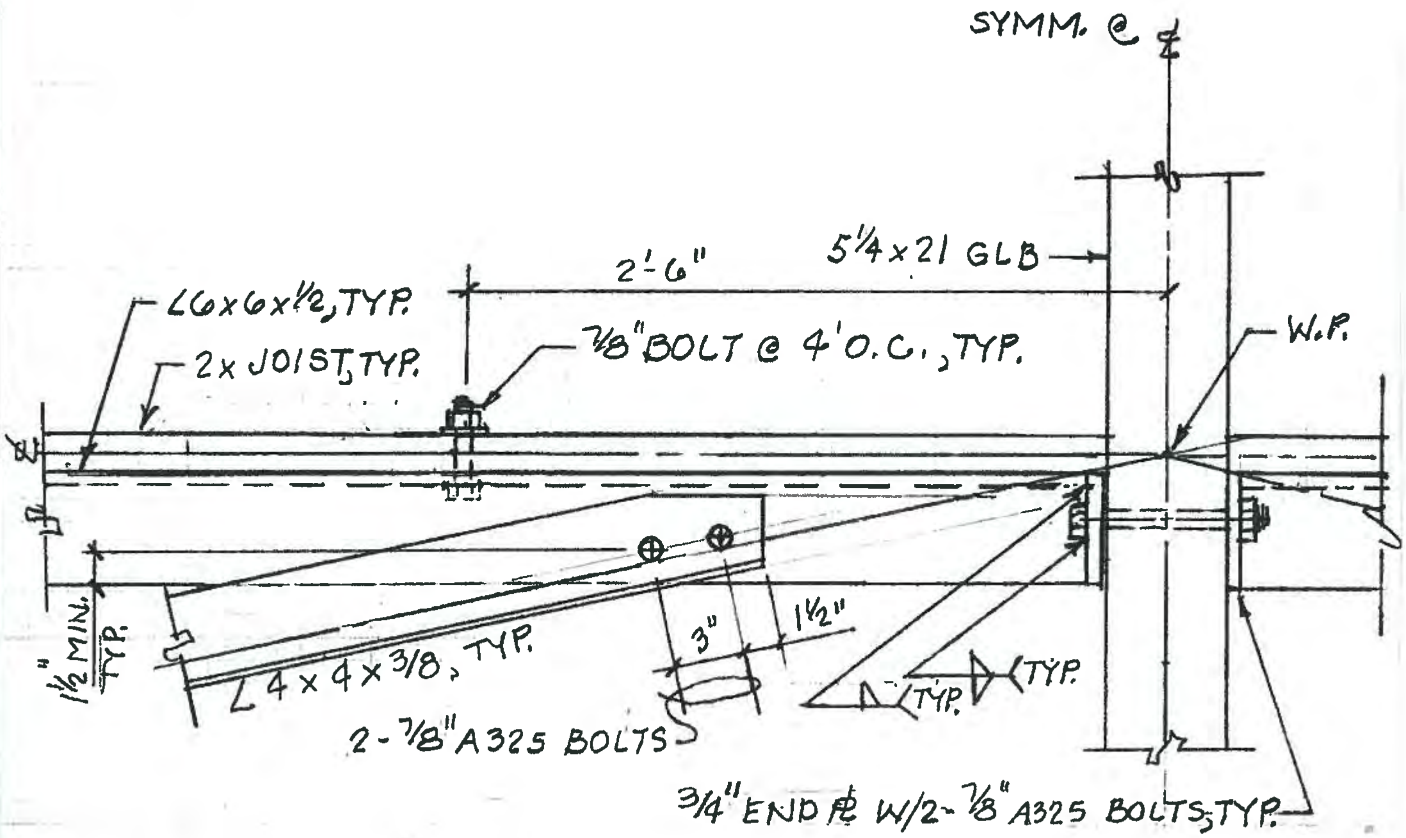
By RLB Date 07/12 Checked Date

Sheet D/1

BRADY

Project EOCWD 6 MG RESERVOIR Job No. EOCWD 1.001.001

Description DETAIL 1



NOTE: ALL STEEL ELEMENTS SHALL BE GALVANIZED IN CONFORMANCE WITH ASTM A123 AND A153.

