OAKRIDGE GEOSCIENCE, INC.

GEOTECHNICAL REPORT

UNDERGROUND PUMP STATION
LAS VIRGENES MUNICIPAL WATER DISTRICT / CALLEGUAS MUNICIPAL WATER DISTRICT INTERCONNECTION PROJECT (PROJECT 450), THOUSAND OAKS, CALIFORNIA

Prepared for:
Phoenix Civil Engineering, Inc.

January 14, 2019
Job No. 003.001

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January 14, 2019
Project No. 003.001

Phoenix Civil Engineering, Inc.
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Attention: Mr. Jon Turner, PE

Subject: Geotechnical Report, Underground Pump Station, Las Virgenes Municipal Water District/Calleguas Municipal Water District Interconnection Project (Project 450), Thousand Oaks, California

Dear Mr. Turner:

Oakridge Geoscience, Inc. (OGI) is pleased to present this geotechnical report for the underground pump station site for the Las Virgenes Municipal Water District/Calleguas Municipal Water District Interconnection Project (Project 450) in Thousand Oaks, California. The geotechnical design study for the pipeline alignment portion of the project was performed separately and provided under separate cover.

The purpose of this report is to summarize the anticipated geotechnical conditions at the underground pump station site and provide geotechnical recommendations in support of the project design by Phoenix Civil Engineering, Inc. Our understanding of the project was based on discussions with you, review of the Pump Station Undergrounding Memorandum dated August 30 2018, and our experience in the project area.

Closure

We appreciate the opportunity to provide geotechnical services for this project and to continue our relationship with Phoenix Civil Engineering, Inc. Please contact us if you have any questions regarding information presented herein.

SINCERELY,
OAKRIDGE GEOSCIENCE, INC.

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1.0 INTRODUCTION

1.1 GENERAL STATEMENT

This geotechnical report summarizes the findings and recommendations of the geotechnical study performed by Oakridge Geoscience, Inc. (OGI) for the underground pump station site for the Las Virgenes Municipal Water District (LVMWD)/Calleguas Municipal Water District (CMWD) Interconnection Project (Project 450) in Thousand Oaks, California. The geotechnical report for the Lindero Canyon Pipeline Alignment portion of the project (interconnection pipeline geotechnical study) was performed separately and provided under separate cover (OGI, 2018).

CMWD and LVMWD are considering design and construction of a pipeline interconnection that would allow transfer of water between the Districts. The CMWD portion of the project involves the design of about 7,500 feet of 30-inch inside-diameter welded steel pipe, a pump station to convey the water into the CMWD system, and a pressure regulating station to flow water to the LVMWD system.

The proposed underground pump station site is located within the Oak Park area of Ventura County as shown on Plate 1.

1.2 PURPOSE

The purpose of this report is to summarize the anticipated geotechnical conditions at the underground pump station site and provide geotechnical recommendations in support of the project design by Phoenix Civil Engineering, Inc. (PCE). Our understanding of the project was based on discussions with you, review of the Pump Station Undergrounding Memorandum dated August 30, 2018, and our experience in the project area.

1.3 PROJECT DESCRIPTION

Project plans by PCE indicate the underground pump station and pressure regulating station will be about 125 feet by 64 feet in plan view, the top of the structure will have a finished grade of elevation (El.) El. +1,059 feet with about a foot of soil cover, the interior finished floor grades will range from about El. +1,048 to +1,042 feet, and the pump columns will extend below the finished floor in the pump room to about El. +1,029.50 feet. The facility will also include an unimproved access road from Lindero Canyon Road. Based on the proposed finished grades, excavation depths are anticipated to range from about 11 to 17 feet or more for the underground pump station structure and to depths of about 30 feet for the pump cans.

1.4 WORK PERFORMED

The scope of services for this study consists of project coordination, project-specific field exploration, laboratory testing, geotechnical engineering evaluation, and preparation of this report. Our proposed scope of services was presented in our proposals dated November 7, 2017 and February 9, 2018 and was authorized by the PCE Agreement Between Consultant and Subconsultant, dated February 5, 2018.

1.4.1 Project Coordination

Prior to field exploration, OGI performed a site reconnaissance to locate and mark the exploration locations for coordination with Underground Service Alert. Additionally, CMWD arranged for clearance of vegetation within the work area for safe access and fire prevention.

1.4.2 Field Exploration

The project-specific subsurface exploration program consisted of advancing two drill holes within the proposed pump station footprint to depths of about 30 to 50 feet below the ground surface (bgs). The approximate locations of the drill holes are shown on Plate 2 and the drill hole logs are provided in Appendix A.

The drill holes were advanced on October 2, 2018 by S/G Drilling, Inc. of Lompoc using a CME 75 truck-mounted drill rig equipped with eight-inch-diameter hollow-stem augers. The drill holes were sampled using a three-inch-outside-diameter modified California split spoon sampler fitted with one-inch-high brass liners and a two-inch-outside-diameter standard penetration test (SPT) split spoon sampler without liners. The split spoon samplers were driven into the materials at the bottom of the drill hole using a 140-pound CME automatic trip hammer with a 30-inch drop. The blowcount is the number of blows from the hammer that were needed to drive the sampler one foot (unless otherwise noted) after the sampler had been seated at least six inches into the material at the bottom of the hole. The sample intervals, blowcounts, and a description of the subsurface conditions encountered are presented on the logs of the drill holes in Appendix A. Following logging and sampling at each location, each drill hole was backfilled to the ground surface with the drill cuttings.

1.4.3 Laboratory Testing

Geotechnical laboratory testing was performed on selected earth materials sampled in the drill holes to characterize the materials and estimate relevant engineering design parameters. The testing program consisted of moisture/density relationships, grainsize, plasticity, consolidation, expansion index, maximum density, and corrosion testing. The laboratory test results are presented on the drill hole logs in Appendix A and in Appendix B.

1.4.4 Geologic/Geotechnical Evaluation and Reporting

We evaluated the field and laboratory geotechnical data, developed geotechnical engineering recommendations for design and construction of the project, and prepared this report to summarize our findings, opinions, and recommendations. Our report includes the following:

- Summary of soil and groundwater conditions encountered;
- CBC seismic design parameters;
- Anticipated excavation conditions;
- Foundation recommendations for the pump station and estimated settlement;
- Lateral earth pressures for the buried pump station;
- Grading recommendations, consisting of clearing and grubbing, stockpiling topsoil (if applicable), preparation of areas to receive fill, thickness of lifts;
- Excavation conditions of earth materials and considerations;

- Suitability of onsite soil for use as fill and select fill material; and
- Preliminary geotechnical input for temporary shoring design.

2.0 FINDINGS

2.1 SITE DESCRIPTION

The pump station site is located in an open field area north of the Ventura/Los Angeles County line between Lindero Canyon Road and the Lindero Creek drainage (Plate 1). Lindero Creek is a blue line stream on the USGS topographic quadrangle and water was observed flowing in the creek at the time of our site reconnaissance and field exploration. Project plans by PCE indicate the existing ground surface elevations at the site slope southeasterly toward the creek.

Elevations within the project footprint range from about El. +1,060 feet in the northwestern corner of the site to about El. +1,051 feet in the southeastern corner of the site. A descending, creek bank borders the eastern portion of the property about 150 feet east of the pump station structure (Plate 2). The creek bank has been eroded to form a stepped profile. The upper slope is about 15-feet high and is oriented at about 1 horizontal to 1 vertical (1h:1v). The toe of the upper slope flattens to form a mid-slope bench before descending at about 1h:1v to near vertical to the flowing creek below. Bedrock materials were observed in the lower portion of the eastern creek bank and within the creek bottom near the pump station site. Also, some signs of bank instability were observed during our site reconnaissance.

2.2 GEOLOGIC SETTING

2.2.1 Regional Geology

The project site is located within the Transverse Ranges geologic/geomorphic province of California. The province is characterized by generally east-west trending mountain ranges composed of sedimentary and volcanic bedrock units ranging in age from Cretaceous to Recent. Major east-west trending folds, reverse faults, and left-lateral strike-slip faults reflect regional north-south compression and are characteristic of the province.

2.2.2 Local Geology

The geology of the project area has been mapped by several authors including Dibblee (1993), Weber (1973), and the California Geological Society (CGS, formerly California Division of Mines and Geology; 2000). Regional mapping by Dibblee (1993) suggests the project area is predominantly underlain by unconsolidated alluvial sediments (Qa) consisting of gravel, sand, and clay underlain by bedrock of the Monterey Formation (Tm) as indicated on Plate 4. Monterey Formation bedrock materials are exposed in the slopes west of Lindero Canyon Road, and on the slopes east of Lindero Creek in the project area. Dibblee indicates the Monterey Formation consists of white weathering, thinly bedded, locally brittle siliceous to punky siltstone materials that have been folded into a series of northwest-trending synclines and anticlines that result in varying dip magnitudes and directions in the project vicinity. The Monterey Formation can contain well-indurated siliceous and dolomitic beds that can range from several inches to several feet in thickness and can be difficult to excavate. The bedrock materials mapped by Dibblee and observed in the eastern creek bank of Lindero Creek near the project site, dip to the northeast about 45 to 60 degrees, however, bedding within the Monterey Formation can vary significantly locally and/or over short distances.

Weber (1973) maps potential “conjectured” faults within the alluvial sediments east of the Lindero Creek drainage, about 500 to 1,000 feet east of Lindero Canyon Road. The mapped conjectured faults are not considered active or potentially active, are northerly-trending (not consistent with the structural grain of the project area), and do not cross or project toward the project site.

2.3 SUBSURFACE CONDITIONS AND ENGINEERING PROPERTIES

Subsurface materials encountered by our explorations are interpreted to consist of artificial fill related to previous disturbance (i.e., discing), alluvium, and bedrock of the Monterey Formation. Earth material descriptions are presented in the following sections.

2.3.1 Artificial Fill (af)

The artificial fill materials encountered in our explorations consist soft sandy clay and clay with gravel to depths of about two- to two-and-a-half feet. The fill materials appear to have been disturbed by prior site activities such as discing, grading, etc.

2.3.2 Alluvium (Qal)

Alluvial soils sampled in the drill holes consist primarily of medium stiff to stiff sandy clay, clay, and clay with gravel/bedrock fragments. Soft sandy elastic silt and clayey silt were encountered at a depth of about five to seven feet in drill holes DH-101 and DH-102, respectively (Appendix A). Also, voids were observed in the sample of clay sediments at a depth of about 25 feet in DH-101. Medium dense to dense clayey sand and silty sand sediments were encountered in samples recovered below about 34 feet in DH-101; granular sediments were not encountered in the sampled materials in DH-102. The results of the field and laboratory tests on samples of alluvial materials suggest:

- Field standard penetration test (SPT) blowcounts within the clayey alluvium generally ranged from about four to 21 blows per foot (bpf) and the blowcounts within the granular materials ranged from about 29 to 42 bpf.
- In-place dry densities ranged from about 70 to 96 pounds per cubic foot (pcf) and the moisture contents ranged from about 17 to 33 percent for the tested cohesive (clayey) samples; similar to the findings from the interconnection pipeline geotechnical study (OGI, 2018).
- The in-place dry density of a sample of granular material was 116 pcf and moisture contents of granular material ranged from about 11 to 29 percent.
- The results of grainsize evaluations indicate fines contents of about 56 to 72 percent for tested cohesive materials and 17 to 39 percent for granular materials.
- The results of plasticity tests indicate liquid limits of 52 to 53 and plasticity indices of 17 and 24, suggesting the tested materials are silt (ML), elastic silt (MH), fat clay (CH), and lean clay (CL).

- The results of an expansion index (EI) test indicate the near surface sandy clay soil has an EI of 54 (moderately expansive).
- The results of a consolidation test on a sample of sandy clay alluvium from a depth of 25 feet in DH-101 indicates the sample has a preconsolidation pressure of about four kips/square foot and a total shear strain of eight percent at a load of 20 kips (Appendix B).
- The maximum density of the fine-grained clay soil is 124 pcf at an optimum moisture content of 12 percent. Comparison of the in-place densities in the upper 10 feet of the site indicate the existing soils have a relative compaction of less than 75 percent as compared to the laboratory maximum density and the existing moisture contents are eight to 15 percent above the optimum moisture content.

2.3.3 Monterey Formation (Tm)

Bedrock of the Monterey Formation was encountered at a depth of about 24 feet (El. +1,035 feet) in DH-102 and 42 feet (El. +1,013 feet) in DH-101 (Plate 3), indicating the bedrock contact slopes toward the creek below the pump station footprint. The bedrock materials were interpreted as interbedded siltstone, claystone, and sandstone with dolomitic lenses. Based on the blowcount data, the weathered siltstone, claystone, and sandstone bedrock materials have the consistency of hard silt and clay and very dense sandy soil materials. Dolomitic beds/lenses consist of cemented, very hard, well indurated bedrock material that can be difficult to excavate. Monterey Formation bedrock is also exposed in the Lindero Creek banks in the project vicinity and on the slopes west of Lindero Canyon Road and east of the creek. We note the bedrock exposed in the cutslopes west of Lindero Canyon Road northwest of the site appear to contain zones of well indurated, hard bedrock materials versus the weathered, less indurated bedrock materials encountered by our explorations.

The results of the laboratory tests on samples of bedrock materials suggest an in-place dry density of 87 pcf and moisture contents of 22 to 28 percent, similar to values reported in the interconnection pipeline geotechnical study (OGI, 2018). The low dry densities and high moisture contents suggest the tested bedrock materials are diatomaceous.

2.4 GROUNDWATER

Groundwater was encountered at a depth of about 33 feet (about El. +1,022 feet) in DH-101; groundwater was not encountered in DH-102. As indicated above, water was observed flowing in the creek at the time of our site reconnaissance and field exploration. Review of CGS (2000) indicates the historic high groundwater may be within 10 feet or less of the ground surface within the vicinity of the pump station site. We note groundwater may be encountered at shallower depths at other times.

