GEOTECHNICAL EXPLORATION REPORT
PROPOSED POLICE HEADQUARTERS
312 E. ALISAL STREET
SALINAS, CALIFORNIA

Prepared for:

Griffin Structures, Inc.

2 Technology, Suite 150
Irvine, California 92618

Project No. 11693.001

August 31, 2017

Leighton Consulting, Inc.

A LEIGHTON GROUP COMPANY

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2 Technology, Suite 150
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Attention: Mr. Gary Chubb

Subject: Geotechnical Exploration Report
Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Per your request and authorization, Leighton Consulting, Inc. (Leighton) is pleased to present this geotechnical exploration report for the proposed Police Headquarters located at 312 E. Alisal Street in the city of Salinas, California. The scope of work for this geotechnical exploration was outlined in our proposal dated June 20, 2017.

We understand that the proposed improvements for the Police Headquarters will include demolition of the existing structures and improvements to allow construction of a new 2-story Police Headquarters Building (44,275 square feet), a new 1-story Support Building (32,500 square feet), associated paved access roads and 371 parking stalls, and secure access entry points. It is our understanding that the southern portion of the project site is planned for future expansion. Other improvements will consist of underground utilities, landscape improvements and WQMP improvements.

Based on our exploration and analysis, the proposed improvements are considered feasible from a geotechnical standpoint. Presented in this report are findings and geotechnical recommendations to aid in the design and construction of the proposed improvements.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ 949.250.1114 Fax

We appreciate this opportunity to be of service. If you have any questions regarding this report or if we can be of further service, please call us at your convenience at **(866) *LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Jeffrey M. Pflueger, PG, CEG 2499
Associate Geologist
Ext 4257; jpflueger@leightongroup.com

Carl C. Kim, PE, GE 2620
Senior Principal Engineer
Ext 4262; ckim@leightongroup.com

JMP/CCK/lr

Distribution: (1) Addressee

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site for the proposed Police Headquarters is located at 312 E. Alisal Street in the city of Salinas, California (Figure 1, *Site Location Map*). It is bounded by E. Alisal Street and Murphy Street to the north, Murphy Street to the west, Southern Pacific Railroad to the southwest, and existing commercial properties and Work Street to the east. The eastern majority of the site consists of several one-story buildings, covered maintenance and/or storage structures, and asphalt paved access drives and parking areas associated with the former Monterey County Public Works facility. This portion of the site is currently being used as a storage facility for a waste management company. The western portion of the site consists of a gravel covered vacant parcel currently being used as a storage area for a trucking company.

Based on a review of historic aerial photos, the site appears to have been in roughly its current configuration since at least 1968 (NETR, 2017).

Based on review of the Site Plan for the project prepared by LPA, Inc., dated June 5, 2017, we understand that the proposed improvements for the Police Headquarters will include demolition of the existing structures and improvements to allow construction of a new 2-story Police Headquarters Building (44,275 square feet), a new 1-story Support Building (32,500 square feet), associated paved access roads and 371 parking stalls, and secure access entry points. It is our understanding that the southern portion of the project site is planned for future expansion. Other improvements will consist of underground utilities, landscape improvements and WQMP improvements. At this time, both the type of structure and loading information of the proposed building are not yet available for review. Based on the existing site conditions, we anticipate only minor grading will be required to prepare the site for the new construction.

The ground surface at the site is relatively flat but gently sloping with elevations across the site that range between approximate Elevations +50 and +53 feet mean sea level (msl). Review of the Salinas, California Quadrangle Topographic Map (USGS, 1947, Photorevised 1984) indicates sheet flow is generally toward the northwest.

1.2 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to evaluate the soil and groundwater conditions at the site through review of available data and exploratory borings in order to provide geotechnical recommendations to aid in design and construction for the project as currently proposed.

The scope of this geotechnical exploration included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the project site. References reviewed in preparation of this report are listed in Section 7.0.

- *Field Exploration* – Our field exploration was performed on July 13 and 14, 2017, and consisted of three (3) hollow-stem auger borings (designated LB-1 through LB-3) drilled to depths between approximately 10 and 51 feet below existing ground surface (bgs). In addition, ten (10) Cone Penetration Test (CPT soundings were advanced to depths between approximately 42 and 51 feet bgs. The approximate locations of the borings and CPTs are shown on Figure 2, *Exploration Location Map*. Prior to the field exploration, the borings and CPTs were marked and Underground Service Alert (USA) was notified for utility clearance.

  During drilling of the hollow-stem auger borings (LB-1 through LB-3), both bulk and drive samples were obtained from the borings for geotechnical laboratory testing. Drive samples were collected from the borings using a Modified California Ring sampler conducted in accordance with ASTM Test Method D3550. Standard Penetration Tests (SPTs) were also performed within the hollow-stem auger borings in accordance with ASTM Test Method D1586. The samplers were driven for a total penetration of 18 inches, unless practical refusal was encountered, using a 140-pound automatic hammer falling freely for 30 inches. The number of blows per 6 inches of penetration was recorded on the boring logs.

  The borings were logged in the field by a member of our technical staff. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory. After completion of drilling, two of the borings (LB-1 and LB-2) were backfilled with cement-bentonite grout to approximately 5 feet bgs per the approved Monterey County Health Department (MCHD) Well Permit. It should be noted that a representative from MCHD was onsite during the backfill operations to verify that the borings were backfilled per the permit requirements. The remaining boring (LB-3) was converted to temporary percolation test well for subsequent percolation testing, and backfilled with excess soil cuttings after completion of the percolation test. The CPTs were backfilled with bentonite grout. The boring and CPT logs are presented in Appendix A, *Field Exploration Logs*.

- *Laboratory Testing* –Laboratory tests were performed on representative soil samples to evaluate geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

  - In-situ Moisture Content and Dry Density (ASTM D2216 and ASTM D2937);
  - Atterberg Limits (ASTM D4318);
  - Expansion Index (ASTM D4829);
  - Modified Proctor Compaction Test (ASTM D1557);
  - Direct Shear (ASTM D 3080);
  - Consolidation (ASTM D2435);
  - R-value (DOT CA 301); and
  - Corrosivity (Soluble Sulfate ASTM C1580, Soluble Chloride ASTM C1411-09, pH ASTM D4972, and Resistivity ASTM G187-12a).

  The in-situ moisture and density of soil samples at depths are shown on the borings logs included in Appendix A. The results of the remaining laboratory tests are presented in Appendix B, *Laboratory Test Results.*

- *Percolation Testing* – Boring LB-3 was drilled to 10 feet bgs and converted to a temporary percolation test well upon completion of drilling and sampling. Boring LB-3 was located in the central portion of the site within the main proposed parking lot area. In-situ percolation testing was performed in general accordance with City of Salinas *Stormwater Development Standards (SWDS) for New Development and Significant Redevelopment Projects* (City of Salinas, 2010). Refer to the discussion of infiltration rate presented in Section 2.4 and the field percolation test data provided in Appendix C, *Percolation Test Results*.

- *Engineering Analysis* – Geotechnical analysis was performed on the collected data to develop conclusions and recommendations for design and construction of the planned improvements.

- *Report Preparation* - This geotechnical report presents our findings, conclusions, and recommendations.

It should be noted that the recommendations in this report are subject to the limitations presented in Section 6.0 of the report.

2.0 GEOTECHNICAL FINDINGS

2.1 <u>Geologic Setting</u>

The site is located in the Salinas River Valley approximately 9.3 miles inland from Monterey Bay within the Coast Ranges geomorphic province of Central California. The Coast Ranges province makes up the coastal region of central and northern California and is characterized by northwest trending mountain ranges and valleys oriented subparallel to the San Andreas fault. The San Andreas fault is a major tectonic transform plate boundary and right-lateral strike-slip fault system that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The Coast Ranges are generally between 2,000 and 6,000 feet above sea level, and are composed of thick Mesozoic and Cenozoic sedimentary rocks. The northern and southern portions of the Coast Ranges are separated by a depression that contains the San Francisco Bay. The Salinas River Valley is a fault bounded topographically low coastal valley setting bordered by the Sierra de Salinas and the Gabilan Ranges on the southwest and northwest sides, respectively. There are no active faults known to cross the project site and the site is not mapped within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The closest active fault to the site (6.7 miles away) is the Rinconada fault.

The site is located approximately 3.3 miles northeast of the Salinas River. The near-surface native soils at the site are Quaternary-age (Holocene) young alluvial valley and flood plain deposits (Dibblee and Minch, 2007) comprised of varying proportions of gravel, sand, silt, and clay likely deposited by the Salinas River and its tributaries. A map showing the mapped geologic units in the vicinity of the project site is presented as Figure 4, *Regional Geology Map*.

2.2 <u>Subsurface Soil Conditions</u>

Based on our subsurface explorations, the site is underlain by a relatively thin veneer of artificial fill materials overlying Quaternary-age young alluvial valley and flood plain deposits. The stratigraphy of the subsurface soils encountered in each soil boring is presented in the boring logs (Appendix A), and a general description of the earth materials as encountered are described below:

<u>Artificial Fill</u>: The existing near-surface artificial fill soils encountered in our exploratory borings are approximately 2 to 4 feet thick across the project site and consist primarily of brown to dark brown, moist, clay. Localized thicker

accumulations of fill materials should be anticipated during future earthwork construction. There are also several documented areas of previously removed underground storage tanks (USTs). The fill at former UST locations is anticipated to be on the order of 20 feet bgs. The existing artificial fill materials encountered at the site are likely associated with the existing improvements and initial development of the site. However, records documenting observation and testing during fill placement were not available for review. Abandoned piping and miscellaneous debris should also be anticipated to be encountered during future earthwork construction.

Quaternary Age Young Alluvial Valley and Flood Plain Deposits: The Quaternary age young alluvial valley and flood plain deposits encountered beneath the artificial fill materials in our exploratory borings and CPTs generally varies between the northern and southern portions of the project site. The alluvial soils as encountered in the northern portion of the site (boring LB-1) generally consist of medium brown to olive gray, moist to very moist, medium stiff to very stiff clay with interbedded silt and silty fine sand in the upper approximately 5 to 20 feet bgs. It should be noted that abundant black ash and remnants of burnt organic material was encountered within the clay in boring LB-1 between approximately 30 and 35 feet bgs. The alluvial soils as encountered in the southern portion of the site (boring LB-2) generally consist of interbedded medium brown, moist, stiff to very stiff silt, clayey silt, silty clay, clay in the upper approximately 20 feet bgs. Below approximately 20 feet bgs in the southern portion of the site, the alluvial soils generally consist of interbedded light yellow brown, slightly moist to moist, medium dense to very dense, silty sand, sand and gravelly sand.

The stratigraphy of the subsurface soils encountered in each soil boring is presented in the boring and CPT logs (Appendix A). The general subsurface conditions across the site are depicted on Figure 3, *Geologic Cross-Sections A-A' and B-B'*.

2.2.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our exploration, the near surface onsite soils consist predominantly of silty sand, clayey sand, sandy clay, sandy silt, to clayey silt. The laboratory test result of representative near-surface (upper 5 feet) samples from borings LB-1 and LB-2 indicate medium to high expansion potential when wetted (EI = 78 and EI = 101). Accordingly, we recommend that the near-surface onsite soils be assumed to have high expansion potential.

Variance in expansion potential of onsite soil is anticipated; therefore, additional testing is recommended upon completion of rough grading to confirm the expansion potential result presented in this report. Standard engineering and earthwork construction practices, such as proper foundation design and controlled moisture conditioning or mixing with non-expansive soils will reduce the impacts associated with expansive soils.

2.2.2 Soil Corrosivity

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2011). The 2016 California Building Code (CBC), provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes, is 500 ppm per California Test 532. Concentrations of chloride ions above the stated concentration or other characteristics such as soil resistivity or redox potential may warrant special corrosion protection measures.