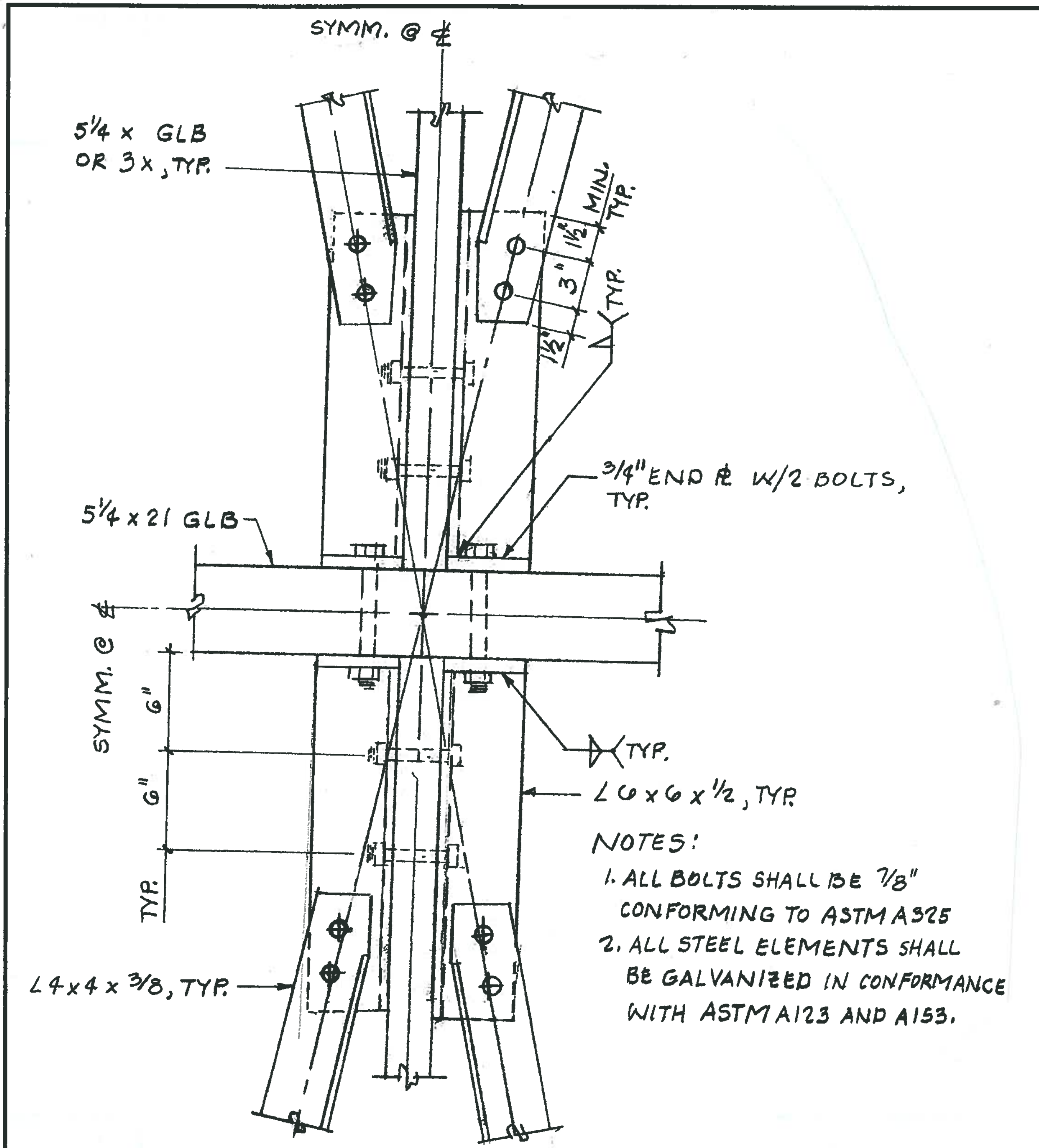
By RLB Date 07/12 Checked ______ Date ______ Sheet D / 2

BRADY

Project EOCWD 6MG RESERVOIR Job No. EOCWD1.001.001

Description

DETAIL 2



NOTES:

1. ALL BOLTS SHALL BE 7/8" CONFORMING TO ASTM A325
2. ALL STEEL ELEMENTS SHALL BE GALVANIZED IN CONFORMANCE WITH ASTM A123 AND A153.