2.5 POTENTIAL VARIATION OF SUBSURFACE MATERIALS

There is a potential for variation in the consistency, density, and strength/hardness of the materials. There is also potential for oversized materials (greater than eight inches in diameter); perched water; zones of poorly consolidated soils; well indurated, very hard bedrock materials; or other conditions not indicated on the drill hole logs. If significant variation in the geologic conditions is observed during construction, we recommend that the geotechnical engineer, in conjunction with the project designer, evaluate the impact of those variations on the project design.

2.6 SEISMIC CONSIDERATIONS AND GEOHAZARDS

2.6.1 Faults

No known active or potentially active faults traverse or trend toward the pump station site. However, there are numerous faults considered active and potentially active by the USGS within an about 20-mile radius of the site as indicated in the following table.

Table 1. Nearby Faults

Fault	Approximate Distance (miles)	Maximum Moment Magnitude (Mmax) USGS
Simi-Santa Rosa	7.5	6.8
Anacapa-Dume	10.3	7.1
Santa Susana	12.7	6.8
Northridge	14.0	6.8
Oak Ridge (onshore)	14.1	7.1

[1] Earthquake distances and magnitudes obtained from the USGS website (2017)

2.6.2 Ground Rupture Potential

The pump station site is not located within a State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zone) and no known active or potentially active faults traverse or trend toward the site. Weber (1973) maps a potential fault trace about 500 feet east of the site subparallel to Lindero Canyon Road, however, the fault is not considered active or potentially active and it does not trend toward or cross the site. Thus, the potential for fault rupture is considered low.

2.6.3 Seismic Considerations for 2016 CBC

We estimated the probabilistic seismic ground acceleration at the pump station site using the USGS web application (USGS; 2018). On the basis of the web-based analyses, the peak horizontal ground acceleration (pga) at the proposed site is estimated to be 0.50g for an earthquake with a 2,475-year return period (2 percent probability of exceedance in 50 years) assuming Site Class D soil conditions. Table 2 summarizes the probabilistically estimated strong ground motion parameters for the project site.

Table 2. Summary of USGS Probabilistic Seismic Hazard Deaggregation Results

Return Period (years)	Mean Magnitude (Mw)	Mean Source Distance (miles)	Peak Horizontal Ground Acceleration
2,475	6.8	11.3	0.50g

2.6.4 2016 CBC Seismic Design Parameters

In accordance with Chapter 16, Section 1613 of the 2016 CBC, the following parameters have been obtained from the USGS Seismic Design Maps web application (USGS, 2018) and shall be incorporated into the seismic design at the project site. The subsurface conditions at the site are considered to satisfy the parameters for Site Class D. The associated seismic design parameters for Site Class D for use in generating the risk-targeted maximum considered earthquake and design level spectra are summarized in the following table.

Table 3. 2016 CBC Seismic Design Parameters

2013 California Building Code Section 1613	Seismic Parameter	Site Class D Values
---	Latitude	34.1684
---	Longitude	-118.7882
Figure 1613.3.1(1)	Mapped Acceleration Response Parameter (S_s)	1.50
Figure 1613.3.1(2)	Mapped Acceleration Response Parameter (S_1)	0.60g
Section 1613.3.2	Site Class	D
Section 1613.3.3 and Table 1613.3.3(1)	Site Coefficient (F_a)	1.0
Section 1613.3.3 and Table 1613.3.3(2)	Site Coefficient (F_v)	1.5
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{MS})	1.50g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{M1})	0.90g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{DS})	1.0g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{D1})	0.6g
Section 1613.3.3	Site Period, Figure 22-12, T_L	T_L = 8 sec
Section 11.8.1	Peak Ground Acceleration $PGA_{M;}$ Equation 11.8-1	0.502g
Section 21.2.1	Adjusted Acceleration Response Parameter (Crs), Figure 22-17	1.032 g
Section 21.2.1.1	Adjusted Acceleration Response Parameter (C_{R1}), Figure 22-18	1.044g

2.6.5 Liquefaction and Dry Seismic Settlement Potential

Soil liquefaction occurs as a result of a loss of shear strength or shearing resistance in loose, saturated soils subjected to earthquake-induced ground shaking. Soil liquefaction occurs in the underlying soils and can be manifested at the ground surface by the formation of sand boils, ground surface settlement and lateral spreading. Dry seismic settlement occurs in weakly- to non-cemented, very loose to medium dense granular soils above the groundwater in response to strong earthquake ground shaking.

The pump station site is underlain by alluvial sediments that range from about 25 to 42 feet thick that are underlain by bedrock materials of the Monterey Formation at the exploration locations. The alluvial sediments consist primarily of medium stiff to stiff sandy clay, clay, and clay with gravel/bedrock fragments with a soft, sandy to clayey silt zone at a depth of about five to seven feet. Additionally, medium dense to dense clayey sand and silty sand sediments were encountered in the eastern-most drill hole (DH-101; Plate 3) from a depth of about 34 to 42 feet. Groundwater was encountered at a depth of about 33 feet in DH-101; however, we utilized a design historic high groundwater level of 10 feet for the site per the CGS (2000).

Based on our evaluation, there is a low potential for liquefaction and dry seismic settlement at the site. The medium stiff to stiff fine-grained (clay and silt) soils have plasticity indexes of 17 to greater than 20. Research by Bray and Sancio (2006) indicates that clay soil with a plasticity index above 12 exhibit “clay like behavior” and are not susceptible to liquefaction. The granular soil layer in DH-101 from 34 to 42 feet has energy and depth corrected blowcounts above 40 bpf indicating that the granular soils are unlikely to liquefy during the design seismic event (Appendix C). The Monterey Formation siltstone bedrock encountered below a depth of 24 to 42 feet in the drill holes also is not susceptible to liquefaction.

The soft, sandy to clayey silt encountered from five to seven feet in drill holes DH-101 and DH-102 potentially could be susceptible to dry seismic settlement during the design seismic event; however, given the high fines content (approximately 60+ percent), in our opinion, settlement is unlikely. In addition, the soft soil layer from five to seven feet will be removed in the footprint area as part of the excavation to construct the pump station facility to a depth of 11 to 17 feet below existing grade. Based on our evaluation, in our opinion, the potential for seismic related settlement at the site is low.

2.6.6 Landsliding and Slope Instability

The underground pump station is located on a gently eastward sloping undeveloped site bound to the east by descending banks of the unlined Lindero Creek. The top of the uppermost bank of the unlined creek is located about 150 feet east of the pump station structure and the active, flowing channel is estimated to be located about 250 feet east of the pump station structure based on Google Earth imagery. Thus, the potential for landsliding and/or slope instability to affect the underground pump station is considered low.

2.6.7 Lateral Movement

The occurrence of lateral spreading is generally associated with sites where liquefaction is possible and: 1) the ground surface is sloping, or 2) there is a free-face condition such as a road cut or riverbank. Existing analytical methods of assessing potential deformations caused by lateral spreading are based on a small number of case histories and generally involve layers of liquefiable soils of greater than about three feet. The procedures are generally considered reasonable in assessing risks where significant lateral deformations are possible (deformations of three feet or greater). The ability to reasonably predict small lateral spreading deformations is, however, considered significantly limited.

As described in Section 2.1, there is a free-face approximately 15-feet high, in the Lindero Canyon drainage located about 150 feet east of the pump station site. However, the analyses for this study indicates there is a low potential for liquefaction of the primarily fine-grained clay soils at the site. Based on our evaluation there is low potential for lateral movement at the site.

2.7 HYDROCONSOLIDATION (COLLAPSE) POTENTIAL

Research by several authors, including Houston et al. (1997; 2001) and Purdue University (Howayek, 2012), indicates hydroconsolidation (collapse) typically occurs in silty and granular soil materials with densities below 105 pcf, degrees of saturation of less than 25 percent, and with high void ratios. In the Ventura County area, our experience indicates hydroconsolidation is commonly associated with silty soils deposited in debris-flow type environments. The depositional environment with high collapse potential previously observed in Ventura, Camarillo, and Simi Valley consists of Holocene- to Late Pleistocene-age alluvial fan deposits above the groundwater. As noted above in the Geologic Setting section of this report, the proposed site is located on Holocene-age alluvial soils.

The sample from a depth of 25 feet in DH-101 had visible pores to about one-millimeter (mm) in diameter. The 25-foot sample was tested for consolidation in general accordance with ASTM D2435 and the results are presented in Appendix B. The sample swelled slightly, 0.2 percent, upon addition of water.

Evaluation of the laboratory index properties (soil density, moisture content, void ratio, and fines content) on the primarily fine-grained clay soils in drill holes DH-101 and DH-102 indicate soil dry densities in the range of 70 to 116 pcf, saturation of 39 to 100 percent, and void ratios of 0.43 to 1.36. Based on our evaluation, there is a low potential for collapse of the primarily fine-grained clay soils below a depth of 10 feet. The dense granular soils located below the groundwater also are unlikely to collapse.

The typical procedure to mitigate shallow collapse potential is to overexcavate and recompact the soil. As summarized below, excavation for the pump station will remove the upper 11 to 17 feet of near-surface soils. Recommendations for a limited overexcavation and recompaction beneath the structure are presented in Section 3 of this report.

2.8 EXPANSIVE SOIL

The fine-grained soil in the upper 10 feet of the drill holes consists primarily of sandy clay, sandy silt, and clayey silt with about 30 to 40 percent sand (60 to 70 percent silt and clay fines). The fine-grained soil has plasticity indexes in the range of 17 to 24, liquid limits of 42 to 53 and an expansion index (EI) of 54 (Appendix B). An EI of 54 classifies as medium expansion potential (EI of 51 to 90). The moisture contents indicate partial soil saturation (40 to 60 percent) above a depth of 15 feet and 90 percent plus saturation at 15 feet and greater. The data indicate if moisture contents in the upper 15 feet of the site were to increase, there is a potential for the in-place soils to expand.

3.0 RECOMMENDATIONS

3.1 SUMMARY OF SUBSURFACE SITE CONDITIONS

The geotechnical conditions in the project area were evaluated from the drill holes advanced for this study, the LVMWD/CMWD interconnection pipeline geotechnical study (OGI, 2018), and from our review of existing geotechnical data as referenced herein.

- The site is underlain by primarily fine-grained sandy clay to clay soil with variable amounts of gravel-size rock fragments, and sandy to clayey plastic silt (MH) lenses;
- Granular soil was encountered in drill hole DH-101 from 34 to 42 feet and siltstone bedrock of the Monterey Formation was encountered at depths of 24 to 42 feet in the drill holes advanced on-site (Appendix A);
- Groundwater was encountered at a depth of 33 feet in drill hole DH-101;
- The fine-grained soil has an expansion index (EI) of 54 (medium expansion);
- Design earthquake ground motion is 0.50 g; and
- Geotechnical evaluations performed for this study indicate a low potential for liquefaction, dry seismic settlement, lateral spreading and hydroconsolidation (collapse).

3.2 SOIL CHEMISTRY AND CORROSION

3.2.1 Test Results

A soil sample obtained from our explorations was provided to Cooper Testing Laboratories for resistivity, pH, chloride, and sulfate testing. The test results are summarized below and the laboratory test report is included in Appendix B.

Table 4. Summary of Chemical Test Results

Drill Hole	USCS Classification	Depth (feet)	Sulfate (mg/kg)	Sulfate) (%)	Chloride (mg/kg)	Resistivity (ohm-cm)	pH
DH-101	Sandy CLAY (CL)	0 to 5’	18	0.0018	17	1,546	7.6

3.2.2 Corrosion and Cement Considerations

Many factors can affect the corrosion potential of soil, including soil moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soil, is the most influential factor. As a general rule, Caltrans (2018) indicates a resistivity value of 1,000 ohm-cm or lower is an indicator of high soluble salt content and a general indicator of corrosion potential. Caltrans considers soils to be corrosive or to represent a corrosive environment if one of the following criteria is met:

- Resistivity value of less than 1,000 ohm-cm;
- Chloride content of 500 ppm or greater;
- Sulfate concentration of 2,000 ppm or greater; or
- pH is 5.5 or less.

As summarized in the table above, the measured electrical resistivity (ASTM G57) for the sample from DH-101 is 1,546 ohm-cm, the chloride content is 17 mg/kg, the sulfate content is 18 mg/kg, and the pH is 7.6. Based on the laboratory test data, the tested soil from DH-101 is not considered corrosive to concrete or steel based on the test data and Caltrans limits.

The test results should be evaluated by a corrosion specialist to confirm the opinions regarding the potential corrosion impacts from the onsite soils to the construction materials proposed for the project.

3.3 EXCAVATIONS

3.3.1 Excavation Conditions

The earth materials encountered in the drill holes advanced for this study consist primarily of medium stiff to stiff sandy clay, clay, and clay with gravel/bedrock fragments and medium dense to dense clayey sand and silty sand sediments (encountered in DH-101). Also, we note soft cohesive soils were encountered at a depth of about five to seven feet bgs in both explorations advanced for this study. The alluvial sediments range from 25 to 42 feet thick and are underlain by bedrock materials of the Monterey Formation at the exploration locations. Based on our observations during drilling, we anticipate conventional heavy-duty grading equipment in good working order should be capable of excavating the earth materials encountered at the pump station site. However, we note difficulty may be encountered when excavating hard, siliceous and dolomitic beds within the Monterey Formation bedrock materials. There is also a potential for the excavations for the pump cans to encounter groundwater.

Bedrock is exposed in the cutslopes on the western side of Lindero Canyon Road, on the slopes east of Lindero Creek, and within the Lindero Creek channel. The bedrock materials encountered in our explorations and along the pipeline alignment near the pump station site (OGI, 2018) were interpreted as thinly interbedded claystone, siltstone, sandy siltstone, and sandy claystone.

Review of the existing data and our experience suggests the Monterey Formation can contain well-indurated siliceous and dolomitic beds that can range from several inches to several feet in thickness and can be difficult to excavate. Bedrock materials exposed in the cutslopes west of Lindero Canyon Road appear to contain zones of more indurated, harder bedrock materials than encountered by our explorations. The potential exists for difficult excavation conditions to be encountered in bedrock materials.