For screening purposes, representative near-surface (upper 5 feet) bulk samples from borings LB-1 and LB-2 indicate were tested from borings LB-1 and LB-2 to provide a preliminary evaluation of corrosivity. The test results indicates soluble sulfate concentrations of 114 to 143 ppm, chloride contents of 85 to 147 ppm, pH values of 6.94 to 8.15, and minimum resistivity values of 928 to 1090 ohm-cm.

The results of the resistivity test indicate that the underlying soil is severely corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil samples, concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318-11. The samples tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix B of this report.

2.2.3 Soil Compressibility

Seven (7() samples of the onsite soils recovered from the borings were subjected to consolidation testing to evaluate the compressibility of these materials under loads representative of anticipated structural bearing stresses. Although not precisely known, the maximum dead plus live column load is estimated to be about 400 kips. The results of in-situ resistance testing in our explorations and laboratory consolidation testing indicate that the onsite soils generally have high compressibility below the bearing grade of the planned foundations. The results of testing are presented in Appendix B.

2.2.4 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory Direct Shear testing. The results of testing are included in Appendix B.

2.3 Groundwater

Groundwater was encountered at a depth of about 19 feet during our subsurface exploration. Groundwater was encountered at a depth of 12 feet by others (Gecon Consultants, Inc., 2010). We recommend a design groundwater level of 10 feet bgs, which is the depth assumed in our analyses.

2.4 Infiltration Characteristics

In-situ percolation testing was performed to evaluate the infiltration characteristics of the site soil in the in the central in the central portion of the site within the main proposed parking lot area. In-situ percolation testing was performed in general accordance with City of Salinas *Stormwater Development Standards (SWDS) for New Development and Significant Redevelopment Projects* (City of Salinas, 2010).

Boring LB-3 was converted to a temporary percolation test well upon completion of drilling and sampling (Figure 2, *Exploration Location Map*). A 2-inch-diameter, perforated PVC pipe was placed in the borehole and the annulus was filled with clean sand (#3 Monterey Sand) from approximately 4 to 10 feet bgs. After the conclusion of the percolation test, the PVC pipe was removed and the test hole was backfilled with excess soil cuttings.

The test was performed using a falling-head method which records the drop of water levels inside the well over the testing period. The measured infiltration rate for the percolation test was calculated by dividing the rate of discharge (i.e., volume of water discharged from the well during the test) by the infiltration surface area, or flow area. Taking into consideration of the drop in water level during the test, the flow area was determined based on the average water height within the test well at the end of the test period. Detailed results of the field testing data and measured infiltration rate for the test well are presented in Appendix C. The test results are summarized below:

Table 1 – Measured (Unfactored) Infiltration Rate

Boring-Percolation Test Well Designation	Approximate Depth of Test Zone Below Ground Surface (feet bgs)	Measured Infiltration Rate (inches per hour)
LB-3	5 to 10	0.01

The test results indicate very low infiltration rates at the tested location and depth due to the fine grained silt and clay in this zone that generally do not provide adequate infiltration potential. The measured infiltration rate at test well location LB-3 (Figure 2) does not meet the minimum acceptable infiltration rate for stormwater infiltration practices (0.5 inches per hour) (City of Salinas, 2010).

3.0 GEOLOGIC/SEISMIC HAZARDS

Geologic and seismic hazards include surface fault rupture, seismic shaking, liquefaction, seismically-induced settlement, lateral spreading, seismically-induced landslides, flooding, seismically-induced flooding, seiches and tsunamis. The following sections discuss these hazards and their potential impact at the project site.

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). The closest active faults to the site are the Rinconada Fault Zone, Zayante-Vergeles Fault Zone and the San Andreas fault, located approximately 6.4 miles, 15.8 miles, and 20.9 miles from the site, respectively. The San Andreas fault is the largest active fault in California. Major regional faults with surface expression in proximity to the site are shown on Figure 5, *Regional Fault and Historic Seismicity Map*).

3.2 Strong Ground Shaking

The site is located within a seismically active region, as is Southern California in general. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. For the purpose of this report, the ground motion at the site due to earthquake shaking will be characterized by the code-based Peak Ground Acceleration (PGA_M) and the design response spectrum.

The code-based Peak Ground Acceleration (PGA_M) for the site was calculated at 0.603g using the United States Geological Survey (USGS) web-based U.S. Seismic Design Maps application (USGS, 2013a). The PGA_M corresponds to a modal earthquake with a probability of exceedance of 2 percent in 50 years (i.e., 2475-year return period). The modal earthquake is a Magnitude 6.6 earthquake with a distance of approximately 18.6 kilometers (11.6 miles) from the site (USGS, 2013b). The seismicity data are also included in Appendix D1.

The code-based site response spectra parameters for the design earthquake are as follows:

Table 2a –Seismic Design Parameters for Police Headquarters

Categorization/Coefficients	Code-Based
Site Longitude (decimal degrees) West	-121.64497
Site Latitude (decimal degrees) North	36.67415
Site Class	E
Risk Category	IV
Mapped Peak Ground Acceleration adjusted for Site Class Effects PGA_M	0.543g
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.591g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.590g
Seismic Design Category (S_1<0.75g)	D
Short Period Site Coefficient at 0.2s Period, F_a	0.9
Long Period Site Coefficient at 1s Period, F_v	2.4
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.432g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.417g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.955g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.945g

Table 3b –Seismic Design Parameters for Support Building

Categorization/Coefficients	Code-Based
Site Longitude (decimal degrees) West	-121.64607
Site Latitude (decimal degrees) North	36.67366
Site Class	E
Risk Category	II
Mapped Peak Ground Acceleration adjusted for Site Class Effects PGA_M	0.542g
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.603g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.589g
Seismic Design Category (S_1<0.75g)	D
Short Period Site Coefficient at 0.2s Period, F_a	0.9
Long Period Site Coefficient at 1s Period, F_v	2.4
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.442g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.415g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.962g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.943g

Seismic response spectra parameters were computed per Chapter 11 of ASCE 7-10 using the Seismic Design Map Tool, Version 3.1.0, last updated on June 23, 2014 by the United States Geological Survey (USGS).

No site-specific ground motion analysis is required because structures at the site will be assigned to Seismic Design Category D based on S_1 is less than 0.75g.

3.3 Historical Seismicity

Although Southern California has been seismically active during the past 200 years, written accounts of only the strongest shocks survive the early part of this period. Early descriptions of earthquakes are rarely specific enough to allow an association with any particular fault zone. It is also not possible to precisely locate epicenters of earthquakes that have occurred prior to the twentieth century.

A search of historical earthquakes was performed using the computer program EQ Search (Blake, 2015) for the time period between 1800 and 2015. Within that time frame, 485 earthquakes between magnitude 4.00 and 9.0 were found within a 62-mile (100-kilometer) radius of the site. Of these earthquakes, the closest were a series of earthquakes located approximately 1.4 miles (2.3 kilometers) from the site, and occurred in 1916 and 1931 (Appendix D1, *Seismicity Data*). Although not precisely located, the epicenter for each of these earthquake events is located to the southwest of the project site and registered magnitude 4.0 Mw and induced estimated peak ground accelerations (PGA) of 0.145g at the project site. The largest PGA at the site is estimated to have been roughly 0.284g from the magnitude 7.0 Mw earthquake that occurred on October 18, 1800.

There are records of two earthquakes with a magnitude 7.0 or larger within the search performed, which were both magnitude 7.0 Mw earthquakes that occurred on October 18, 1800 and October 17, 1989. For a general view of recorded historical seismic activity see Figure 5, *Regional Fault and Historic Seismicity Map*.

Review of additional data available from the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) indicates that the highest recorded ground acceleration in the vicinity of the project site was 0.120g for a station located approximately 500 feet to the southeast of the project site at Salinas City Yard. The recorded ground acceleration was from the magnitude 7.0 Mw earthquake that occurred in Loma Prieta on October 17, 1989.

3.4 Liquefaction Potential

The term liquefaction is generally referenced to loss of strength and stiffness in soils due to build-up of pore water pressure when subject to cyclic or monotonic loading. Both sandy and clayey soils are susceptible to loss of strength and stiffness. Because of the difference in strength characteristic and methods for evaluating strength loss potential for granular and clayey soils, the term liquefaction is used for granular soils while cyclic softening is used for fine-grained soils (i.e. clays and plastic silts).

In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically induced lateral ground deformations.

As shown on Figure 6, *Liquefaction Susceptibility Map*, and based on information available through the Monterey County Resource Management Agency, the project site is located within an area that has been identified as highly susceptible to liquefaction. As a part of this geotechnical exploration report, we have evaluated the liquefaction potential at the site and estimate the corresponding seismically induced ground deformations using the computer program CLiq (v.1.7.6.49).

Based on our evaluation and analysis of the 10 CPTs, the potential for liquefaction at the site is low. Results are shown in Appendix E.

Only CPT-6 located in the southernmost portion of the site indicated any liquefaction potential. Considering that liquefaction requires relatively continuous susceptible layers over a broad area, the isolated result from CPT-6 is not deemed representative of the actual hazard. The overall potential for liquefaction at the site is deemed low.

3.5 Seismically-Induced Settlement

Strong ground motion during earthquakes tends to rearrange looser soils particles into a more compact arrangement, especially in granular soil deposits. The cumulative effects of soil particles rearrangement during earthquake ground shaking will result in settlement. In general, a poorly graded granular deposit is more susceptible to settlement than a fine-grained or well-graded soil.

Based on our analysis of the CPTs, seismically-induced settlement at the site is expected to be on the order of 1 inch or less. Larger settlement was calculated for CPT-6 (about 1½ inches). However, that result is deemed to be an inconsistent outlier.

3.6 Seismically-Induced Lateral Ground Displacements

Depending on the site topography, modes of seismically induced lateral ground displacement associated with soil liquefaction consist of, ground oscillation (ground slope less than 0.3 percent), lateral spread (0.3 to 5 percent ground slope), or flow failure (ground slope greater than 5 percent). Based on the relatively level topography and the low potential for liquefaction, the potential for lateral ground displacement is low.

3.7 Seismically-Induced Landslides

The potential for seismically induced landsliding is considered low due to the location of the site and the lack of slopes at or nearby the site. Proposed slopes, if any, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

3.8 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site is located within "Zone X," which corresponds to a 0.2% annual chance flood hazard area or 500-year flood hazard zone (Figure 7, *Flood Hazard Zone Map)*.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site is located within a flood impact zone for the Nacimiento and San Antonio Dams as indicated on Figure 8, *Dam Inundation Map*. However, catastrophic failure of this dam is expected to be a very unlikely event in that dam safety regulations exist and are enforced by the Division of Safety of Dams, Army Corp of Engineers and Department of Water Resources. Inspectors may require dam owners to perform work, maintenance or implement controls if issues are found with the safety of the dam. Therefore, it is our opinion that the potential for seismically induced flooding to affect the site due to dam failure is low.

3.9 Seiches and Tsunamis

Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the project site and the lack of large enclosed water bodies nearby, seiche and tsunami risks are considered very low.

3.10 Subsidence

Subsidence is sinking of the Earth's surface in response to geologic or man-induced causes. Subsurface solution of limestone during cave formation may lead to a series of subsidence features at the ground surface, which, collectively, are termed karst topography. Since the site is not underlain by limestone, the potential for subsidence to affect the site due to this condition is not a consideration for the project.

4.0 FINDINGS AND CONCLUSIONS

No evidence of adverse geological or geotechnical hazards was noted at the site that will preclude the development of the project. Presented below is a summary of findings based upon the results of our geotechnical evaluation of the site:

- The site is not located in a designated Alquist-Priolo Earthquake Fault Zone. The nearest fault to the site is the Rinconada fault which is located more than 6 miles from the site. The site is expect to experience moderate to strong ground shaking resulting from an earthquake from one of the major regional faults.