By RLB Date 07/12 Checked Date

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APPENDIX E

COST ESTIMATE FOR IMPROVEMENTS

3710 Ruffin Road
San Diego CA, 92123
Tel: 858.496.0500
Fax: 858.496.0505
www.richardbrady.com

NAVFAC 11013/7 (1-78)
Supersedes NAVDOCKS 2417 and 2417A

COST ESTIMATE

DATE PREPARED: 23 JUL 12

SHEET E1 OF 1

ACTIVITY AND LOCATION: EOCWD 6 MG RESERVOIR

CONSTRUCTION CONTRACT NO.:

IDENTIFICATION NUMBER:

ESTIMATED BY: RLB

CATEGORY CODE NUMBER:

PROJECT TITLE: ENGINEERING EVALUATION

STATUS OF DESIGN: ☐ PED ☐ 30% ☐ 100% ☐ FINAL ☒ Other (Specify) CONCEPT

JOB ORDER NUMBER: EOCWD.1.001.001

ITEM DESCRIPTION	QUANTITY NUMBER	QUANTITY UNIT	MATERIAL COST UNIT COST	MATERIAL COST TOTAL	LABOR COST UNIT COST	LABOR COST TOTAL	ENGINEERING ESTIMATE UNIT COST	ENGINEERING ESTIMATE TOTAL
DIVISION 01 - GENERAL REQUIREMENTS								30,000
DIVISION 02 - EXISTING CONDITIONS								
METAL ROOF DEMOLITION	476	SQ.					95.00	45,220
METAL ROOF REMOVAL	350	CY					10.00	3,500
DIVISION 04 - MASONRY								
ADHESIVE ANCHORS	225	EA					30.00	6,750
DIVISION 05 - METALS								
LIGHT STRUCTURAL STEEL	9	TN					4500.00	40,500
DIVISION 06 - WOOD AND COMPOSITES								
23/32" ROOF SHEATHING	47600	SF					1.50	71,400
DIVISION 07 - THERMAL AND MOISTURE PROTECTION								
SPF ROOFING	476	SQ					180.00	85,680
						SUB-TOTAL		283,050
OVERHEAD, PROFIT, BONDS (25%)								70,760
DESIGN CONTINGENCY (20%)								56,610
ESCALATION								39,580
						TOTAL		450,000

S/N 0105-LF-010-1335

EAST ORANGE COUNTY WATER DISTRICT

PLC Control Panel Upgrades
RTU 4 – 6 MG Reservoir Station

PLC Control Description

T21062xE

January 11, 2023

Tesco®
CONTROLS, INC.

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TABLE OF CONTENTS

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SYSTEM INFORMATION

Project Information

TESCO T#	21062xE
End User	East Orange Water District
Project/Site Name	PLC Control Panel Upgrades – RTU 4 – 6 MG Reservoir Station

PLC Information

Hardware	Manufacturer M340	P/N BMXP34020H	FW v. 3.2
Software	Schneider Control Expert		SW v. 15.0

HMI Information

Hardware	Advantech	P/N TPC-B200-J12AE	FW v. XX.XX
Software	Ignition		SW v. XX.XX

END SECTION

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PROCESS OVERVIEW

This document describes the control strategy governed by the programmable logic controller (PLC) provided by Tesco Controls, Inc., (TESCO) to East Orange County to control the equipment associated with 6 MG Reservoir.

The PLC controls and monitors the following process areas, which are described in more detail in the sections below.

- Reservoir Station (RSV)
- Control Panel (CP)

NOTE

For more detailed information on the equipment, instruments, and other devices not provided by TESCO, please refer to the appropriate manufacturers' user documentation. This document focuses on the features and functions provided by the TESCO-supplied PLC only.

Reservoir Station (RSV)

The Reservoir Station process area is used to monitor Tank level/flow, Chlorine Residual, and Discharge Pressure.

Equipment Tag	Equipment Description
NA	NA

Instrument Tag	Instrument Description
LIT 40 FIT QR PT 34	6 MG Tank Level Station Flow Chlorine Residual Discharge Pressure

Control Panel (CP)

The control Panel process area consists of instrumentation to monitor panel intrusion, power conditions (ATS, UPS, phase monitor and power fail) and their respective alarms.