3.3.2 Overexcavation

Soft cohesive soils were encountered at a depth of about five to seven feet bgs in both explorations advanced for this study as depicted on the interpreted profile on Plate 3. As indicated on Plate 3, the soft zone extends to approximately El. +1,046 feet, which is about three feet below the proposed pump station floor in the eastern portion of the building. The soft soil materials will need to be excavated to expose the underlying medium stiff to stiff alluvial sediments. The excavation for the structure should extend to a depth of at least two feet below the proposed foundation level beneath the entire pump station footprint and extend outward a minimum of three feet beyond the edges of the structure. The exposed alluvial materials should be scarified to a depth of 12-inches, moisture conditioned or dried back as appropriate, and recompacted to 90 percent relative compaction as compared to the latest ASTM 1557 test method. The upper one-foot of soil beneath the foundation should consist of compacted aggregate base (processed miscellaneous base) as outlined in Section 3.7.3.

3.3.3 Dewatering

Groundwater was encountered at a depth of about 33 feet (about E. +1,022 feet) in DH-101; groundwater was not encountered in DH-102. As indicated above, water was observed flowing in the creek at the time of our site reconnaissance and field exploration. Review of CGS (2000) indicates the historic high groundwater may be within 10 feet or less of the ground surface within the vicinity of the pump station site. We note groundwater may be encountered at shallower depths at other times.

We note that groundwater seepage may be encountered at shallower depths, especially in areas adjacent to irrigated or landscaped areas, areas underlain by less permeable alluvial or bedrock materials, or in areas proximal to the Lindero Creek drainage. Therefore, the potential exists to encounter groundwater and that dewatering may be required.

If dewatering is required as part of the project, a dewatering system capable of dewatering project elements should be designed and installed by an experienced company specializing in groundwater dewatering systems. That system should be capable of lowering the groundwater surface to a depth of at least three-feet below the required depth of excavation.

3.4 BELOW-GRADE STRUCTURES

3.4.1 Allowable Bearing Capacity and Static Settlement

The underground pump station structure will consist of concrete walls and concrete foundations. The foundation levels for the structure are anticipated to be about 11 feet to 17 feet below the ground surface based on the project plans. The excavation will result in a net unloading for the soil beneath the structure. The weight of the existing soil in upper 11 to 17 feet is estimated to range from about 1,100 to 1,800 pounds per square foot (psf). The load of new pump station (concrete structure and one-foot of soil cover) likely will be about 500 psf. Based on the anticipated construction method, the proposed structure can be supported on a mat-type foundation constructed on a two-foot thick compacted fill pad placed over the processed and compacted subgrade surface. The two-foot thick compacted fill pad should consist of one-foot of compacted aggregate base or ¾-inch crushed rock over one-foot of compacted fill. The fill should be compacted to 95 percent relative compaction.

If the contractor excavates a slot-cut or large area to construct the pump cans, the excavation should be backfilled with fill compacted to 95 percent relative compaction. In addition, the thickness of the fill beneath the structure should not vary more than 15 percent across the structure in any direction. Therefore, if a slot-cut or large excavation is made, additional excavation will be required outside of the slot-cut to provide a relatively uniform thickness of fill beneath the structure. If the pump can excavations are "drilled in", the backfill around the cans should consist of sand-cement slurry or flowable fill that can fill voids and have a strength of about 100 pounds per square inch (psi) or greater.

We recommend a maximum allowable (net) bearing pressure of 2,000 psf be used for the design of the proposed foundations. The maximum allowable bearing pressure is considered applicable to a mat-type foundation slab and can be increased by one-third when considering short-term or seismic loads.

Considering the embedment depth and size of the foundation system, a higher bearing capacity could be used. However, we recommend the bearing value be limited to 2,000 psf in an effort to keep potential static settlements of the multi-tiered structure to less than one-half inch over a distance of 30 feet.

3.4.2 Uplift Resistance

The underground pump station structure could be subject to uplift forces if groundwater conditions were to change in the future and rise to reported historical levels of about 10 feet bgs (CGS, 2000). The magnitude of the uplift pressure acting on the structures will depend on the groundwater level at the structure location. Groundwater was encountered in DH-101 at a depth of about 33 feet bgs. However, the CGS (2000) indicate historic groundwater levels of 10 feet bgs have been reported in the project area.

In our opinion, pump station and manholes/vaults (if applicable) should be designed for uplift assuming groundwater is present at a depth of about 10 feet bgs. The uplift forces may be resisted by: 1) the gross weight of the structure; 2) friction between soil interfaces for structures with footing extensions; and 3) the weight of soil located above footing extensions beyond the outside walls of the structures.

The frictional resistance can be estimated to increase linearly with depth. We recommend an ultimate frictional resistance of 35 psf per foot of depth be used for soil interfaces above the groundwater level. For soil below the groundwater level, the rate of increase in the frictional resistance should be assumed at 15 psf per foot of depth. If footing extensions are used, the potential wedge of soil providing resistance to uplift should be considered to be bounded by the outside walls of the structure and a vertical plane extending up from the edge of the footing extension. A unit weight of 125 pcf should be used to estimate the weight of unsaturated soil above the footing extensions. For soils below the groundwater level, a unit weight of 60 pcf should be used.

3.4.3 Modulus of Subgrade Reaction

The design of large footings and slabs may be predicated on an analogy with a beam on an elastic half-space. A modulus of subgrade reaction of 200 pounds per cubic inch (pci) may be used for design. The modulus of subgrade reaction is for a one-foot-square plate, and the value should be corrected for beam or mat size and shape, assuming a cohesionless subgrade.

The design engineer should design the reinforcement, and the amount and layout of the reinforcement should conform to pertinent structural code requirements. However, the minimum reinforcement should not be less than that required for shrinkage and temperature control.

3.4.4 Lateral Bearing Pressures

In accordance with 2016 CBC Section 1806.3.1, resistance to lateral loading may be provided by both friction acting at the base of foundations and by lateral bearing pressure. The presumptive values for lateral bearing pressure given within 2016 CBC, Table 1806.2 as well as the allowable increase for depth noted in 2016 CBC Section 1806.3.3 shall be superseded by the site-specific values in the following table. The lateral bearing pressure in the upper one foot of the site should be neglected unless the ground surface is covered with asphalt or concrete, and the lateral bearing pressure may be increased by 300 psf for each additional foot of embedment to a maximum value of 2,500 psf for level ground adjacent to the structure.

Table 5. Summary of Lateral Bearing Pressures

Allowable Bearing Material[1]	Allowable Lateral Bearing Pressure[2]	Maximum Lateral Bearing Pressure[3]	Allowable Coefficient of Friction[4]
Level Pad Surfaces			
Existing Alluvium	300 psf/ft	2,500 psf	0.35
Slope or Within 5 Horizontal Feet of a Slope Face			
Existing Alluvium	200 psf/ft	2,000 psf	0.25

[1] These allowable bearing materials supersede the presumptive materials given within 2016 CBC Section 1809.2. These materials must be undisturbed and verified in the field at the time of construction by the Project Engineering Geologist/Geotechnical Engineer in accordance with 2016 CBC Section 1705.6 and Table 1705.6.

[2] These allowable lateral bearing pressures supersede the presumptive values given within 2016 CBC Table 1806.2.

[2] In accordance with 2016 CBC Section 1806.1, the allowable passive earth pressure indicated above is for static loads (including the total of dead and frequently applied live loads), and may be increased by one-third for short duration loading (including the effects of wind or seismic forces) as allowed in 2016 CBC Section 1805.3.2.

[3] These maximum lateral bearing pressures supersede the presumptive maximum value given within 2016 CBC Section 1806.3.3

[4] These allowable coefficients of friction supersede the presumptive values given within 2016 CBC Table 1806.2. In accordance with 2016 CBC Section 1806.3.2, lateral sliding resistance shall not exceed one-half of the dead load.

3.4.5 Structure Backfill Material

As described previously the onsite soils consist primarily of moderately expansive clay. The expansive clay soil should not be placed as backfill within three feet of the structure. The backfill within three feet of the structure and extending the full height of the structure should consist of granular soil (sand (SP/SW) or silty sand (SM)) with an expansion index (EI) of less than 20 and a sand equivalent (SE) of 30 or greater.

3.5 LATERAL EARTH LOADS FOR BURIED STRUCTURES

Retaining structures free to rotate or translate laterally (e.g., cantilevered retaining walls) through a horizontal distance to wall height ratio of greater than about 0.004 can be assumed as unrestrained or yielding retaining structures. Such walls can generally move enough to develop active conditions. Retaining structures unable to rotate or deflect laterally (e.g., restrained below-grade or basement walls) are referred to as restrained or rigid walls. We have assumed the below grade walls for the pump station will be rigid and should be designed for at-rest conditions.

The presumptive values for lateral soil load given within 2016 CBC, Table 1610.1 shall be superseded by the following site-specific values. These load values are expressed as equivalent fluid densities (applied consistent with CBC requirements) and are intended for structural design of buried walls and retaining walls of up to a maximum of 20 feet in retained height. Based on discussions with PCE staff, we understand the buried pump station also will be designed to resist an H-20 traffic loading on the top and sides of the structure.

Table 6. Lateral Earth Pressures

Average Slope Gradient Above Wall	Active Lateral Soil Load (pcf)[1]	At-Rest Lateral Soil Load (pcf)[2]	Seismically Induced Soil Load (pcf)[3]
Level	60	100	20

[1, 2] These load values supersede the presumptive values for lateral soil loads given within 2016 CBC, Table 1610.1. Additional surcharge from other structures shall be included in the design of the wall. This listed soil pressure is for supporting soils with a prevailing Expansive Index (EI) of no greater than 20. Soils with an EI greater than 20 shall not be used as backfill material. The lateral earth pressures apply to both drained and undrained conditions.

[1] This listed soil pressure assumes the wall will be allowed to deflect between 0.01H to 0.02H, in accordance with 2016 CBC Section 1610.1.

[2] Applicable to restrained wall conditions, in accordance with Section 1610.1 of the 2016 CBC.

[3] In accordance with 2016 CBC Section 1803.5.12, if it is anticipated that the earthquake induced acceleration for the site will exceed 0.4g, then seismic loading may be applied to walls of at least six feet or taller in accordance with Section 1610.1 of the 2016 CBC at the option of the Project Structural Engineer. When utilized, this loading may be applied as an inverted-oriented triangular-load.

3.6 OPEN CUT CONSTRUCTION AND EXCAVATIONS

3.6.1 Existing Utilities

Trenches should be excavated no closer than a 1h:1v projection up from the bottom of the excavation in areas where an existing utility/pipeline parallels or subparallels the trench excavation. The minimum clear distance between an existing utility and the trench should be evaluated by the contractor. We recommend existing utility/pipelines be supported/ protected or that the trench be shored to prevent loss of lateral support for existing utility/pipelines when: 1) the trench is closer than a 1h:1v projection to the existing utility, 2) the stability of the existing utility is in question, or 3) there is a potential for sloughing of the trench sidewalls adjacent to the existing utility. CMWD Standard Drawings 1101, 1102, and 1103 apply to crossing of existing utilities.

3.6.2 Temporary Excavations

Excavations more than four feet deep should be sloped, shored, or shielded in accordance with federal and state standards, project specifications, and safe construction practices. The contractor is responsible for providing and maintaining safe excavations, according to Occupational Safety and Health Administration (OSHA) regulations.

Soft to very stiff sandy clay, clayey silt, and clay; medium dense to dense clayey sand and sand; and siltstone/claystone bedrock of the Monterey Formation were encountered in the explorations advanced for this study in the proposed construction depths (OSHA Type B and C soils). Therefore, per OSHA's 29 CFR Part 1926, unsupported excavations in these soils should be sloped no steeper than 1h:1v in clay soils/claystone or 1.5h:1v in granular soils. In addition, flatter slopes may be warranted depending on exposed soil conditions. Temporary excavations should be monitored for stability during construction and be modified, if necessary. Excavations lacking adequate sidewall support could move or become unstable and result in damage to existing improvements adjacent to the excavations.

3.7 FILL MATERIALS

3.7.1 General Fill

Soil generated during excavation for the pump station likely will need to be processed and blended with granular soil for use as General Fill.

General fill materials should meet the following requirements:

- No rocks larger than six inches in maximum dimension;
- No more than 15 percent material larger than two inches;
- Low expansive potential (EI ≤ 50);
- Plasticity Index less than 15; and
- Less than 60 percent passing the No. 200 sieve.

On the basis of the data from our explorations and review of previous data, we anticipate most of the fine-grained clayey on-site soils will not meet the above criteria for general fill and likely will need to be mixed/blended with granular soil to reduce the fines content and expansion potential for use as general fill. The fine-grained onsite soil also will not meet the criteria for backfill below structures as described in Section 3.4.5.

3.7.2 Backfill Material

In general, backfill should be moisture conditioned to within two percent of optimum, placed in loose lift thicknesses no greater than eight inches, and mechanically compacted. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material prior to placing the next layer. Backfill should be compacted to at least 90 percent relative compaction, as determined from the latest ASTM D1557 test method.

We note the test results from this study indicate the moisture contents of the onsite materials within the anticipated construction zone are above optimum, ranging from about 20 to as high as 33 percent. The test results suggest the moisture contents are eight to 20 percent above optimum moisture content for clay soils (assuming an optimum moisture content of 12 percent). Therefore, the onsite soils likely will need to be dried back to use as compacted fill.

3.7.3 Fill Material Selection

Recommended fill material selection requirements for subgrade fill, aggregate base, and use of onsite materials are presented below. Areas or zones where the various fill materials may be used are described below.

Compacted Fill. The material generated from the overexcavation can be utilized as compacted fill as long as those materials satisfy criteria for the respective types of fill listed below.

General Fill. General fill should consist of granular soil materials (SP, SW, SM, SC, and CL) free of organics, oversize rock (greater than six inches in diameter), trash, debris, and other deleterious or unsuitable materials. The soil should have an expansion index less than 50 (moderate expansion index). The fill materials should have less than 10 percent larger than four inches in diameter and rock fragments larger than six inches should be removed from the fill or broken up into smaller pieces smaller than four inches prior to placement as fill.