- The site is located within an area shown as highly susceptible to liquefaction. However, based on our evaluation and analysis of the ten (10) CPTs, the potential for liquefaction at the site is low.

- The onsite undisturbed soils should exhibit adequate strength when subjected to the anticipated loading of the proposed improvements.

- Based on field observations and comparison of laboratory test results to California Building Code guidelines for expansive soils (CBC, 2016), the near surface onsite soils exhibit moderate to high potential for expansion when subjected to an increase in moisture.

- Concrete in contact with the near surface onsite soil is expected to have low exposure to water-soluble sulfates and low exposure to chloride in the soil. The onsite soil is considered severely corrosive to ferrous metal.

- The subsurface soils are anticipated to be readily excavated using conventional earthmoving equipment in good working condition.

5.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project in general accordance with 2016 CBC requirements. The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans prepared for the project.

5.1 Earthwork

Site grading recommendations are presented in the following paragraphs. The General Earthwork and Grading Recommendations are included in Appendix E. In case of conflict the following recommendations shall supersede those provided in Appendix E.

5.1.1 Site Preparation

Prior to construction, the project areas should be cleared of any existing improvements, vegetation, trash and debris, which should be properly disposed of offsite. Efforts should be made to remove or reroute any existing utility lines that interfere the proposed construction. Any resulting cavities should be properly backfilled and compacted.

5.1.2 Site Grading

A majority of the project area is covered with artificial fill (encountered between approximately 2 and 4 feet bgs at the boring locations). Localized thicker accumulations of undocumented fill materials should be anticipated during future earthwork construction (on the order of up to 20 feet bgs at former UST locations). To provide a uniform support and reduce the potential for differential settlement, all existing fill should be removed to expose suitable native soils and replaced as engineered fill to provide supports for the proposed buildings and other structural improvements. Removals should be performed such that a minimum of 3 feet of engineered fill is established below the bottom of all new foundations. Where feasible, overexcavation and recompaction should extend a minimum horizontal distance of 3 feet from the edges of the foundations (i.e., a 1:1 projection line down from the bottom edges of the foundations).

For new improvements outside the building footprint not structurally connected to the new buildings (i.e. pavement areas), the existing fill should be removed to a depth to allow placement of 1 foot of engineered fill under the planned improvements. The engineered fill should extend at least 1 foot beyond the edge of the new improvements.

Due to the expansive nature of near-surface soils, all concrete slabs on grade, including floor slabs, should be underlain by at least 2 feet of relatively non-expansive soils.

Leighton should verify the vertical and lateral removal limits during grading as local conditions may require additional removals (i.e., encountering additional undocumented fill or other deleterious materials).

Subgrade Preparation: After completion of the overexcavations and prior to fill placement, the moisture content of the soils should be determined, and the soils slowly and uniformly moistened (or dried) as necessary to bring the soils to a uniform moist condition and compacted to at least 90 percent relative compaction based on ASTM Test Method D 1557. Any soft or unsuitable earth materials encountered at the bottom of the excavations should be removed and replaced with compacted fill.

All concrete slabs on grade, including floor slabs, should be underlain by at least 2 feet of relatively non-expansive engineered fill.

Fill Placement: The onsite soils, less any deleterious material or organic matter, can be used in required fills. Oversized material greater than 6 inches in maximum dimension should not be placed in the fill. Inert construction debris (processed to be less than 4 inches in maximum dimension) may be used in engineered fill, except within the upper 2 feet below concrete slabs-on-grade, which should be constructed with relatively non-expansive fill. Any soil to be placed as fill, whether onsite soils or imported material, should be reviewed and possibly tested by Leighton.

All fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to at least 2 to 4 percentage points above optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

Any required import material should consist of non-corrosive and predominantly granular soils with an Expansion Index (EI) of 20 or less. The imported materials should contain sufficient fines (binder material) so as to result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being transported to the site.

Shrinkage and Bulking: The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience. We preliminarily estimate the onsite undocumented fill and alluvial materials requiring removal and recompaction will have a shrinkage factor of approximately 10 percent during grading.

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

5.2 Foundation Recommendations

We recommend that the proposed buildings be supported on a shallow spread footing foundation system established on undisturbed natural soils or engineered fill. Foundations may be designed to impose an average bearing pressure of 2,000 pounds per square foot (psf). A one-third increase in the bearing value for short duration loading, such as wind or seismic forces, may be used.

The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads.

Footings should have a minimum width of 12 inches for continuous footings and 18 inches for isolated footings. Footings should have a minimum embedment of 12 inches below the lowest adjacent grade.

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.25 can be used between the footings and the floor slab and the supporting soils.

The ultimate passive resistance of undisturbed natural soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot (pcf). The ultimate passive resistance of engineered fill can be assumed to be equal to the pressure developed by a fluid with a density of 300 pcf.

The friction resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

The estimated total settlement of the structures supported on spread footings as recommended above is less than 1½ inch. The differential settlement between adjacent columns is estimated to be less than ½ inch over a horizontal distance of 30 feet.

5.3 Conventional Slab-On-Grade Recommendations

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 5.1 (underlain by at least 2 feet of relatively non-expansive natural soils or engineered fill). From a geotechnical standpoint, we recommend slab-on-grade be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 24 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil-thick vapor barrier between the slab and subgrade. A coefficient of friction of 0.35 can be used between the floor slab and the vapor barrier.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential but not eliminate for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

5.3.1 <u>Moisture Vapor Retarder</u>

The following recommendations are for informational purposes since they are unrelated to the geotechnical performance of the foundation. Post-construction moisture migration should be expected below the foundation.

In general, interior floor slabs at or near the existing ground surface with moisture sensitive floor coverings are recommended to be underlain by a minimum 10-mil thick vapor retarder that has a permeance of less than 0.3 perms, as determined by ASTM E 96, and meets the applicable code requirements (ASTM E1745). The use of a capillary moisture break (crushed gravel layer) in conjunction with a vapor retarder is not considered to be necessary due to the lack of shallow groundwater conditions unless required by code. A sand layer below the synthetic sheeting will, however, serve to protect the sheeting from punctures if the underlying soils or gravel layer contain sharp, angular particles. Sand layer thickness above the barrier should be determined by the engineer/architect as they deem necessary. Sand layers should be installed where applicable in accordance with ACI Publication 302 Guide for Concrete Floor and Slab Construction.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

5.4 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the Standard Specifications for Public Works Construction, ("Greenbook"), Latest Edition. Utility trenches can be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Granular Bedding:** 1) ½-inch open grade aggregate or 2) a uniform sand material with a Sand Equivalent (SE) greater-than-or-equal-to 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), water densified in place, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), latest Edition.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D1557 laboratory maximum density.

5.5 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings a minimum of 2 percent for earthen surfaces for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by the use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering.

5.6 Corrosion Protection Measures

Two representative bulk soil samples were tested to evaluate corrosion potential to buried concrete (e.g., footings, retaining walls). The chemical analysis test results are summarized in Table 7 below.

Table 4 – Chemical Analysis Test Results

Test Parameter	Test Results Borings LB-1 and LB-2 at 0' to 5'	General Classification of Hazard
Water-Soluble Sulfate in Soil (ppm)	114 to 143	Negligible sulfate exposure to buried concrete
Water-Soluble Chloride in Soil (ppm)	85 to 147	Non-corrosive to embedded steel
pH	6.94 to 8.15	Neutral to alkaline
Minimum Resistivity (saturated, ohm-cm)	928 to 1090	Severely Corrosive to buried metals

[1]ASTM STP 1013 titled Effect of Soil Characteristics on Corrosion (February, 1989)

Based on the results of laboratory testing, reinforced concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates and chloride in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC 2016 requirements. However, concrete exposed to recycled water should be designed using Type V cement.

Based on our laboratory testing, the near-surface onsite soil is considered severely corrosive to ferrous metals. Typical corrosion protection in accordance with the Greenbook and the Plumbing Code should be provided.

5.7 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 9. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Table 5 – Conventional Retaining Wall Design Parameters

Equivalent Fluid Pressure (psf/ft)	
Condition	**Level Backfill**
Active	40
Seismic Increment (Additive to Active Pressure)	20
At-Rest	65
Passive	250
Coefficient of Friction	0.25

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Foundation design recommendations for retaining walls are presented in Section 5.2 of the report.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

The onsite soil is expansive and should not be used for retaining wall backfill.

5.8 Pavement Design

The paving thicknesses presented in the tables below are based on the currently available subsurface data. We assumed an average R-value of 5 for design base on the results of laboratory testing.

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction as recommended, the minimum recommended paving thicknesses are presented in the following table:

Table 6 – Preliminary Asphalt Concrete Pavement Structural Section

Area	Traffic Index (TI)	Asphalt Concrete (inches)	Base Course (inches)
Car Parking	4	3	8
Light Truck	5	3	10
Heavy Truck	6	4	12
Main Drives	7	4	16

The asphalt paving sections were determined using the Caltrans design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

Portland cement concrete paving sections as well as all other concrete slabs and walks supported on grade should be underlain by at least 2 feet of properly compacted fill consisting of relatively non-expansive natural soils or engineered fill. We have assumed that such a subgrade will have an R-value of at least 40, which will need to be verified during grading.

Portland cement concrete (PCC) paving sections were determined in accordance with procedures developed by the Portland Cement Association. PCC paving sections for a range of Traffic Indices are presented in the following table. We have assumed that the Portland Cement Concrete will have a compressive strength of at least 3,000 pounds per square inch.

Table 7 – PCC Pavement Structural Section

Area	Traffic Index (TI)	Asphalt Concrete (inches)	Base Course (inches)
Car Parking	4	6 ½	4
Light Truck	5	7	4
Heavy Truck	6	7 ½	4

The paving should be provided with expansion joints at regular intervals no more than 15 feet in each direction. Load transfer devices, such as dowels or keys, are recommended at joints in the paving to reduce possible offsets. The paving sections in the above table have been developed based on the strength of unreinforced concrete. Steel reinforcing may be added to the paving to reduce cracking and to prolong the life of the paving.

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). The existing asphalt paving may be used for base course if it is crushed and processed to meet the requirements of crushed miscellaneous base per the Green Book. The base course should be compacted to at least 95 percent relative compaction. The asphalt concrete should conform to the specifications outlined in Section 203-6 of the Green Book, and asphalt concrete construction methods should meet the requirements of Section 302-5 of the Green Book.

5.9 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations, limited laboratory testing and information available at the time the report is prepared. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- During overexcavation of unsuitable soil.
- Subgrade preparation;
- Compaction of all fill materials;

- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

6.0 LIMITATIONS

This report was based solely on data obtained from a limited number of geotechnical explorations, soil samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton has the opportunity to observe subsurface conditions during grading and construction, to confirm that our preliminary data are representative for the site. Leighton should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. The findings, conclusion, and recommendations included in this report are considered preliminary and are subject to verification. We do not make any warranty, either expressed or implied.