Instrument Tag	Instrument Description
-	RTU Panel Intrusion
-	Reservoir Intrusion
-	UPS in Bypass

END SECTION

DEFINITIONS

The definitions for common terms, abbreviations, and acronyms used throughout this document are listed below.

HMI	Human Machine Interface
HOA	Hand-Off-Auto
I/O	Input/Output
LOR	Local-Off-Remote
MCC	Motor Control Center
PLC	Programmable Logic Controller
RTC	Real Time Clock
RVSS	Reduced Voltage Soft Starter
VFD	Variable Frequency Drive

END SECTION

GENERAL CONTROLS AND PROGRAM FUNCTIONS

Operating Modes

Each piece of equipment can be placed in one of the following operating modes.

LOCAL	The mode in which the equipment is controlled according to the system's hardwired control logic (no PLC control). Process and equipment conditions are still monitored, and alarms will be generated, but the PLC has no control over the equipment. A piece of equipment is in its LOCAL mode when an operator places its LOR switch in the LOCAL position.
AUTO	The mode in which the equipment is controlled according to the PLC's automatic control logic. A piece of equipment is in its AUTO mode when an operator places its LOR switch in the REMOTE position and selects AUTO at the HMI/SCADA.
MANUAL	The mode in which the equipment is controlled via the HMI's software selector switches (e.g., START/STOP, OPEN/CLOSE) and numerical inputs (e.g., speed setpoints, position setpoints). A piece of equipment is in its MANUAL mode when an operator places its LOR switch in the REMOTE position and selects MANUAL at the HMI/SCADA.

NOTE

If the PLC loses communication to either the HMI or SCADA, all equipment will default their respective AUTO modes.

Any transition from AUTO mode to MANUAL mode will be bumpless, meaning that any equipment that is actively running will remain in operation once the mode transition is complete.

Equipment Constraints

Each piece of equipment has a set of constraints that will interrupt its normal operation if they occur. These constraints are divided into three types.

Fault	An equipment-related condition that will prevent equipment from operating OR will shut down equipment if currently operating.
Interlock	A process-related condition that will prevent equipment from operating OR will shut down equipment if currently operating.
Inhibit	An equipment- or process-related condition that will prevent equipment from starting to operating BUT will not shut down equipment if currently operating.

Equipment Status

A piece of equipment is considered **available** if either of the following conditions is true.

- The equipment is in REMOTE, is not operating, and does not have any active faults, interlocks, or inhibits.
- The equipment is in REMOTE, is operating, and does not have any active faults or interlocks.

> NOTE
>
> Any active faults or interlocks must be reset and the alarms must be reset by an operator for the equipment to become available again.

A piece of equipment is considered **auto-available** if it is available and it is in AUTO mode.

A piece of equipment is considered **ready** to operate if either of following conditions is true.

- The equipment is available, its restart delay timer has expired (if applicable), and its system's sequential start delay timer has expired (if applicable).
- The equipment is available and is already operating.

Equipment Sequencing, Alternation, and Backup

For groups of equipment (e.g., pumps) that operate using a LEAD/LAG1/LAG2/etc. control scheme under AUTO control, each member of the group receives an assignment depending on the active equipment sequence mode.

Fixed Sequence

If Fixed Sequence mode is active, the equipment assignments are manually selected by an operator. These assignments will not change unless they are manually adjusted or Auto-Alternation Sequence mode is active.

An operator can place the equipment sequence in Fixed by setting the Alternation Sequence setpoint to a permutation of 1 through the total number of equipment. For example, for a four-pump system, a setpoint value of 3241 is valid, where each digit from left to right represents the LEAD, LAG1, LAG2, and STANDBY assignments, respectively.

Auto-Alternation

If Auto-Alternation mode is active, the equipment assignments will alternate each time all pumps stop running. This allows the equipment runtimes to be distributed evenly.

An operator can place the equipment sequence in Auto-Alternation mode by setting the Alternation Sequence setpoint to a value of 0.

Backup

If a piece of equipment is not auto-available when it receives a start command, or becomes not auto-available while running, the next available equipment will be called to run in its place as a backup.