Select Fill. The backfill within three feet of the structure (below the structure and adjacent to the walls for the full height of the structure) should consist of granular soil (sand (SP/SW) or silty sand (SM)) with an expansion index (EI) of less than 20 and a sand equivalent of 30 or greater**.**

Aggregate and Miscellaneous Base. Base materials should consist of material conforming to Caltrans Standard Specifications for Class 2 Aggregate Base, Section 26-1.02 (Caltrans, 2018) or Section 200-2.5 of the Greenbook (2018) for Processed Miscellaneous Base.

3.7.4 Imported Fill

Imported fill materials may be used for general fill or select fill, provided the imported fill satisfies the requirements for its intended use. Imported fill material should consist of granular soil such as sand (SP), silty sand (SM), or clayey sand (SC) and can be mixed with the onsite soils prior to placement as compacted fill to reduce the expansion index of the onsite soil. Imported fill material should be evaluated by the project engineer to verify suitability for its intended use prior to being transported to the site. Pipe bedding and pipe zone backfill materials will comply with CMWD Standard Drawing 301.

4.0 LIMITATIONS

4.1 REPORT USE

This report was prepared for exclusive use of Phoenix Civil Engineering, Inc., Calleguas Municipal Water District, and their authorized agents only for the underground pump station for the LVMWD/CMWD Interconnection Project (Project 450). The findings, conclusions, and recommendations presented herein were prepared in accordance with generally accepted geotechnical engineering practices of the project region. No other warranty, express or implied, is made.

Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the project as described in this report, the conclusions and recommendations in this report shall not be considered valid unless the changes are reviewed, and the conclusions and recommendations of this report are modified or validated in writing by OGI.

4.2 HAZARDOUS MATERIALS

This report does not provide information regarding the presence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere.

4.3 LOCAL PRACTICE

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

4.4 PLAN REVIEW

We recommend OGI be provided the opportunity to review and comment on the geotechnical aspects of any project plans and specifications prepared for this project before they are finalized. The purpose of that review will be to evaluate if the recommendations in this report have been properly interpreted and implemented in the design and specifications.

4.5 CONSTRUCTION MONITORING

Users of this report should recognize the construction process is an integral design component with respect to the geotechnical aspects of a project, and geotechnical engineering is inexact due to the variability of natural and man-induced processes, which can produce unanticipated or changed conditions. Proper geotechnical observation and testing during construction is imperative in allowing the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that OGI be retained during project construction to observe compliance with project plans and specifications and to recommend design changes, if needed, in the event that subsurface conditions differ from those anticipated.

5.0 REFERENCES

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______ (2018), https://geohazards.usgs.gov/deaggint

Weber, H.F., Jr., et al. (1973), in *Geology and Mineral Resources Study of Southern Ventura County, California*, California Division of Mines and Geology (CDMG) Preliminary Report No. 14, 102 pp.

PLATES

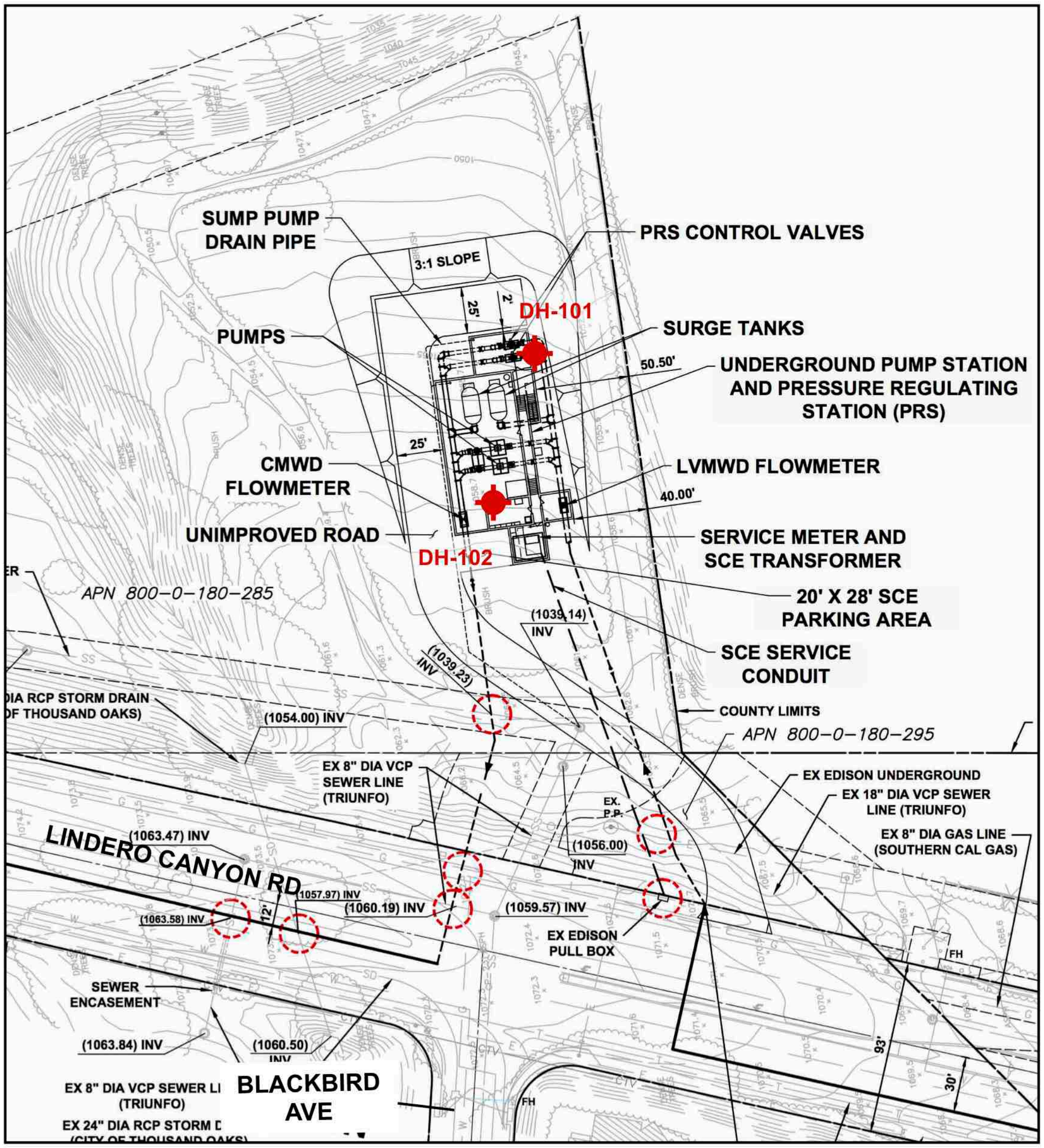
Source: Mapquest (2018)

LOCATION MAP
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

PLATE 1

Phoenix Civil Engineering, Inc.
Project No. 003.001

OAKRIDGE GEOSCIENCE, INC.

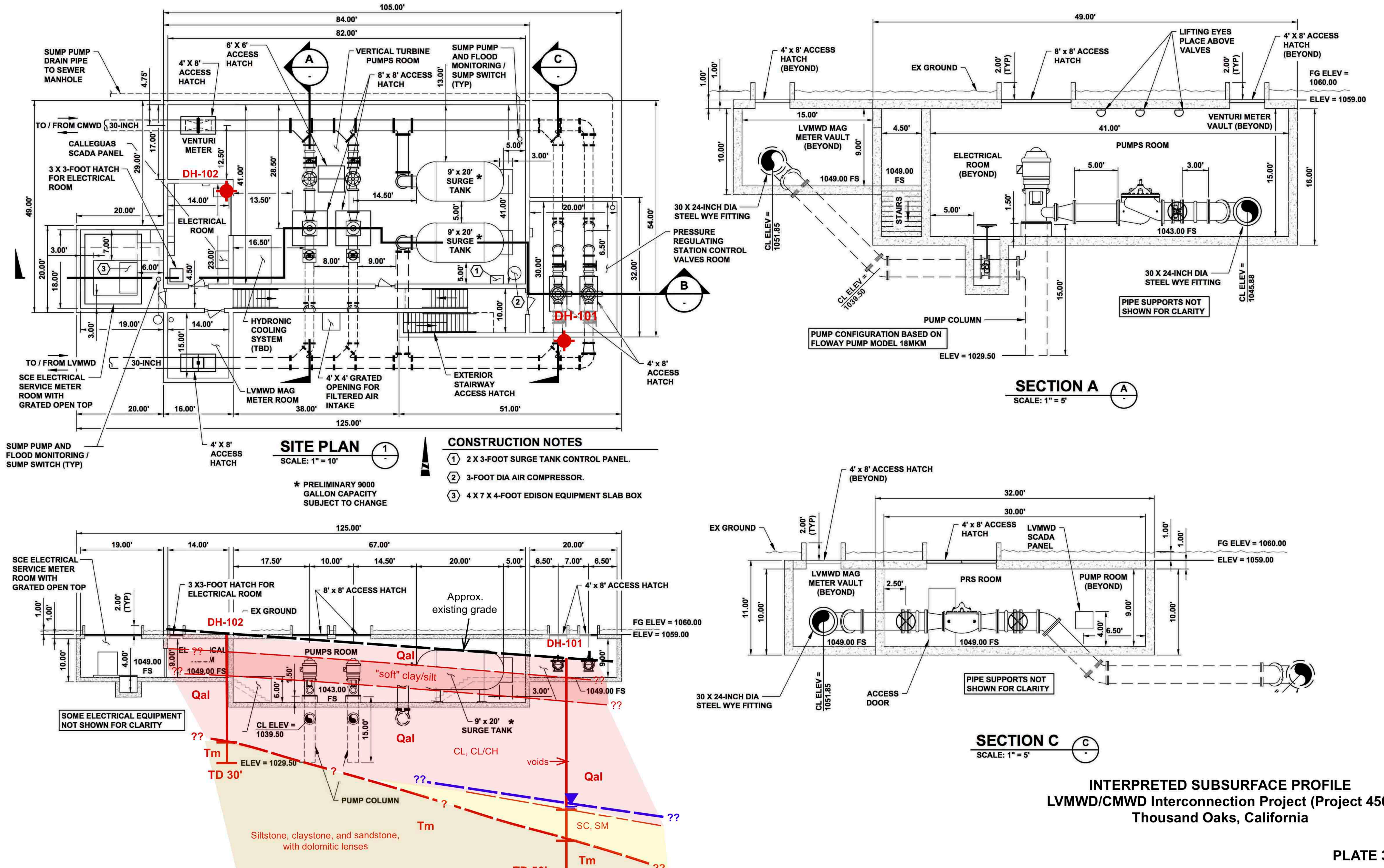


DH-101 Approximate location of Drill Hole advanced for this study

0 50 100
Approx. Scale (feet)

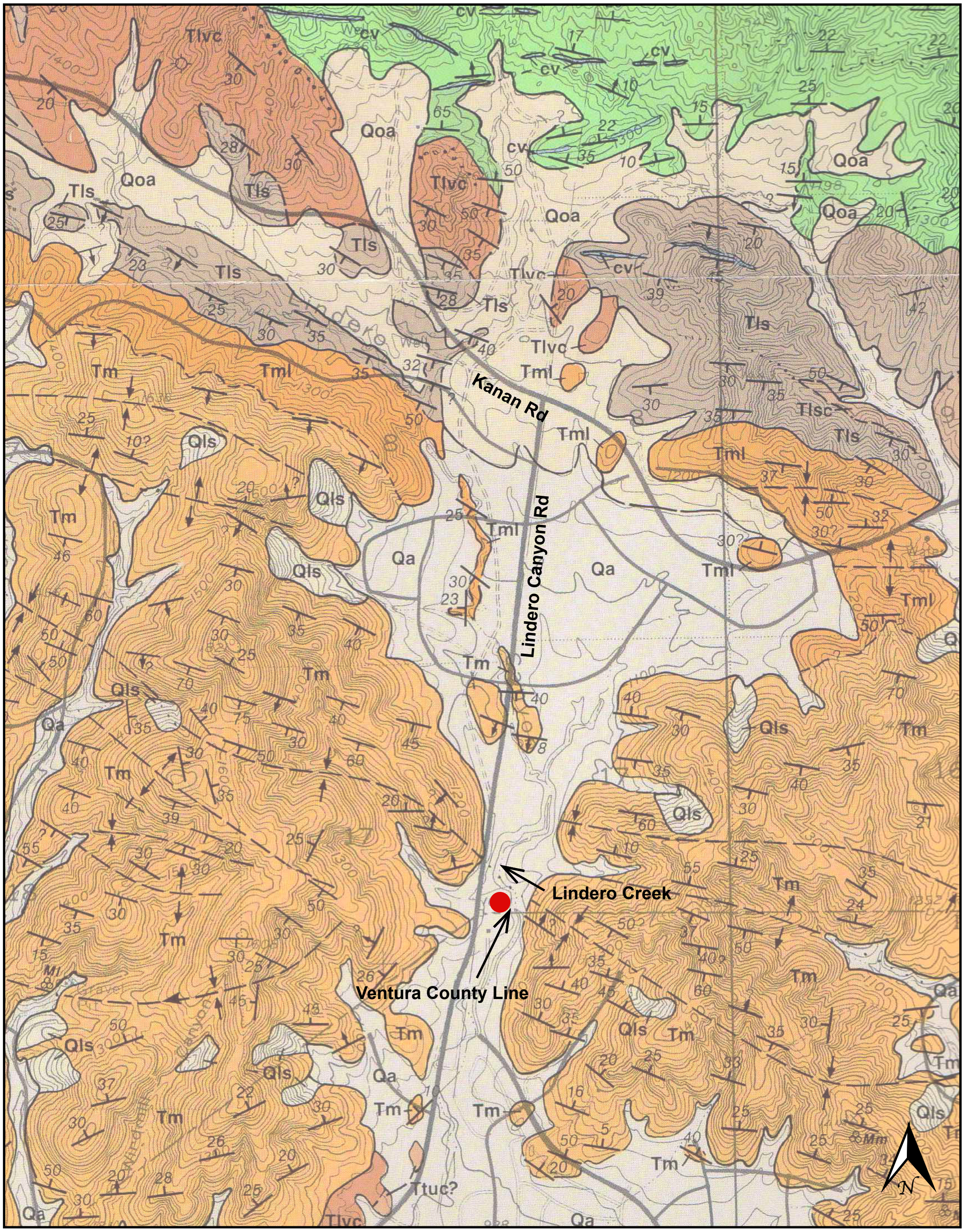
EXPLORATION LOCATION MAP
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

PLATE 2



INTERPRETED SUBSURFACE PROFILE
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

PLATE 3



Source: Dibblee (1993)

Qa - Alluvium
Qoa - Older Alluvium
Qls - Landslide Deposits
Tm/Tml - Monterey Formation
Tls/Tlvc - Topanga Formation
23 Geologic Bedding Orientation
Approximate Pump Station Location

REGIONAL GEOLOGIC MAP
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

APPENDIX A

LOG OF DRILL HOLE DH-101

LOCATION: See location map

SURFACE EL. (ft): 1055' approx.