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Leighton

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

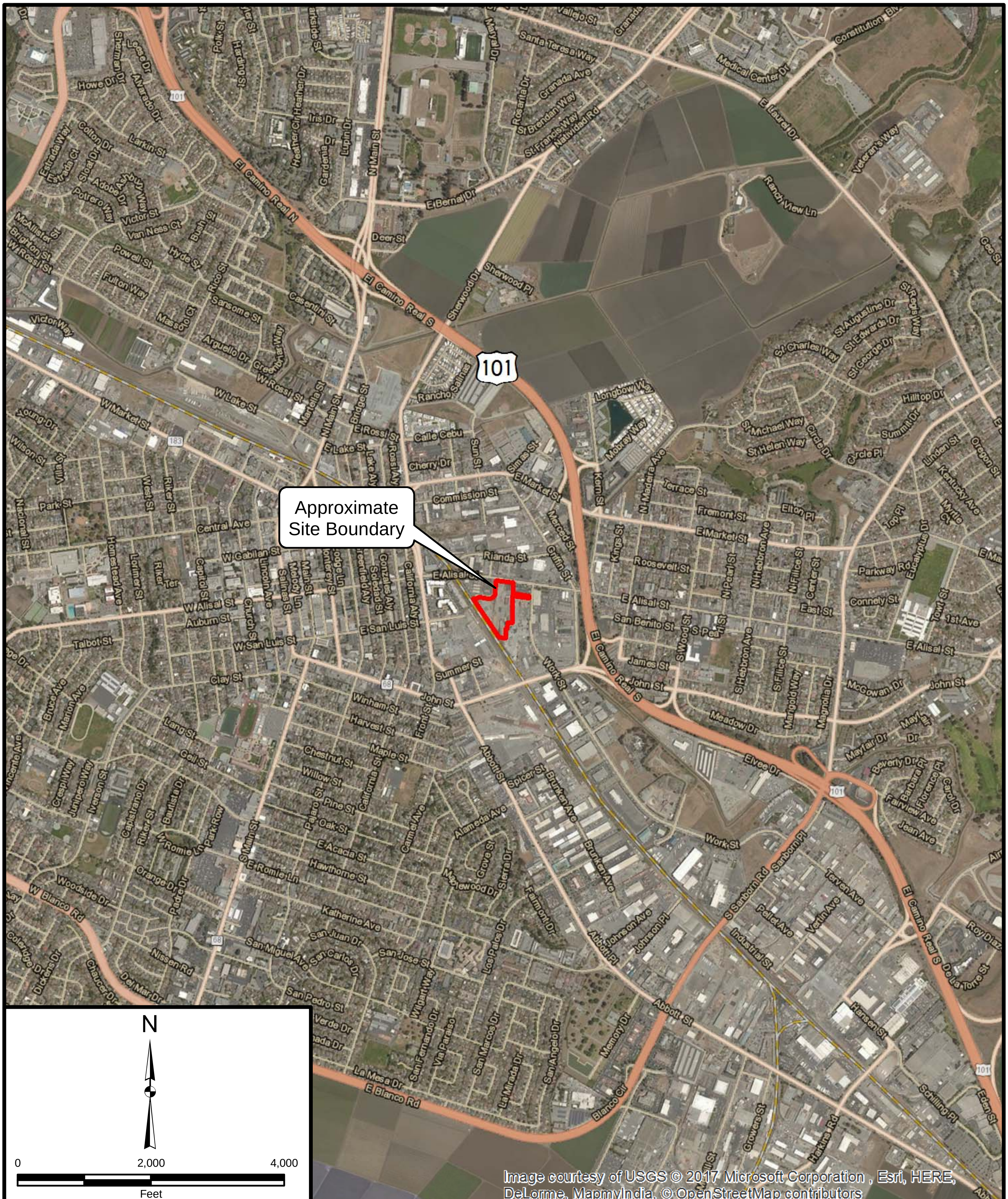
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733

e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 11693.001	Eng/Geol: CCK/JMP	SITE LOCATION MAP	Figure 1
Scale:1 " = 2,000 '	Date: August 2017	Proposed Police Headquarters 312 E. Alisal Street Salinas, California	Leighton
Base Map: ESRI ArcGIS Online 2017 Thematic Information: Leighton Author: Leighton Geomatics (btran)			

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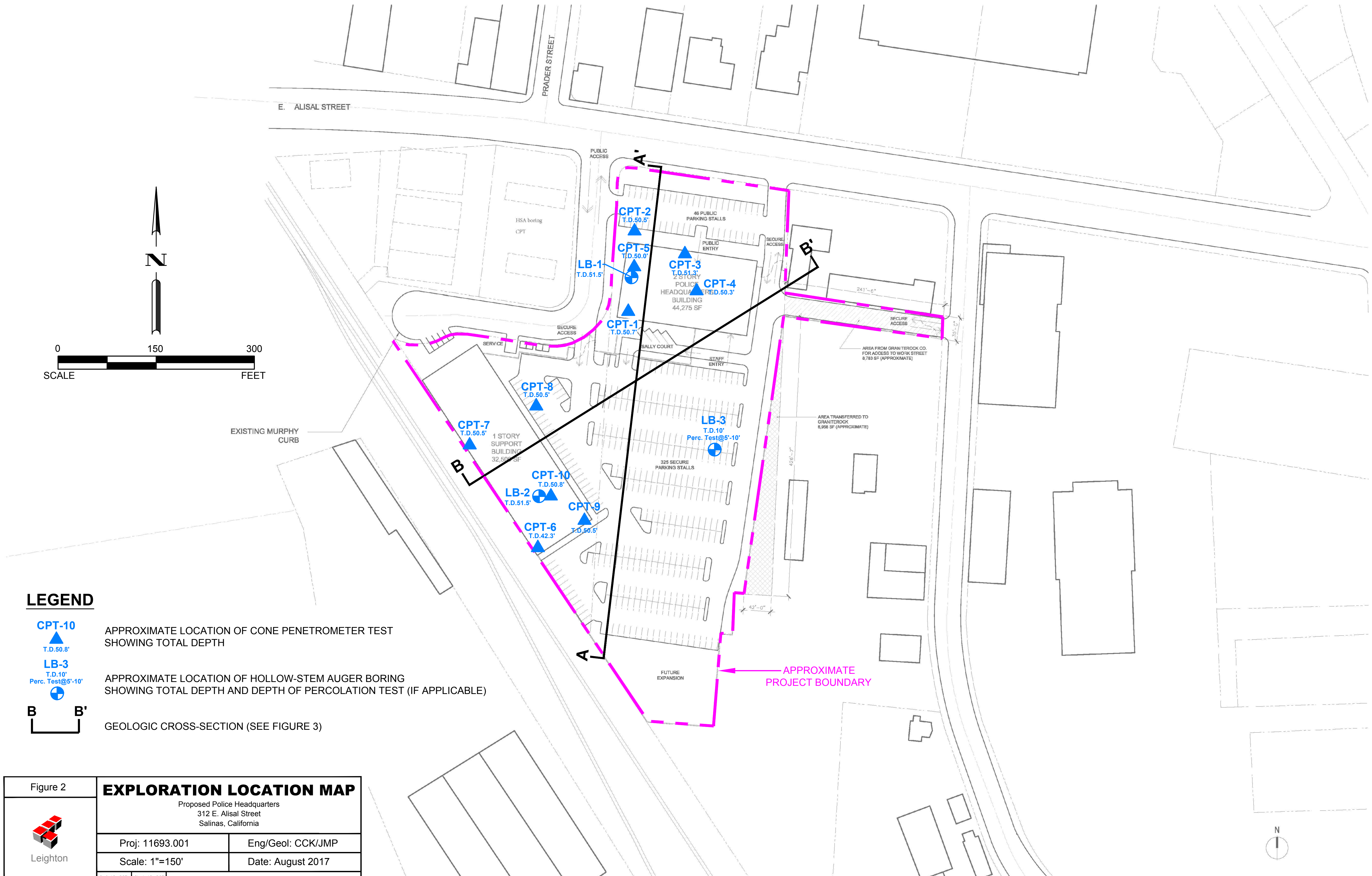


LEGEND

CPT-10 T.D.50.8' — APPROXIMATE LOCATION OF CONE PENETROMETER TEST SHOWING TOTAL DEPTH

LB-3 T.D.10' Perc. Test@5'-10' — APPROXIMATE LOCATION OF HOLLOW-STEM AUGER BORING SHOWING TOTAL DEPTH AND DEPTH OF PERCOLATION TEST (IF APPLICABLE)

B B' — GEOLOGIC CROSS-SECTION (SEE FIGURE 3)

Figure 2	EXPLORATION LOCATION MAP Proposed Police Headquarters 312 E. Alisal Street Salinas, California	
Leighton	Proj: 11693.001	Eng/Geol: CCK/JMP
	Scale: 1"=150'	Date: August 2017