A piece of equipment running as backup will not alternate the equipment assignments.

Real Time Clock

The PLC has an internal real time clock (RTC). The RTC can be adjusted by an operator through the HMI/SCADA and is used for PLC functions such as recording equipment starts and runtimes.

Watchdog Timer

The PLC has an internal watchdog timer. This timer is available to be monitored by external control systems (e.g., SCADA, vendor packages) to verify the health of the network communications between these systems. Each second, the watchdog timer increments a dedicated data register by 1. This register will continuously increment from 0 to 32000, then resets itself and repeats the cycle again. If a remote device that polls the PLC sees the watchdog timer stop this incrementation cycle (i.e., the register value does not change), a communication or plant failure alarm should be annunciated by the remote device.

Sequential Start Delay Timers

Each process area has a sequential start delay timer. This timer prevents multiple pieces of equipment from starting simultaneously to avoid large, instantaneous spikes in the demand for utility or generator power.

Restart Delay Timers

For each piece of motorized equipment (e.g., VFD pump), there is a separate restart delay timer. These timers prevent an individual piece of equipment from immediately restarting after shutting down to allow the equipment to cool down after operation, to prevent excessive wear on the equipment, and to allow the process to settle before starting again (e.g., pump backspin).

Control Delay Timers

Each control condition and setpoint (e.g., pump start level) has an associated control delay timer. These timers require the control condition to be true for an operator-adjustable amount of time before enacting the control action (e.g., level must be above setpoint for control delay before starting pump).

Setpoint Minima and Maxima

All operator-adjustable setpoints (e.g., control setpoints, alarm setpoints) are limited by minimum and maximum values that are hardcoded into the PLC programming. If an operator enters a value that is outside the range defined by the minima and maxima, the PLC will default the entry to the minimum (if value is below range) or the maximum (if value is above range).

VFD Speed Parameters

For each motor that is controlled by a VFD, an operator can adjust the parameters for minimum speed and maximum speed at the VFD controller itself. The VFD controller will restrict any motor speed commands from the PLC to be within this range.

END SECTION

ALARMS

The PLC provides alarms for conditions that should be made visible to operators and/or that provide protective action.

Alarm Types

Each alarm is classified as one of two types.

Non-latching	Alarms that, after they are annunciated, will self-reset once their alarm conditions are no longer true.
Latching	Alarms that, after they are annunciated, will reset only if their alarm conditions are no longer true AND they are reset by an operator.

NOTE

When a non-latching alarm is annunciated, its status will be retained in the PLC for at least a minimum amount of time to ensure that the alarm can be polled at SCADA.

Alarm Features

Alarms have the following features.

Alarm condition	A condition (or set of conditions) based on the PLC's physical/network I/O or internal system logic that is considered "off-normal" or that operations should be aware of if it is active.
Alarm delay	An operator-adjustable time setpoint (in seconds) for which an alarm condition must be true before the alarm itself is active and annunciated.
Alarm annunciation	The status indication that an alarm condition is true, displayed so that it is visible to operators.
Alarm acknowledge	An operator selection that allows an operator to inform the PLC that an active alarm has been witnessed. An acknowledged alarm will remain active and annunciated.
Alarm reset	An operator selection that allows an operator to unlatch an active, latching alarm IF its alarm condition is no longer true. An alarm reset will have no effect if the alarm condition is still true.
Alarm disable	An operator-initiated toggle that allows an operator to disable an alarm so that it will not become active, even if its alarm condition becomes true for the duration of the alarm delay.
Alarm inhibit	A logically defined toggle that will prevent an alarm from becoming active, even if its alarm condition becomes true, but does not reset an alarm if it is already active.
Alarm setpoint	An operator-adjustable value that the scaled engineering value must exceed (increasing for high alarms, decreasing for low alarms) for the alarm to be triggered.
Alarm setpoint clear	An operator-adjustable value that the scaled engineering value must exceed (decreasing for high alarms, increasing for low alarms) after an alarm becomes active for the alarm to be automatically reset. Alarm setpoint clears apply to non-latching alarms only.

END SECTION

RESERVOIR STATION (RSV)

The Reservoir Station process area is used to monitor Tank level/flow, Chlorine Residual, and Discharge Pressure.