ELEV. (ft) DEPTH (ft)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
1054			1	--	**ARTIFICIAL FILL (af)** Sandy CLAY (CL): soft, dark brown, dry to damp, with scattered angular siltstone and sandstone angular fragments, and with fine roots		17		60	
2, 1052			2	(8)	**ALLUVIUM (Qal)** Sandy CLAY (CL): medium stiff, dark brown, damp, with fine to coarse fine angular siltstone and sandstone fragments	70	20			
4, 1050, 6			3	4	Sandy Elastic SILT (MH): soft, moderate yellowish brown, damp, with caliche veinlets		23	51/20	65	
1048, 8, 1046			4	(23)	- stiff, with 1/4- to 3/8" angular siltstone fragments, at 7-1/2'	80	22			
10, 1044, 12, 1042			5	10	CLAY (CL/CH): stiff, dark brown, damp, with scattered 1/4- to 1/2" angular siltstone and sandstone fragments and fine caliche veins		24		72	
14, 1040, 16, 1038, 18			6	(33)	- very stiff, increased moisture content, increased content of finer angular rock fragments, with few coarser rock fragments, and with scattered fine charcoal inclusions, at 14'	92	29			p>4.5
1036			7	11	Fat CLAY (CH)/Elastic Silt (MH) with Gravel: stiff, dark brown, moist, angular siltstone fragments, w/clayey gravel lens at about 19-1/2'		33	53/24	39	

CONTRACTOR:	S/G Drilling	TOTAL DEPTH (ft):	50'
METHOD:	8" hollowstem auger	WATER DEPTH (ft):	33.3'
BACKFILL:	Cuttings	LOGGED BY:	L Prentice
DATE:	October 2, 2018	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-101 (Continued)

LOCATION: See location map
SURFACE EL. (ft): 1055' approx.

ELEV. (ft) DEPTH (ft)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
			7	11						
1034										
22										
1032										
24					- stiff, increased moisture content, with scattered voids, at 24'					p2.0
1030			8	(18)	- with fine angular rock fragment/gravel lens, at about 24-1/2'		30			p2.5
										p3.0
26										
1028										
28										
1026										
30			9	8	- medium stiff, at 29'					
1024										
32										
1022					- measured water at about 33.3' after sampling at 35'					
34					Clayey SAND (SC) with Gravel: medium dense, moderate to dark brown, wet					
1020			10	(46)	Silty SAND (SM): medium dense, moderate brown, wet, medium sand	116	11			
36										
1018										
38										
1016										
			11a	42			29		17	

CONTRACTOR:	S/G Drilling	NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.	TOTAL DEPTH (ft):	50'
METHOD:	8" hollowstem auger		WATER DEPTH (ft):	33.3'
BACKFILL:	Cuttings		LOGGED BY:	L Prentice
DATE:	October 2, 2018		CHECKED BY:	C Prentice

LVMWD/CMWD INTERCONNECTION PROJECT (PROJECT 450)
Thousand Oaks, California

LOG OF DRILL HOLE DH-101 (Continued)

LOCATION: See location map
SURFACE EL. (ft): 1055' approx.

ELEV. (ft)	DEPTH (ft)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
				11b	42	Clayey SAND (SC) with Gravel: dense, moderate brown, wet, angular siltstone fragments		23		28	
1014											
	42					- driller noted harder drilling at about 42'					
						MONTEREY FORMATION (Tm)					
1012											
	44					SILTSTONE (Rx): extremely weathered, low hardness, dark brownish black to black, platy/slatey, approx. 45 degree apparent dip [SILT (ML) thinly interbedded with Silty SAND (SM): hard, damp to moist]					
1010				12 12a	(90:10)			27			
	46										
1008											
	48										
1006						- interbedded with light bluish gray sandstone layers about 1 to 2" thick at about 6" intervals, at 49'					
	50			13	92:11			22			
1004											
	52										
1002											
	54										
1000											
	56										
998											
	58										
996											

CONTRACTOR:	S/G Drilling	TOTAL DEPTH (ft):	50'
METHOD:	8" hollowstem auger	WATER DEPTH (ft):	33.3'
BACKFILL:	Cuttings	LOGGED BY:	L Prentice
DATE:	October 2, 2018	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-102

LOCATION: See location map
SURFACE EL. (ft): 1059' approx.

ELEV. (ft) DEPTH (ft)	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
1058			**ARTIFICIAL FILL (af)** CLAY (CL) with Gravel: soft, dark brown, dry, with angular siltstone fragments					
2								
1056	1	(21)	**ALLUVIUM (Qal)** SILT (ML): stiff, dark brown, dry to damp, with angular siltstone and sandstone fragments, and fine caliche veins		23	48/18	64	
4								
1054								
6	2	4	Clayey SILT (ML): soft, moderate yellowish brown, dry to damp, with fine sand and few fine caliche veins					
1052								
8	3	(16)	CLAY (CL): stiff, dark brown, damp, with angular siltstone and sandstone fragments to about 1/2" and with fine caliche veins	78	26	42/17		
1050								
10			- medium stiff, sandy, at 10'					
1048	4	8			25		58	
12								
1046								
14			- very stiff, at 14'					
1044	5	(27)		96	25			
16								
1042								
18								
1040	6	12	CLAY (CL) with Gravel: stiff, dark brown, moist, fine to coarse angular siltstone and sandstone fragments		28		56	

CONTRACTOR:	S/G Drilling	TOTAL DEPTH (ft):	30'
METHOD:	8" hollowstem auger	WATER DEPTH (ft):	Not encountered
BACKFILL:	Cuttings	LOGGED BY:	L Prentice
DATE:	October 2, 2018	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LVMWD/CMWD INTERCONNECTION PROJECT (PROJECT 450)
Thousand Oaks, California

LOG OF DRILL HOLE DH-102 (Continued)

LOCATION: See location map

SURFACE EL. (ft): 1059' approx.

ELEV. (ft) DEPTH (ft)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
			6	12						
1038										
22										
1036										
24					- sampler bouncing, at 25-1/4'					
					MONTEREY FORMATION (Tm)					
1034			7	(36/ 35:3)	SILTSTONE interbedded with CLAYSTONE (Rx): extremely weathered, low hardness, moderate yellowish brown, with iron oxide staining [SILT (ML) interbedded with CLAY (CL), hard, damp to moist]	87	28			
26										
1032										
28										
1030					- increased induration, with dolomitic lenses, at 29-1/2'					
30			8	50:5						
1028										
32										
1026										
34										
1024										
36										
1022										
38										
1020										

CONTRACTOR:	S/G Drilling	TOTAL DEPTH (ft):	30'
METHOD:	8" hollowstem auger	WATER DEPTH (ft):	Not encountered
BACKFILL:	Cuttings	LOGGED BY:	L Prentice
DATE:	October 2, 2018	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

SUMMARY OF SAMPLING DETAILS

Symbol	Sample Number	Blowcount Push, or Grab	Sampler Type
	1	Bulk	Bulk Sample
	2	23	Standard Penetration Test (SPT) Sampler (1-3/8" ID/2" OD) driven
	3	(23)	Modified California Liner Sampler driven (2-3/8" ID/3" OD)
	4	Push	Thin-walled sampler pushed (2-7/8" ID/3" OD)
	5	(23)	Modified California Liner Sampler (disturbed) driven (2-3/8" ID/3" OD)

Blowcount Information

Blowcount	Description
63	63 blows for 1' penetration after initial 6" seating
89/11	89 blows for 11" penetration after initial 6" seating
33/6	33 blows for 6" drive after initial 6" seating
Ref	>50 blows for initial 6" seating
(23)	Blowcounts for modified California sampler

MATERIAL SYMBOLS AND CLASSIFICATIONS

- Lean CLAY (CL)
- Fat CLAY (CH)
- Sandy CLAY (CL)
- SILT (ML)
- Elastic SILT (MH)
- Clayey SILT (ML)
- Clayey GRAVEL (GP and GW)
- Sandy SILT (ML)
- Silty SAND (SM)
- SAND with Silt (SP-SM and SW-SM)
- SAND (SP and SW)
- Clayey SAND (SC)
- SAND with Clay (SP-SC and SW-SC)
- CLAYSTONE
- SILTSTONE
- SANDSTONE
- VOLCANIC
- DOLOMITIC
- SILICEOUS
- PAVING AND BASE MATERIALS
- CONCRETE
- GRAVEL (GP and GW)
- GRAVEL with Sand (GP and GW)
- SAND with Gravel (SP and SW)
- SAND with Silt and Gravel (SP-SM and SW-SM)
- Silty SAND with Gravel (SM)
- Clayey SAND with Gravel (SC)

Other Symbols

- Groundwater
- Strata break

SUMMARY OF TERMS AND SYMBOLS USED ON LOGS

Summary of Rock Logging Descriptions

Weathering for Intact Rock (after USBR 2001)

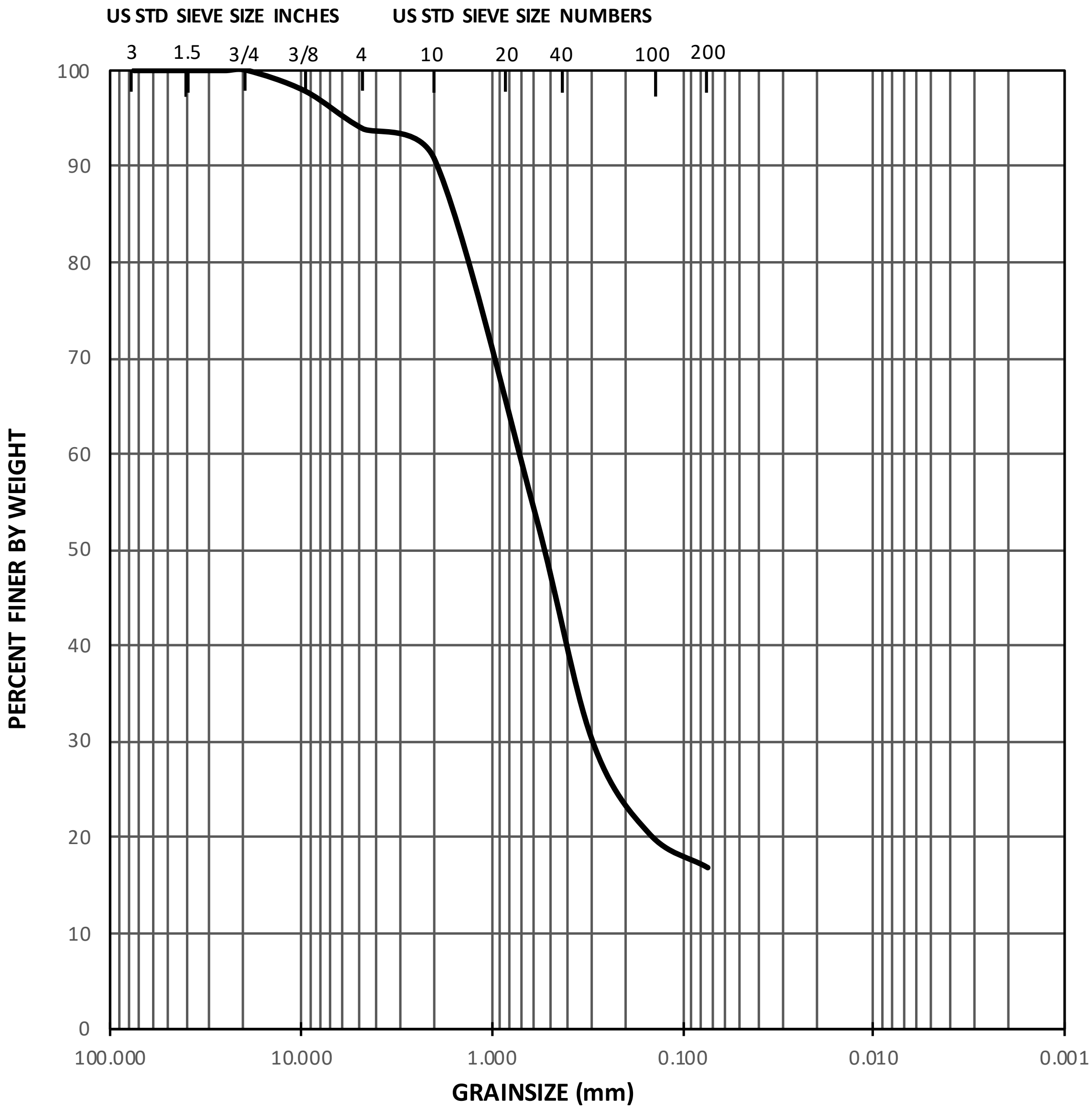
Descriptors	Diagnostic Features					General characteristics (strength, excavation, etc.) §
	Chemical weathering-Discoloration And/or oxidation		Mechanical weathering- Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and solutioning		
	Body of rock	Fracture Surfaces		Texture	Leaching	
Fresh	No discoloration, not Oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No leaching	Hammer rings when crystalline rocks are struck. Almost always rock excavation except for naturally weak or weakly cemented rocks such as siltstones or shales.
Slightly weathered	Discoloration or oxidation is Limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are stuck. Body of rock not weakened. With few exceptions, such as siltstones or shales, classified as rock excavation.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty" feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened. Depending on fracturing, usually is rock excavation except in naturally weak rocks such as siltstones or shales.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions	All fracture surfaces are discolored or oxidized, surfaces friable	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated	Texture altered by chemical disintegra -tion (hy-dration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened. Usually common excavation.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Always common excavation. Resistant minerals such as quartz may be present as "stringers" or "dikes."