LPA

Salinas Public Safety Facility

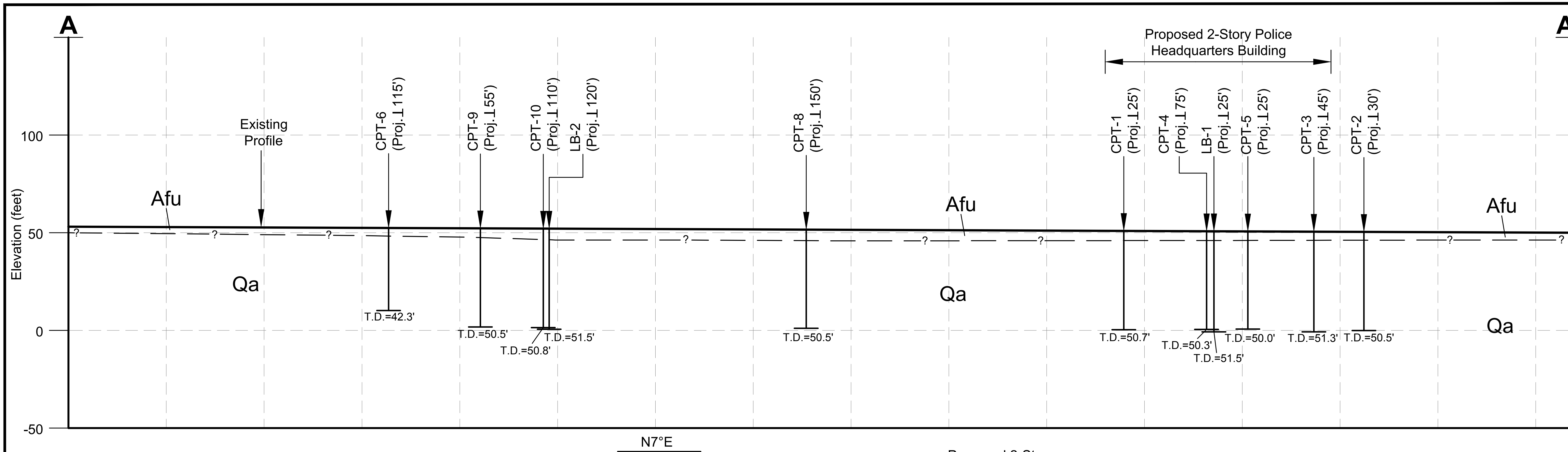
312 E ALISAL ST
SALINAS, CA 93901

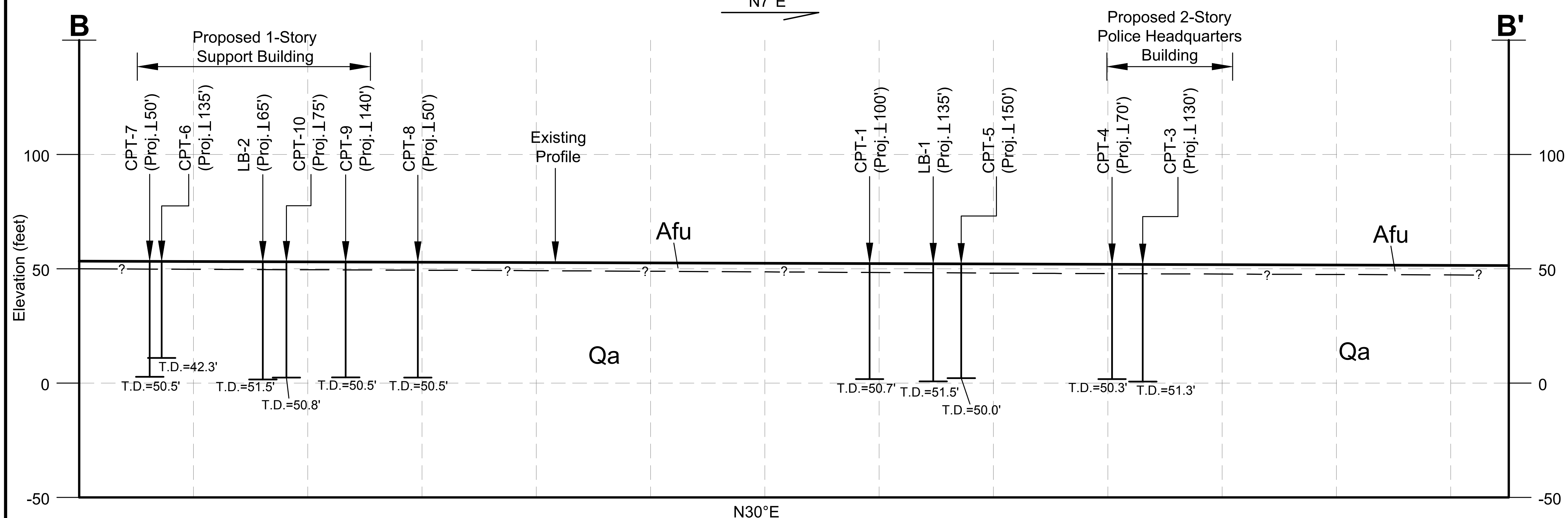
Developed for
Client Name

GRIFFIN SWINERTON

SITE PLAN

A1.01





Legend

Afu Artificial Fill, Undocumented

Qa Quaternary Young Alluviul Valley and Flood Plain Deposits

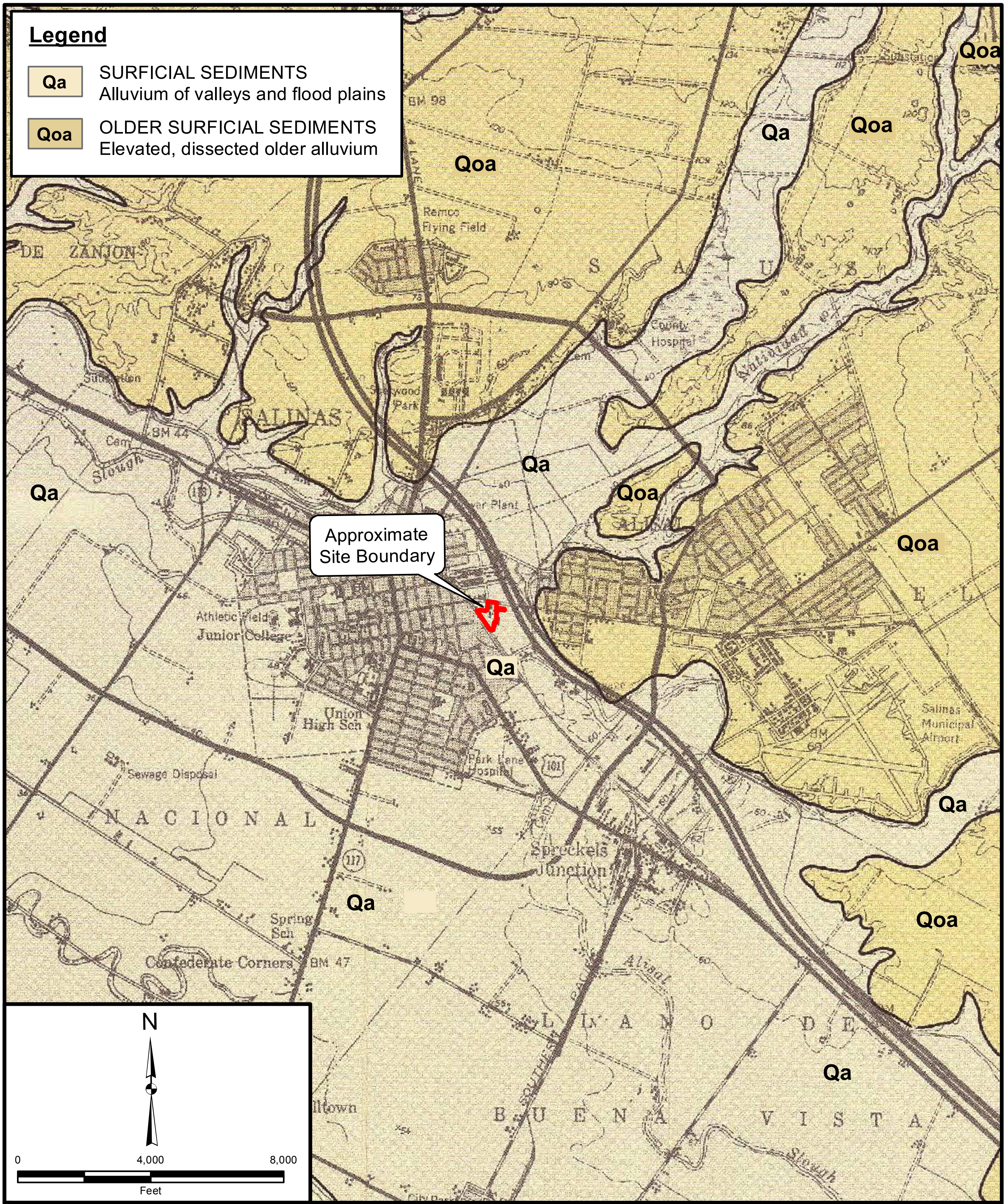
Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1"=50'	Date: August 2017
Reference:	
Author: MAM	

GEOLOGIC CROSS-SECTIONS A-A' & B-B'

Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 3

Leighton



Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: August 2017

Base Map: Geologic Map of the Monterey Peninsula and Vicinity, Monterey, Salinas, Point Sur, and Jamesburg 15-Minute Quadrangles, Monterey County, California by Thomas W. Dibblee, Jr. 1999.
Author: Leighton Geomatics (mmurphy)

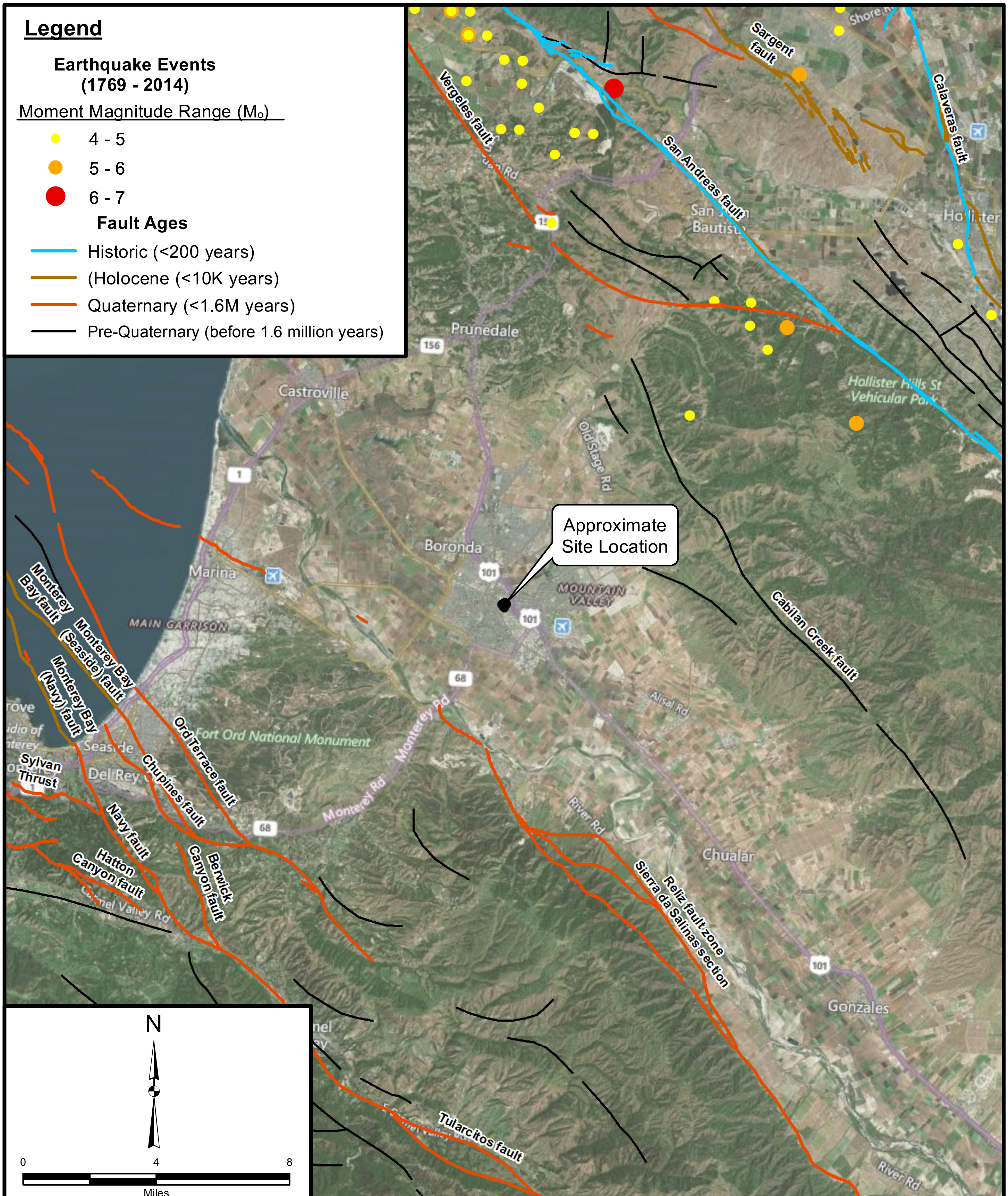
REGIONAL GEOLOGY MAP

Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 4

Leighton

Map Saved as P:\Drafting\11693\001\Maps\11693-001_F04_RGM_2017-08-10.mxd on 8/10/2017 3:34:28 PM



Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4 miles	Date: August 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (mmurphy)

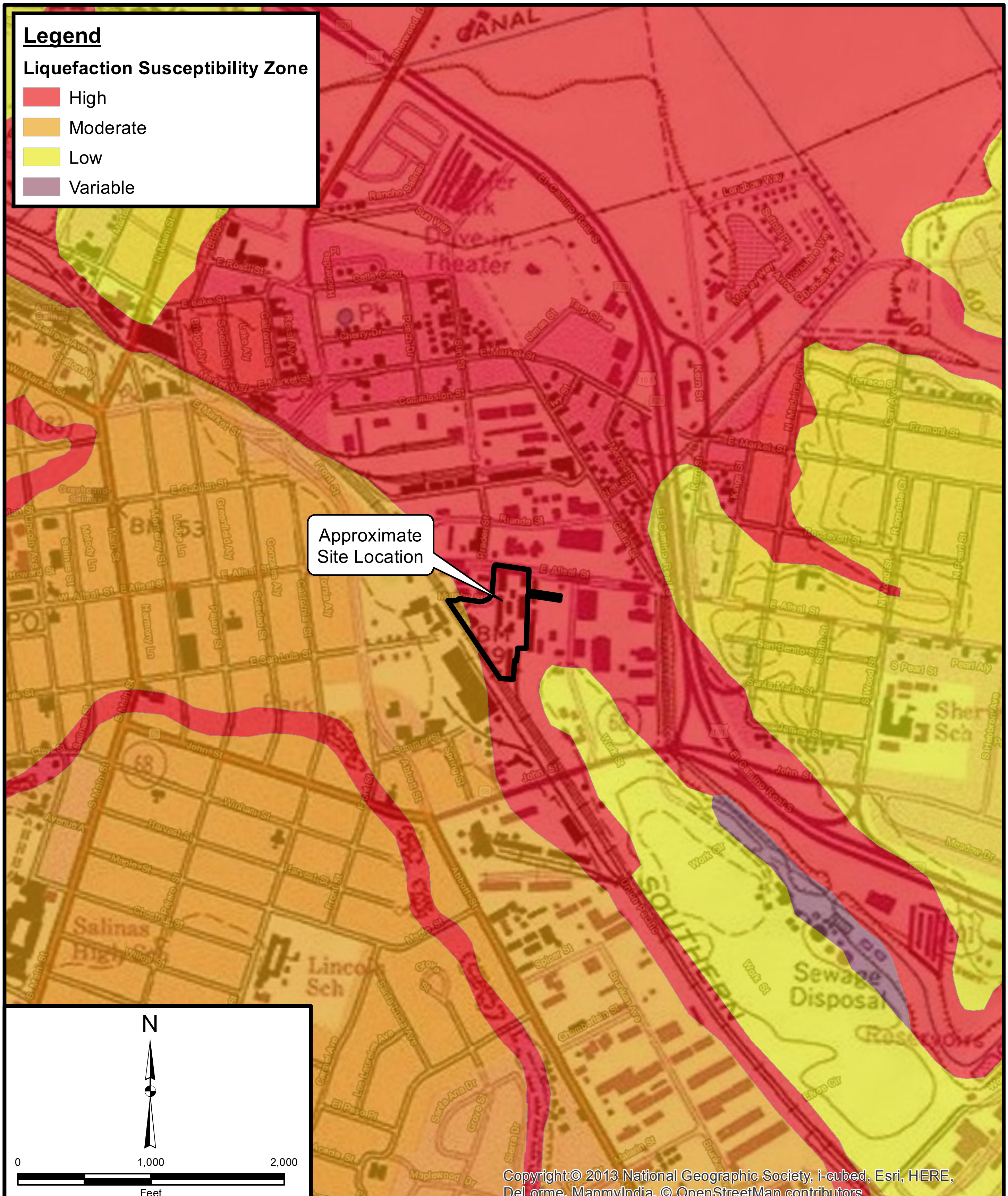
REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 5