Equipment Constraints

The following tables list the constraints associated with the equipment. All constraints are applicable in both MANUAL and AUTO control. Any constraints shown in **bold** are hardwired to their respective equipment and will affect the equipment in LOCAL control (constraints not shown in bold do not apply to LOCAL control).

Reservoir Station Constraints	
Faults	NA
Interlocks	NA

LOCAL Control

There are no Local Controls to control equipment.

MANUAL Control

There are no Manual Controls for equipment.

AUTO Control

There are no Auto Controls for equipment.

END SECTION

CONTROL PANEL (CP)

The Control Panel process area consists of instrumentation to monitor power conditions (UPS in Bypass), panel intrusion and their respective alarms.

RTU Panel Intrusion

The RTU panel intrusion alarm will be generated based on the panel limit switch position.

Reservoir Panel Intrusion

The Reservoir panel intrusion alarm will be generated based on the hatch limit switch position.

UPS in Bypass

The UPS in bypass alarm will generated based on the UPS input generation.

END SECTION

TOTALS AND STATISTICS

The PLC provides the following totals for the plant equipment and process flows.

Equipment starts	The number of times a piece of equipment has been started and stopped.
Equipment runtime	The amount of elapsed time that a piece of equipment has been running.
Flow total	The total volume of a fluid that has passed through a process line.

For each type of total, the PLC records four different statistics.

Statistic	Description
Nonresettable	Cannot be reset.
Resettable	Can be manually reset to zero (0) by an operator tor at the HMI/SCADA.
Current day	Automatically reset to zero (0) at 11:59PM each day.
Previous day	Automatically overwritten with current day value at 11:59PM each day.

Note *If a statistic reaches a maximum value (as limited by the size of its PLC data register), the totalization of that statistic will restart from zero (0).*

Flow Totals

Flow totals are recorded for the following process flows.

Process Flow	Engineering Units
Station Flow Pulse *	hundred cubic feet

Note * *These process flow totals are calculated based on each digital input pulse triggered within the sample time.*

END SECTION

NETWORK COMMUNICATIONS

The PLC interfaces with the following devices and systems over the system's network communication.

- Human Machine Interface (HMI)
- Supervisory Control and Data Acquisition (SCADA)
- Peer-to-Peer Device

Human Machine Interface (HMI)

The human machine interface (HMI) provides operators with local monitoring and control of the system.

Communication Protocol	Modbus TCP/IP
Communication Medium	Copper

Supervisory Control and Data Acquisition (SCADA)

The supervisory control and data acquisition (SCADA) system provides operators and other site personnel with remote monitoring and control of the system.

Communication Protocol	Modbus TCP/IP
Communication Medium	Copper

Peer-to-Peer Device

The peer-to-peer device provides process the PLC with process data from the vendor package.

Communication Protocol	Modbus TCP/IP
Communication Medium	Copper

END SECTION

DOCUMENT REVISION HISTORY

Rev. #	Date	By	Description
1	2023.01.11	Garcia, Alejandro	Initial submittal.

END SECTION

APPENDIX A – ALARMS

The following tables list all alarms annunciated by the PLC to the HMI/SCADA.

Reservoir Station (RSV)

Alarm	Type	Notes	Action
6.0 MG Tank Level Xducer Fail	Non-latching		Alarm
6.0 MG Tank Level High	Non-latching		Alarm
6.0 MG Tank Level Low	Non-latching		Alarm
Station Flow Xducer Fail	Non-latching		Alarm
Station Flow High	Non-latching		Alarm
Station Flow Low	Non-latching		Alarm
Chlorine Residual Xducer Fail	Non-latching		Alarm
Chlorine Residual High	Non-latching		Alarm
Chlorine Residual Low	Non-latching		Alarm
Discharge Pressure Xducer Fail	Non-latching		Alarm
Discharge Pressure High	Non-latching		Alarm
Discharge Pressure Low	Non-latching		Alarm

Control Panel (CP)

Alarm	Type	Notes	Action
UPS in Bypass	Non-latching		Alarm
RTU panel intrusion	Non-latching		Alarm
Reservoir panel intrusion	Non-latching		Alarm

END SECTION