Rock Hardness (after USBR 2001)

Descriptor	Criteria
Extremely hard	Cannot be scratched with a pocketknife or sharp knife. Can only be chipped with repeated hammer blows.
Very hard	Cannot be scratched with a pocketknife. Breaks with repeated hammer blows.
Hard	Can be scratched with a pocketknife with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately hard	Can be scratched with a pocketknife with light or moderate pressure. Breaks with light hammer blow or heavy manual pressure.
Moderately soft	Can be grooved 2 mm (1/6 inch) deep with a pocketknife with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocketknife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

SUMMARY OF ROCK TERMS USED ON LOGS

APPENDIX B



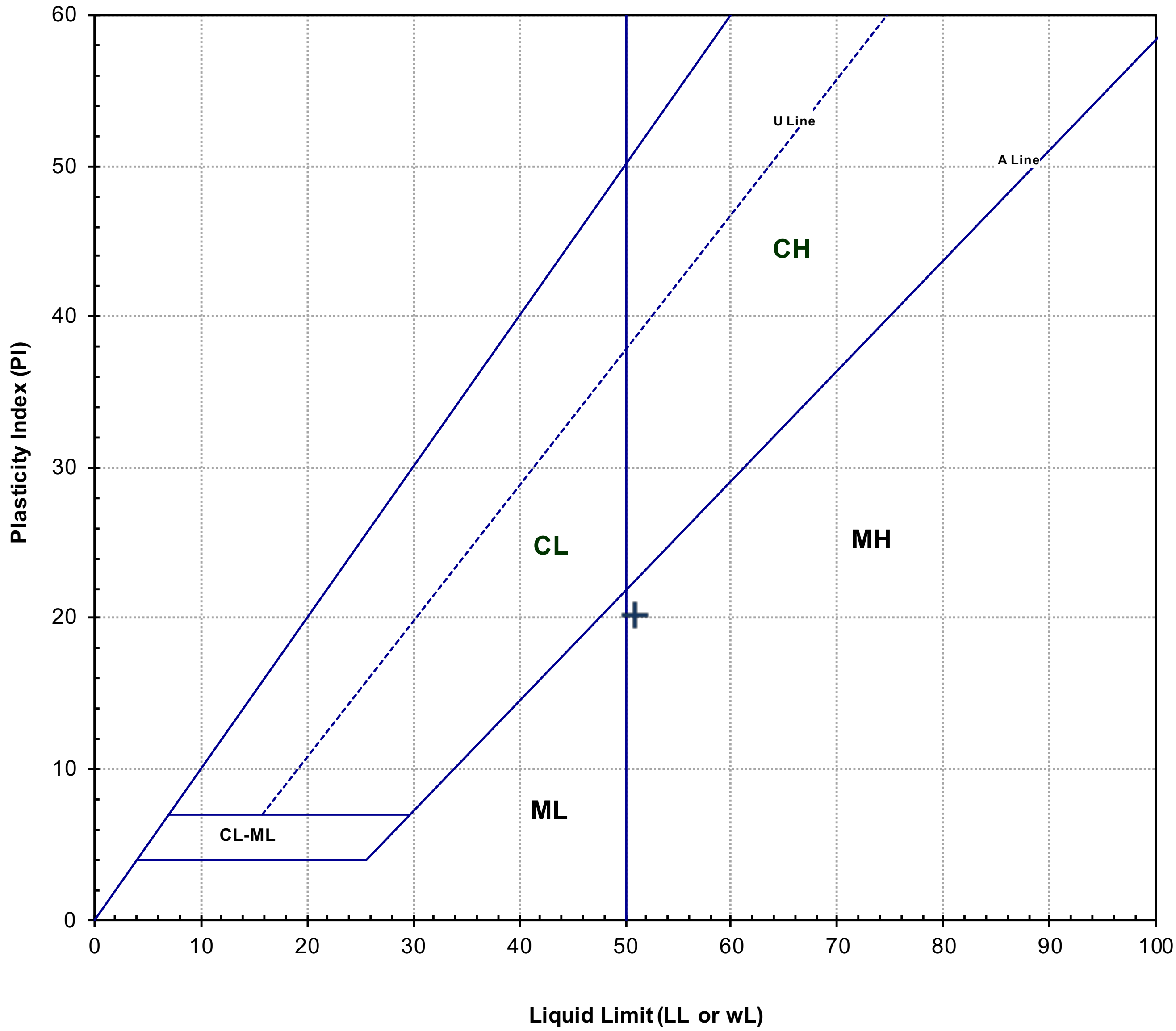
GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LOCATION DH-101

DEPTH 40

CLASSIFICATION	PASSING NO. 200 (%)
Silty SAND (SM)	17

GRAINSIZE DISTRIBUTION
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

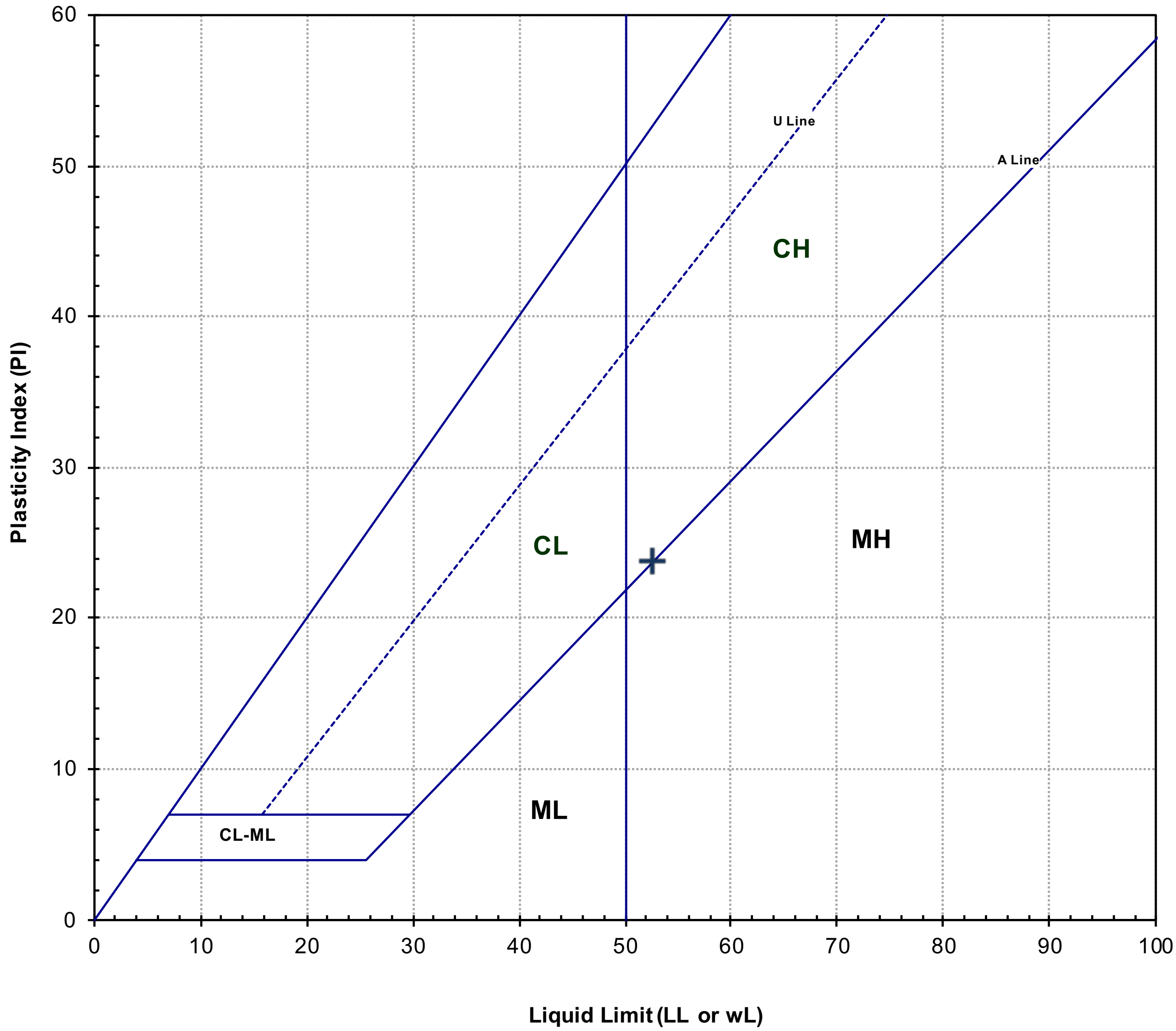


			LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
LOCATION	DH-101	**CLASSIFICATION**			
DEPTH	5'	Sandy Elastic SILT (MH)	51	31	20

PLASTICITY CHART

LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California

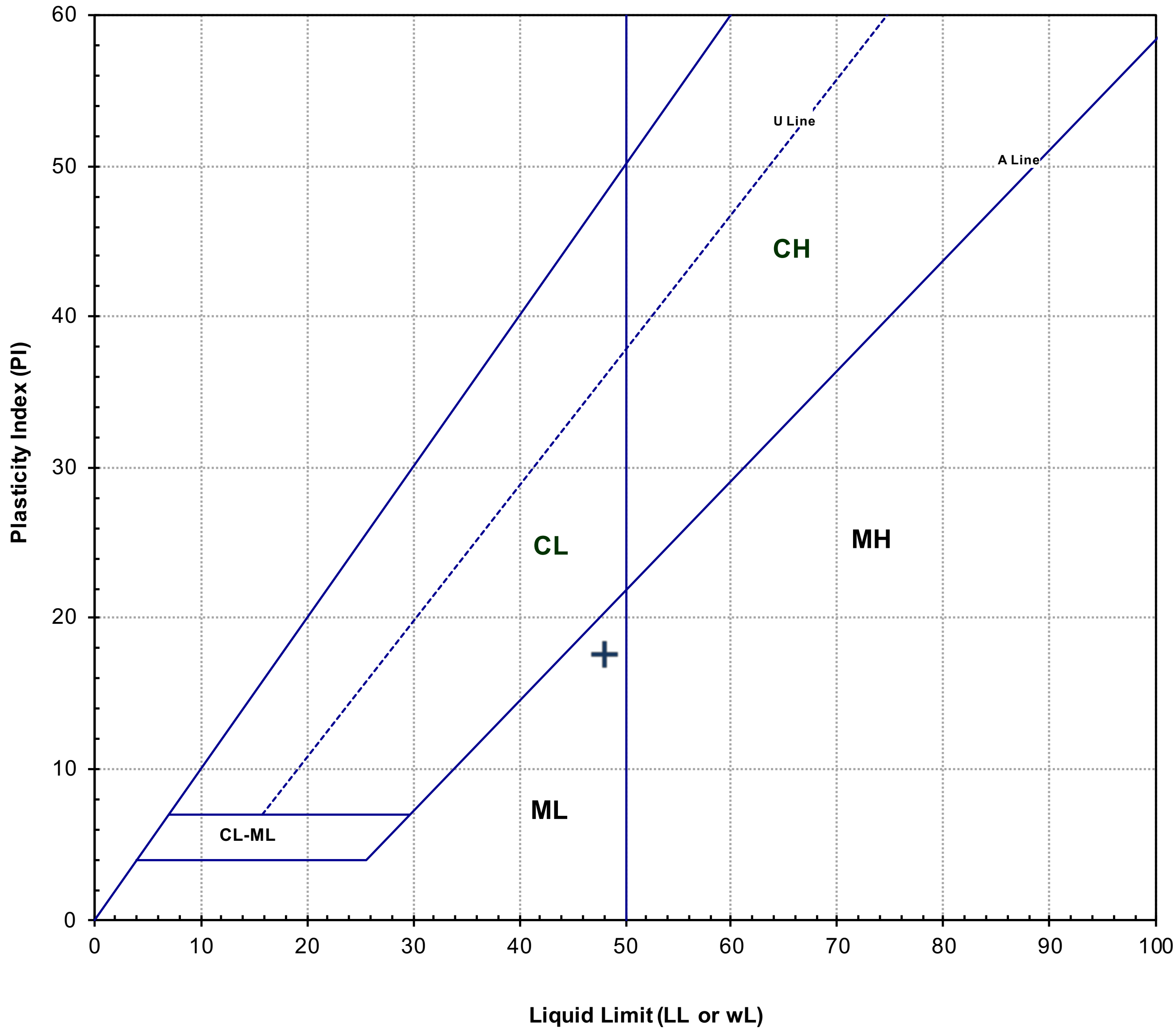
PLATE B-2a



			LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
LOCATION	DH-101	**CLASSIFICATION**			
DEPTH	20'	Fat CLAY (CH)/Elastic Silt (MH) with Gravel	53	29	24