Leighton

Map Saved as P:\Drafting\11693\001\Maps\11693-001_F05_RFSM_2017-08-11.mxd on 8/11/2017 11:18:13 AM



Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1 " = 1,000 '	Date: August 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Liquefaction Susceptibility Zones, Monterey County, California, 2015
Author: Leighton Geomatics (mmurphy)

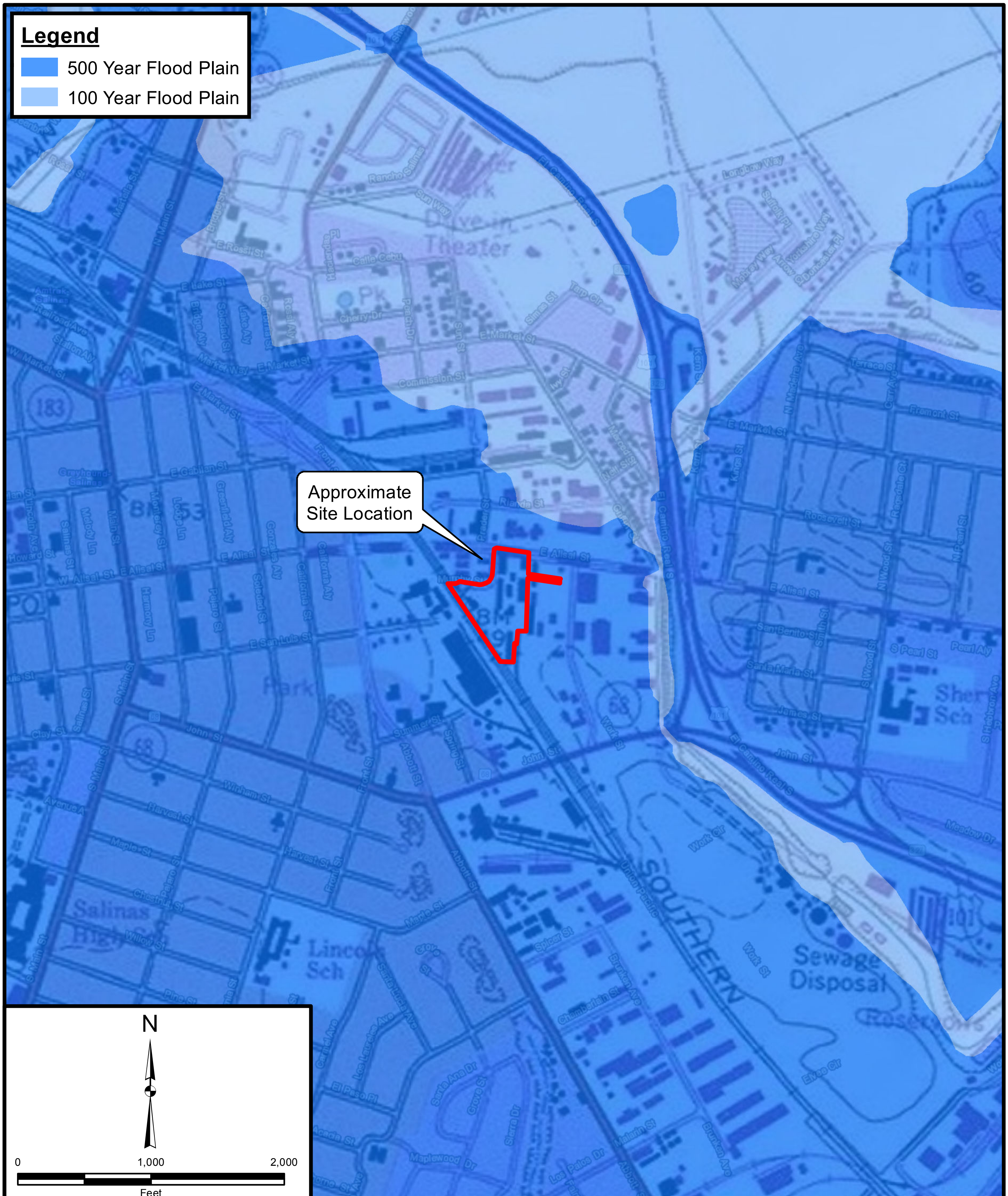
LIQUEFACTION SUSCEPTIBILITY MAP

Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 6

Leighton

Map Saved as P:\Drafting\11693\001\Maps\11693-001_F06_SHM_2017-08-07.mxd on 8/7/2017 5:56:01 PM



Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1 " = 1,000 '	Date: August 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (mmurphy)

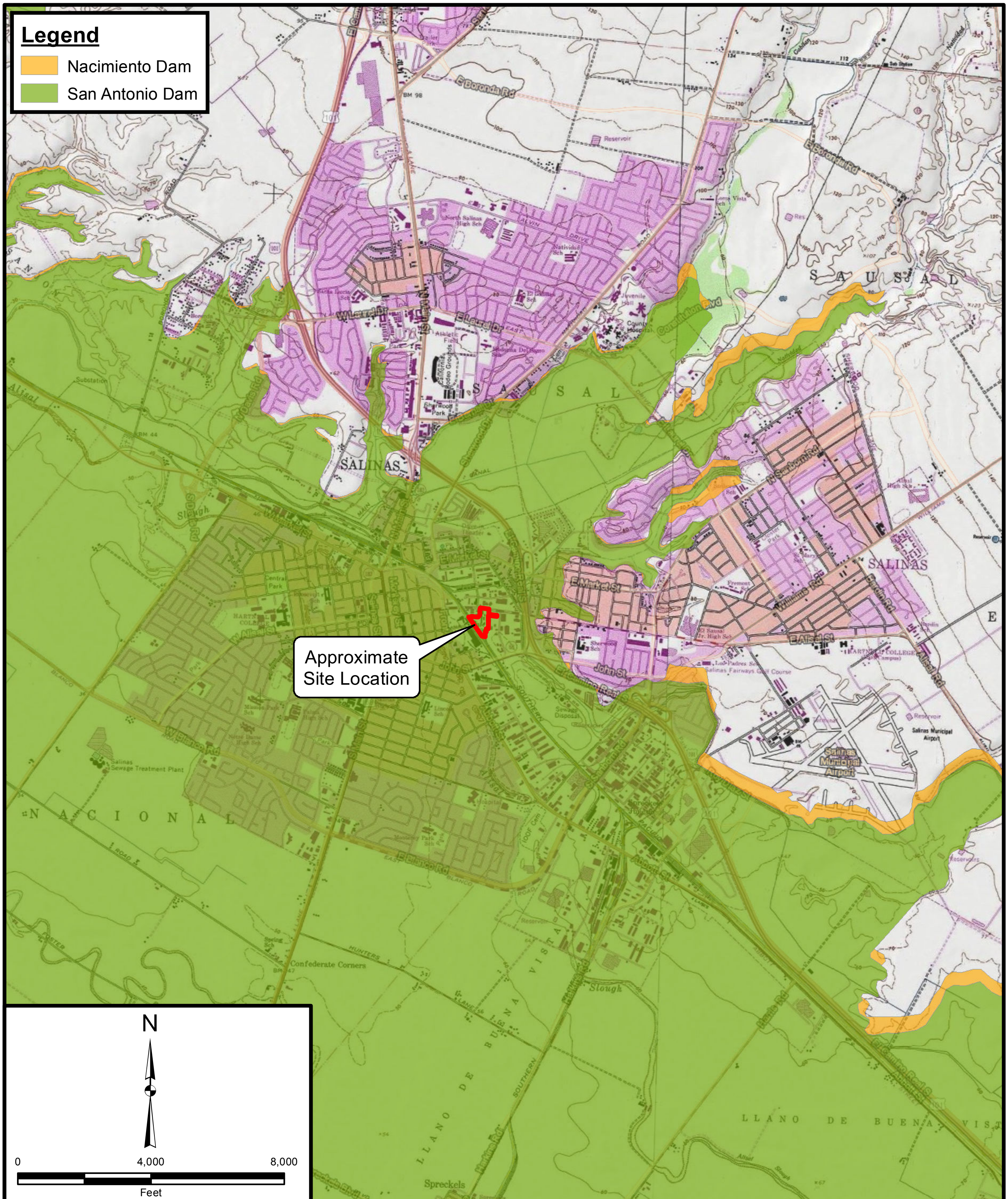
FLOOD HAZARD ZONE MAP

Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 7

Leighton

Map Saved as P:\Drafting\11693\001\Maps\11693-001_F07_FHM_2017-08-07.mxd on 8/7/2017 1:47:58 PM



Project: 11693.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: August 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (mmurphy)

DAM INUNDATION MAP

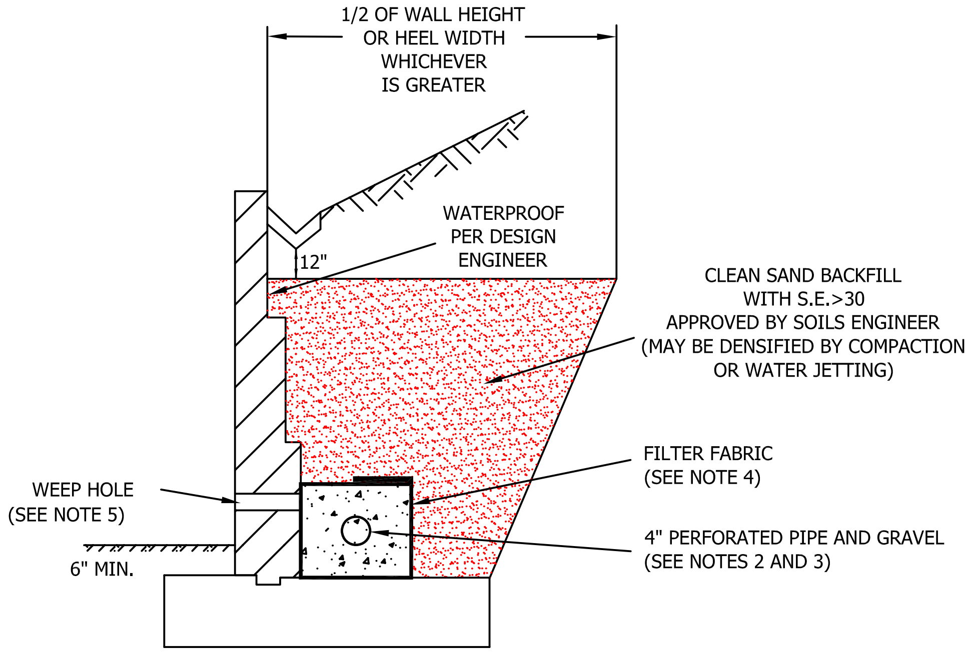
Proposed Police Headquarters
312 E. Alisal Street
Salinas, California

Figure 8

Leighton

Map Saved as P:\Drafting\11693\001\Maps\11693-001_F08_DIM_2017-08-07.mxd on 8/7/2017 1:52:01 PM

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50



NOTE: AS AN ALTERNATE TO CLEAN SAND BACKFILL, CLEAN GRAVEL MAY BE UTILIZED WITH APPROVED FILTER FABRIC. A SECOND ALTERNATE IS TO UTILIZE AN AGGREGATE BASE MATERIAL COMPACTED TO 90% RELATIVE COMPACTION. A SAMPLE OF THE PROPOSED BASE MUST BE APPROVED BY THE GEOTECHNICAL CONSULTANT PRIOR TO BACKFILL FOR SUITABILITY. COMPACTION SHOULD BE ACHIEVED WITHOUT DAMAGING THE WALL.