PLASTICITY CHART

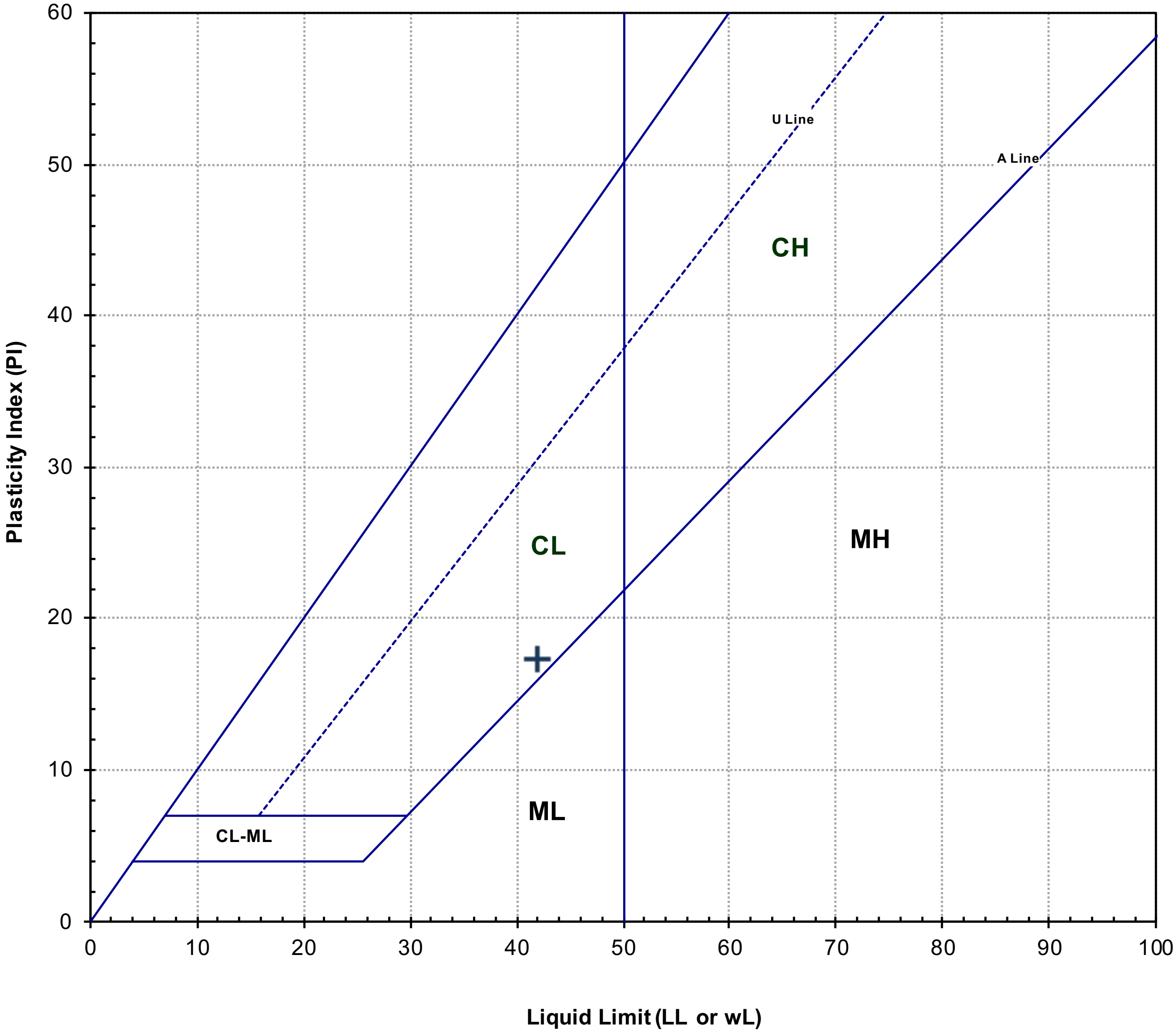
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California



			LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
		CLASSIFICATION			
LOCATION	DH-102				
DEPTH	5'	SILT (ML)	48	30	18

PLASTICITY CHART

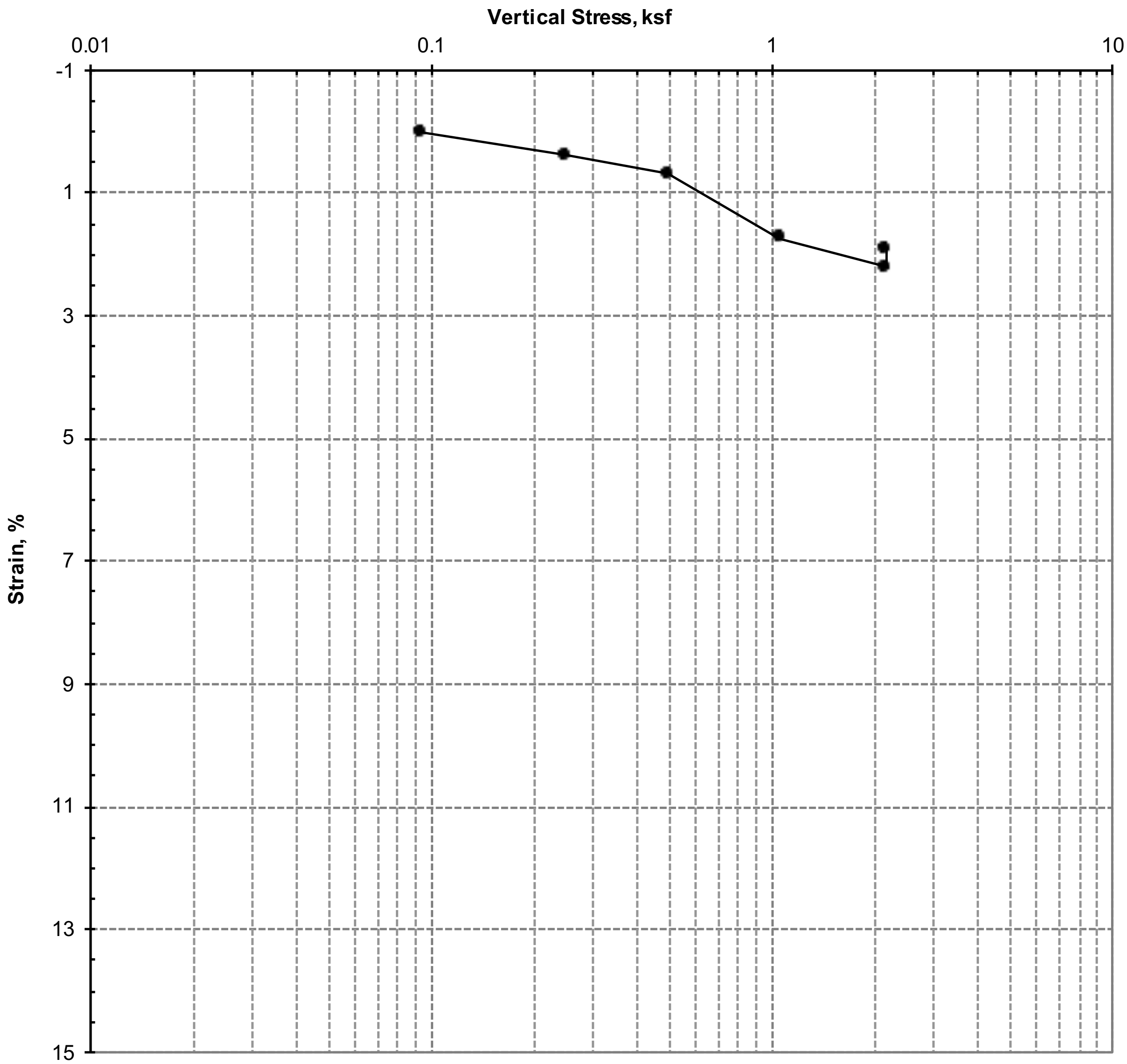
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California



		CLASSIFICATION	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
LOCATION	DH-102				
DEPTH	5'	CLAY (CL)	42	25	17

PLASTICITY CHART

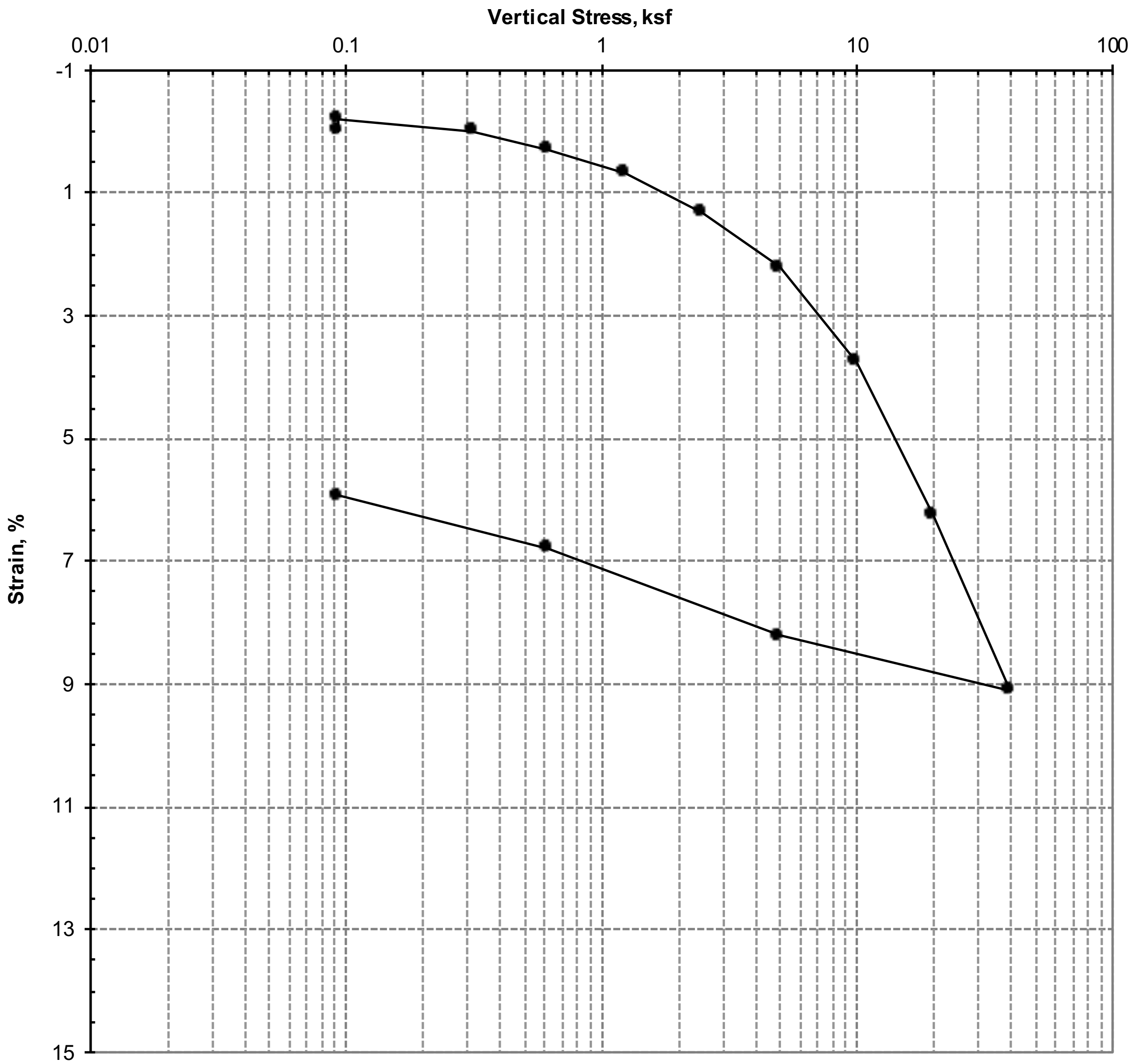
LVMWD/CMWD Interconnection Project (Project 450)
Thousand Oaks, California



SAMPLE ID			SUMMARY		
Boring, Sample #, Depth	DH-101 , #8 , 24.0 ft		Preconsolidation Pressure, ksf		---
USCS Classification:	Sandy fat CLAY (CH): dark brown, moist		Inundation Increment, ksf		2.17
PROPERTIES	Initial	Final	Liquid Limit		---
Water Content, %	35.9%	36.2%	Plastic Limit		---
Dry Unit Weight, pcf	82.6	84.2	Plasticity Index		---
Saturation, %	93%	98%	Passing #200		---
Void Ratio	1.04	1.00	Estimated Gs		2.7
Diameter, in	2.42	2.42	**REMARKS**	Test Method: ASTM D2435	
Height, in	0.89	0.87		Project: PCE / CMWD PS	

INCREMENTAL CONSOLIDATION TEST

PLATE B-3



SAMPLE ID		
	Boring, Sample #, Depth	DH-101 , #8 , 24.0 ft
	USCS Classification:	Sandy fat CLAY (CH): dark brown, moist

PROPERTIES	Initial	Final
Water Content, %	47.6%	44.1%
Dry Unit Weight, pcf	71.3	75.8
Saturation, %	94%	97%
Void Ratio	1.36	1.22
Diameter, in	2.42	2.42
Height, in	1.08	1.01

SUMMARY	
Preconsolidation Pressure, ksf	---
Inundation Increment, ksf	0.10
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Passing #200	---
Estimated Gs	2.7

REMARKS
Test Method: ASTM D2435
Project: PCE / CMWD PS

INCREMENTAL CONSOLIDATION TEST

PLATE B-4

Project Name	CMWD/LVMWD	Project No.	003.001
Tested By	ND	Testing Date	10/14/18

SPECIMEN ID AND CLASSIFICATION					
Boring No.	DH-101	Sample No.	1	Depth (ft)	0-5
Soil Description	Lean CLAY (CL): brown, moist				

TEST DATA		
	Molding	After Soaking
Moisture Tin ID:	T-32	T-15
Tin Mass (g)	478.22	633.72
Moist Soil + Tin (g)	707.02	1009.23
Dry Soil + Tin (g)	672.3	908.34
Ring Height (in.)	1.000	--
Ring Diameter (in.)	4.000	--
Ring Mass (g)	200.73	--
Ring + Soil Mass (g)	533.6	--

DIAL READINGS			
Date	Time	Reading	Remarks
10/14/18	1:42	0.0001	No Water
			Water
			Water
			Water
			Water
10/17/18	12:05	0.0541	Water
10/17/18	12:05	0.0541	Final

RESULTS		
	Molding	After Soaking
Moisture Content	17.9%	36.7%
Dry Unit Weight (pcf)	85.6	81.2
Saturation	49.9%	92.3%
Expansion Index	54	