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

Leighton

Figure 9

APPENDIX A1

Boring Logs

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	11693.001	**Date Drilled**	7-13-17
Project	Salinas Police HQ	**Logged By**	JMP
Drilling Co.	Woodward Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	50'
Location		**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
50	0			BB-1				CL	@Surface: 3-inches Asphalt Concrete over 6-inches Aggregate Base @0.75': **<u>Artificial Fill, undocumented (Afu):</u>** CLAY, dark gray brown, soft, moist, some orange oxidation	MD,EI, CR,CN, DS,RV
									<u>Quaternary Alluvium (Qa):</u> @4': CLAY, medium brown, soft, very moist	
45	5			R1	5 6 8			ML	@5': SILT, medium brown stiff, very moist, micaceous	DS
				S1	4 2 4			CL	@7.5': (Limited Recovery) CLAY, dark brown, medium stiff, moist, thinly laminated	AL
40	10			R2	7 7 7					DS, CN
								SM	@11': Silty SAND, bluish gray to medium brown, loose, very moist, fine sand, micaceous	
35	15			S2	1 2 2		27		@15': Silty SAND, bluish gray to medium brown, loose, very moist to wet, fine sand, micaceous	
									@18.9': perched groundwater encountered	
30	20			R3	5 6 8			CL	@20': CLAY, olive gray with orange oxidation, stiff, very moist	AL, CN
25	25			S3	3 3 6		46		@25': CLAY, gray to olive gray with orange oxidation, stiff, very moist	
20	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-1

Project No.	11693.001	**Date Drilled**	7-13-17
Project	Salinas Police HQ	**Logged By**	JMP
Drilling Co.	Woodward Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	50'
Location		**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
20	30			R4	3 5 7			CL	@30': CLAY, gray, medium stiff, very moist, with abundant black ash and burnt organic material, organic odor	AL, CN
15	35			S4	3 5 5		66		@35': CLAY, gray with interlayered black ash/organics, stiff, very moist, organic odor	
10	40			R5	5 5 7				@40': CLAY, gray, medium stiff, very moist	AL, CN
5	45			S5	3 4 5		65		@45': CLAY, gray with interlayered black ash/organics, stiff, very moist, organic odor	
0	50			R6	5 6 11	70	53		@50': CLAY, gray, very stiff, very moist	
									Total Depth of Boring: 51.5 feet bgs Perched groundwater at 18.9 feet bgs Boring backfilled with cement/bentonite grout via tremmie pipe on 7/13/17	
-5	55									
-10	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-2

Project No.	11693.001	**Date Drilled**	7-13-17
Project	Salinas Police HQ	**Logged By**	JMP
Drilling Co.	Woodward Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	52'
Location		**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			BB-1					@Surface: Gravel **Artificial Fill, undocumented (Afu):**	MD,EI, CR,CN, DS,RV
								CL	@0': Gravelly SAND (base material), light gray brown, dry to slightly moist	
50								CL	@1': CLAY, dark brown, moist	
								ML	**Quaternary Alluvium (Qa):** @2': CLAY, medium brown, moist @3': SILT, medium brown, moist, micaceous	
	5			R1	9 12 15	92	23	CL	@5': Silty CLAY, medium olive brown, very stiff, moist, micaceous	
45				S1	5 6 7		18			
								SM	@8': Silty SAND, medium yellow brown, medium dense, slightly moist to moist	
	10			R2	8 9 9					CN
40										
	15			R3	6 4 5	93	25		@15': Loose	
35										
									@19': perched groundwater encountered	
	20			S2	2 4 5		40	ML-CL	@20': Clayey SILT to Silty CLAY with sand, blue gray, stiff, wet	
30										
	25			R4	10 20 22	105	4	SM/SP	@25': Silty SAND to SAND, light yellow brown, medium dense, slightly moist to moist, fine to medium sand	
25										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
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- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-2

Project No.	11693.001	**Date Drilled**	7-13-17
Project	Salinas Police HQ	**Logged By**	JMP
Drilling Co.	Woodward Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	52'
Location		**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S3	10 17 22		18	SW	@30': Gravelly Silty SAND, light yellow brown, dense, moist, fine to coarse sand, fine subangular gravels, interbedded clay layer at top of sample	
20										
	35			R5	23 35 50/5	103	4	SM	@35': Silty SAND, light yellow brown, very dense, slightly moist to moist, fine sand	
15										
	40			S4	14 25 35		8	SP	@40': SAND, light yellow brown, very dense, slightly moist, fine to medium sand, trace coarse sand	
10										
	45			R6	21 50/6	95	13	SW	@45': Gravelly SAND with Clay, light yellow brown, very dense, moist, medium to coarse sand, fine gravels, pockets of clay	
5										
	50			S5	17 28 30		6	SP	@50': SAND, light yellow brown, very dense, slightly moist, fine to medium sand	
0									Total Depth of Boring: 51.5 feet bgs Perched groundwater at 19 feet bgs Boring backfilled with cement/bentonite grout via tremmie pipe on 7/13/17	
	55									
-5										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-3

Project No.	11693.001	**Date Drilled**	7-13-17
Project	Salinas Police HQ	**Logged By**	JMP
Drilling Co.	Woodward Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	52'
Location		**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1				CL	@Surface: 3-inches of Asphalt Concrete over 7-inches of Aggregate Base @0.8': CLAY, dark brown, moist	
50									**Quaternary Alluvium (Qa):** @2': CLAY, medium brown, moist @3': SILT, medium brown, moist	
	5			S1	4 4 4		19	ML	@5': SILT, medium brown, medium stiff to stiff, moist	
45				S2	3 3 5		27	ML-CL	@7': same as above with interlayered Silty CLAY, dark brown, medium stiff to stiff, moist	
				S3	3 4 7		33	CL	@8.5': CLAY, medium brown, stiff, moist to very moist	
	10									
40									Total Depth of Boring: 10 feet bgs Boring installed with a temporary percolation well Screened (0.020-inches) from 5-10 feet bgs and annulus filled with #3 Monterey Sand from 4.5-10 feet bgs Pipe removed from well and boring backfilled with cuttings upon completion of percolation test 7/14/17	
	15									
35										
	20									
30										
	25									
25										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
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- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

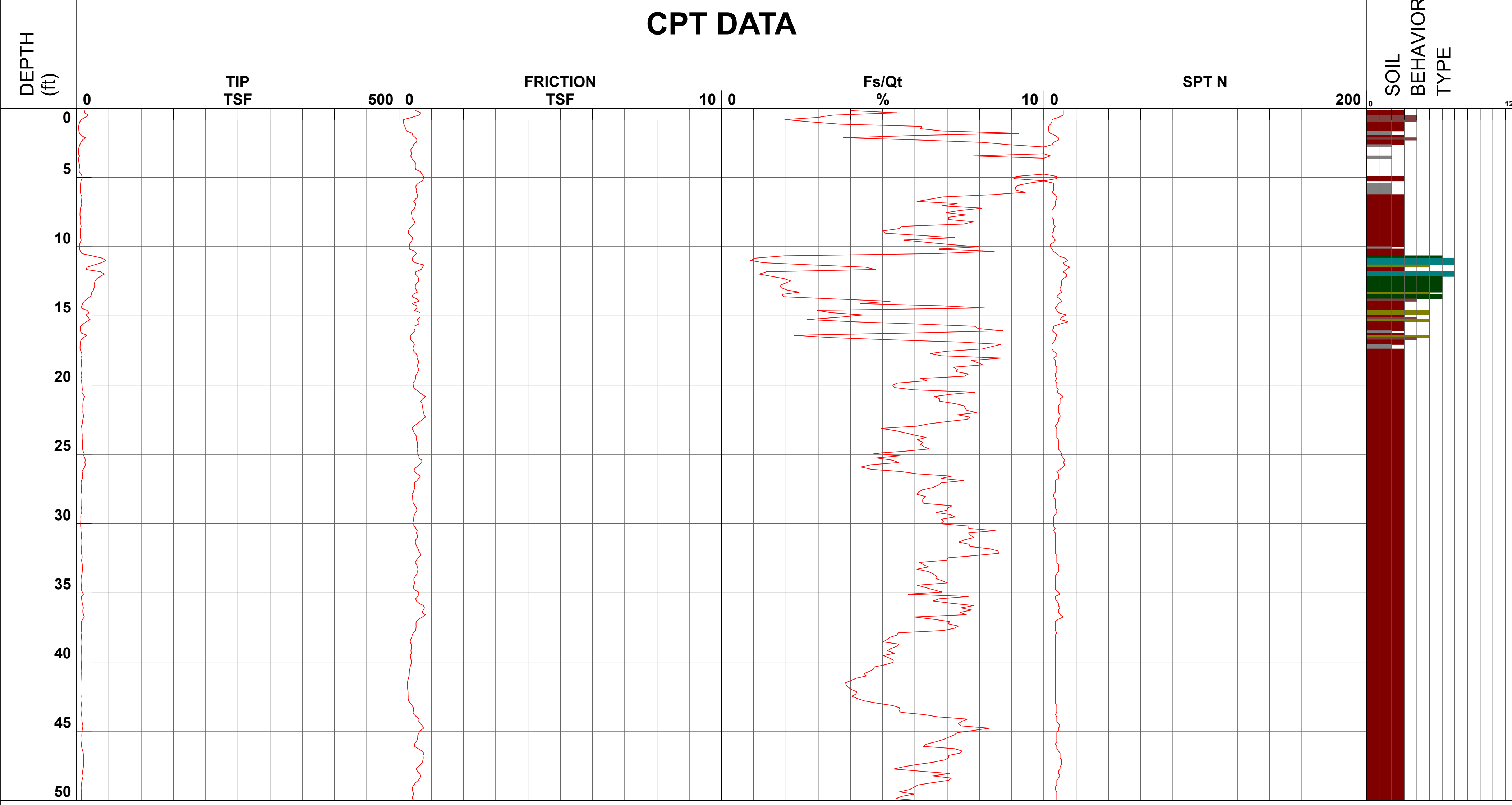
APPENDIX A2

CPT Plots and Data

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(107).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-01	Date and Time	7/14/2017 1:03:27 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