APPENDIX C

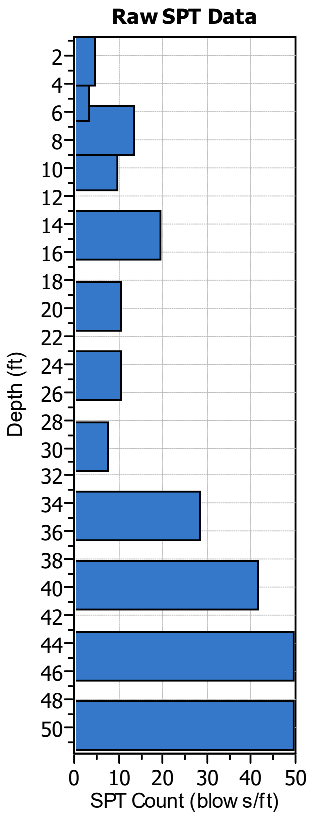
SPT BASED LIQUEFACTION ANALYSIS REPORT

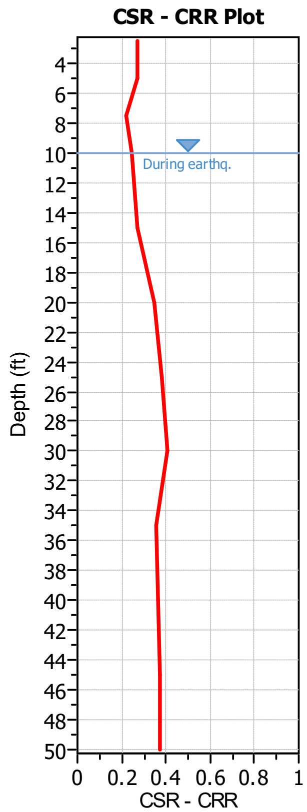
Project title CMWD/LVMWD Interconnection **SPT Name: DH-101**

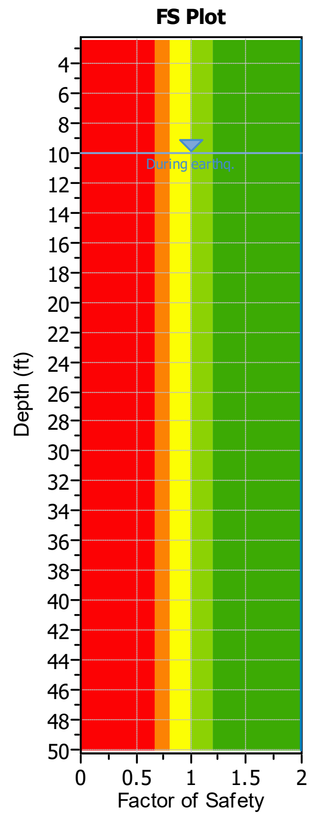
Location : Oak Park, Ventura County, CA

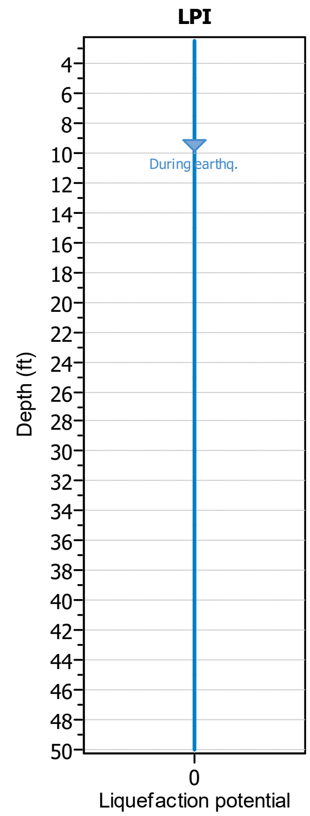
:: Input parameters and analysis properties ::

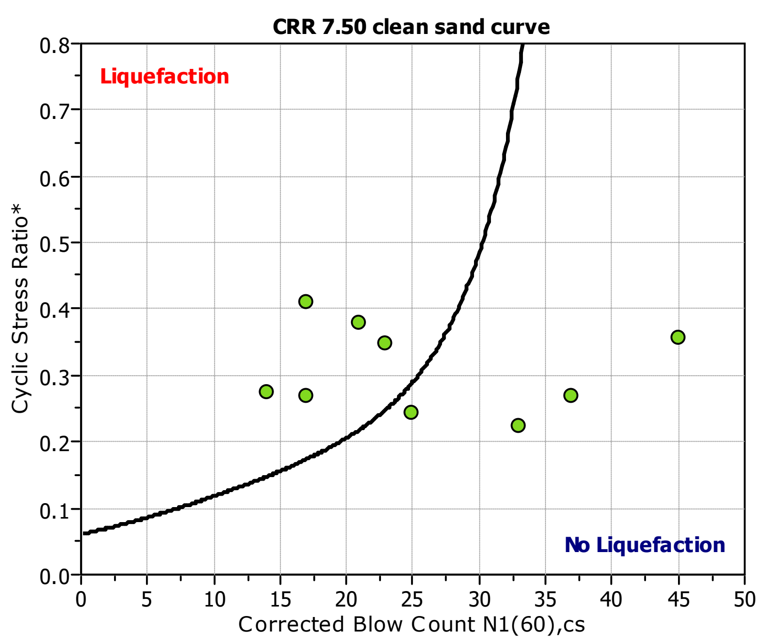
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	33.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	10.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.80 ft
Borehole diameter:	200mm	Peak ground acceleration:	0.50 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.20		











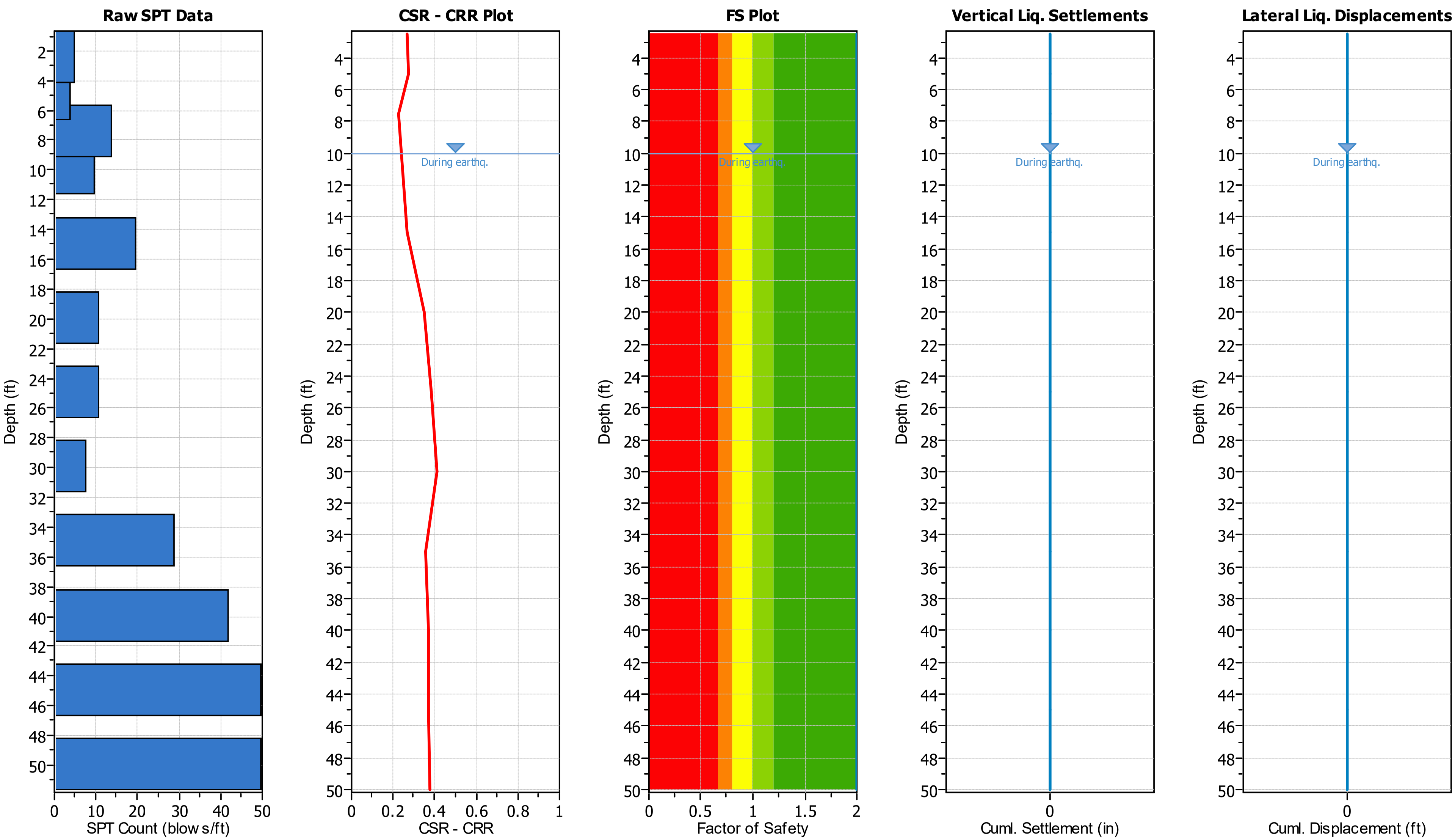
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
2.50	5	60.00	90.00	2.50	No
5.00	4	65.00	90.00	2.50	No
7.50	14	65.00	100.00	2.50	No
10.00	10	72.00	100.00	2.50	No
15.00	20	72.00	120.00	5.00	No
20.00	11	70.00	120.00	5.00	No
25.00	11	70.00	120.00	5.00	No
30.00	8	70.00	120.00	5.00	No
35.00	29	17.00	120.00	4.00	Yes
40.00	42	28.00	120.00	6.00	Yes
45.00	50	70.00	120.00	5.00	No
50.00	50	70.00	120.00	5.00	No

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
2.50	5	90.00	0.11	0.00	0.11	0.44	1.70	1.20	1.15	0.75	1.20	11	60.00	5.60	17	4.000
5.00	4	90.00	0.23	0.00	0.23	0.46	1.70	1.20	1.15	0.75	1.20	8	65.00	5.59	14	4.000
7.50	14	100.00	0.35	0.00	0.35	0.33	1.44	1.20	1.15	0.80	1.20	27	65.00	5.59	33	4.000
10.00	10	100.00	0.47	0.00	0.47	0.39	1.36	1.20	1.15	0.85	1.20	19	72.00	5.57	25	4.000
15.00	20	120.00	0.78	0.00	0.78	0.32	1.10	1.20	1.15	0.85	1.20	31	72.00	5.57	37	4.000
20.00	11	120.00	1.07	0.00	1.07	0.47	0.99	1.20	1.15	0.95	1.20	17	70.00	5.57	23	4.000
25.00	11	120.00	1.38	0.00	1.38	0.44	0.89	1.20	1.15	0.95	1.20	15	70.00	5.57	21	4.000
30.00	8	120.00	1.68	0.00	1.68	0.49	0.80	1.20	1.15	1.00	1.20	11	70.00	5.57	17	4.000
35.00	29	120.00	1.98	0.06	1.91	0.27	0.85	1.20	1.15	1.00	1.20	41	17.00	3.85	45	4.000
40.00	42	120.00	2.27	0.22	2.06	0.15	0.90	1.20	1.15	1.00	1.20	63	28.00	5.27	68	4.000
45.00	50	120.00	2.58	0.37	2.20	0.08	0.94	1.20	1.15	1.00	1.20	78	70.00	5.57	84	4.000
50.00	50	120.00	2.88	0.53	2.34	0.09	0.93	1.20	1.15	1.00	1.20	77	70.00	5.57	83	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
2.50	90.00	0.11	0.00	0.11	1.00	1.00	0.325	1.38	17	1.10	0.296	1.10	0.269	2.000	●
5.00	90.00	0.23	0.00	0.23	0.99	1.00	0.322	1.29	14	1.07	0.300	1.10	0.273	2.000	●
7.50	100.00	0.35	0.00	0.35	0.98	1.00	0.319	2.19	33	1.30	0.245	1.10	0.223	2.000	●
10.00	100.00	0.47	0.00	0.47	0.97	1.00	0.315	1.72	25	1.18	0.267	1.10	0.243	2.000	●
15.00	120.00	0.78	0.16	0.62	0.95	1.00	0.386	2.20	37	1.30	0.296	1.10	0.269	2.000	●
20.00	120.00	1.07	0.31	0.76	0.92	1.00	0.422	1.62	23	1.16	0.365	1.05	0.348	2.000	●
25.00	120.00	1.38	0.47	0.91	0.89	1.00	0.441	1.53	21	1.14	0.388	1.02	0.380	2.000	●
30.00	120.00	1.68	0.62	1.05	0.87	1.00	0.449	1.38	17	1.10	0.409	1.00	0.409	2.000	●
35.00	120.00	1.98	0.78	1.20	0.84	1.00	0.449	2.20	45	1.30	0.345	0.96	0.357	2.000	●
40.00	120.00	2.27	0.94	1.34	0.81	1.00	0.445	2.20	68	1.30	0.341	0.93	0.367	2.000	●
45.00	120.00	2.58	1.09	1.48	0.78	1.00	0.438	2.20	84	1.30	0.336	0.90	0.373	2.000	●
50.00	120.00	2.88	1.25	1.63	0.75	1.00	0.429	2.20	83	1.30	0.329	0.87	0.377	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted
FS:	Calculated factor of safety against soil liquefaction

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
2.50	2.000	0.00	9.62	2.50	0.00
5.00	2.000	0.00	9.24	2.50	0.00
7.50	2.000	0.00	8.86	2.50	0.00
10.00	2.000	0.00	8.48	2.50	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
2.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
5.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
7.50	27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
10.00	25	0.00	0.00	2.000	0.00	0.00	2.50	0.000	0.00
15.00	37	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
20.00	23	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
25.00	21	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	17	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
35.00	45	0.25	-1.19	2.000	0.00	0.00	4.00	0.000	0.00
40.00	68	0.00	-3.12	2.000	0.00	0.00	6.00	0.000	0.00
45.00	84	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
50.00	83	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 0.000 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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