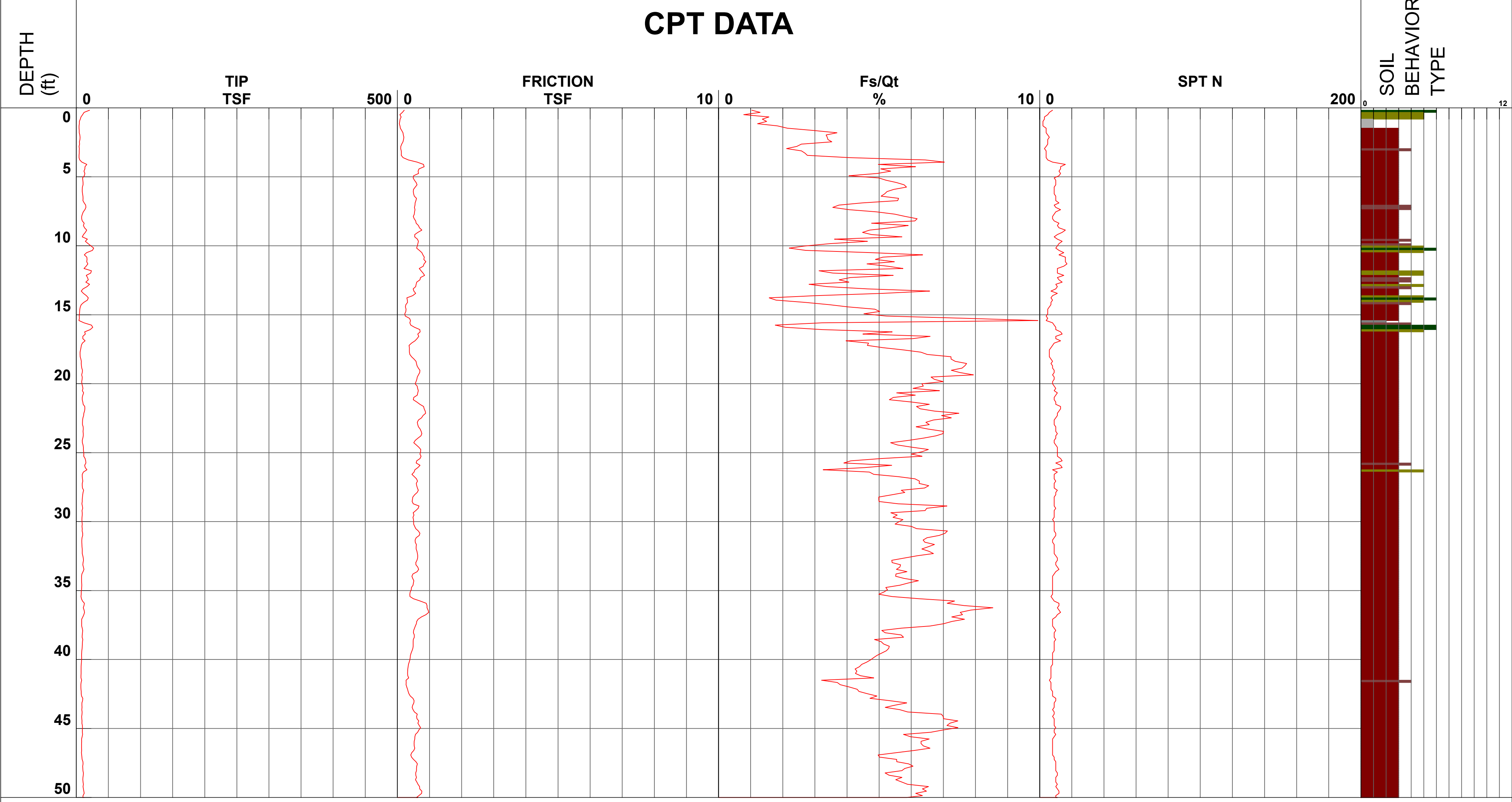
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(108).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-02	Date and Time	7/14/2017 1:38:33 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

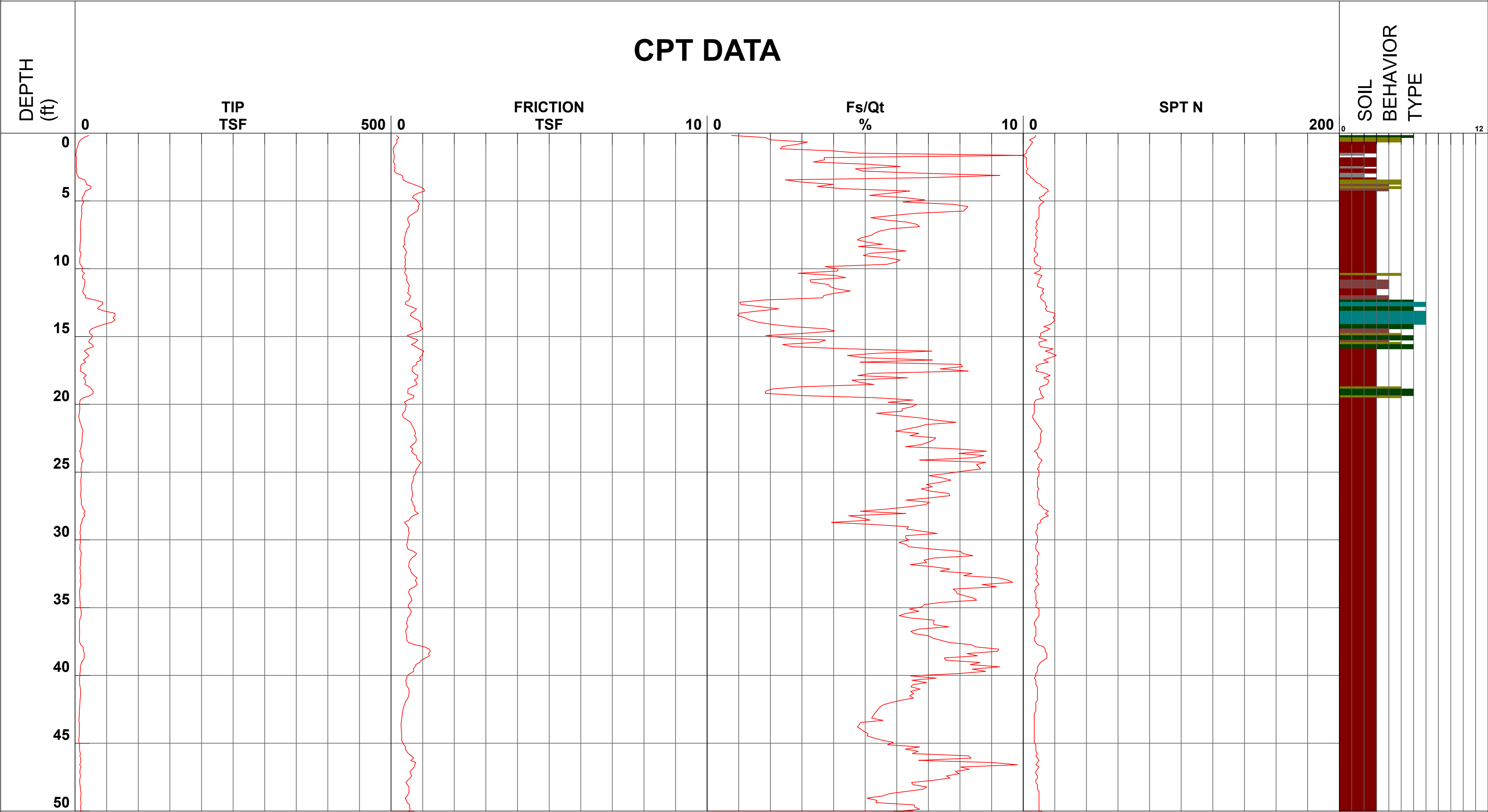
S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(105).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-03	Date and Time	7/14/2017 11:42:09 AM	Maximum Depth	51.34 ft
EST GW Depth During Test	15.00 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

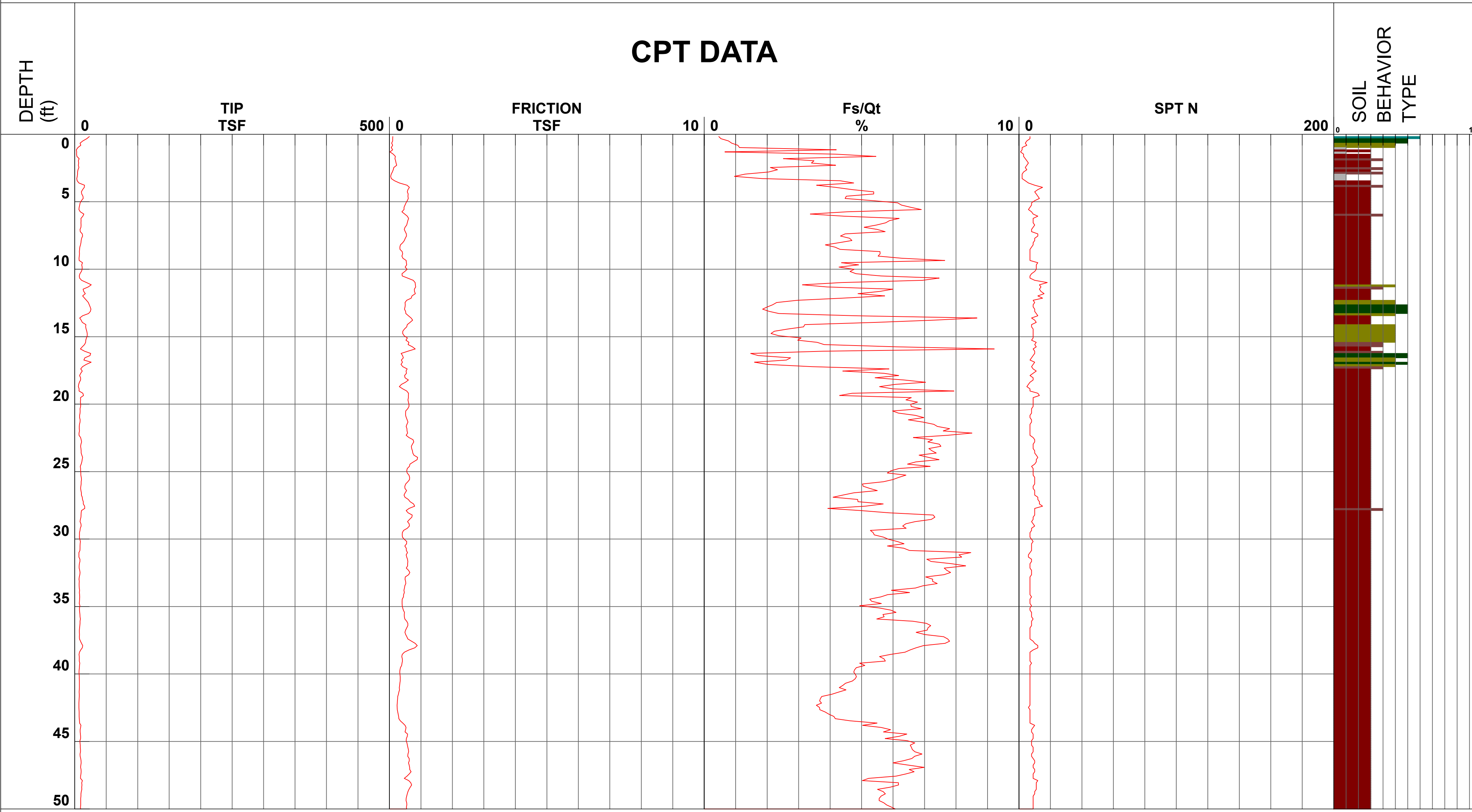
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(109).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-04	Date and Time	7/14/2017 2:17:46 PM	Maximum Depth	50.36 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

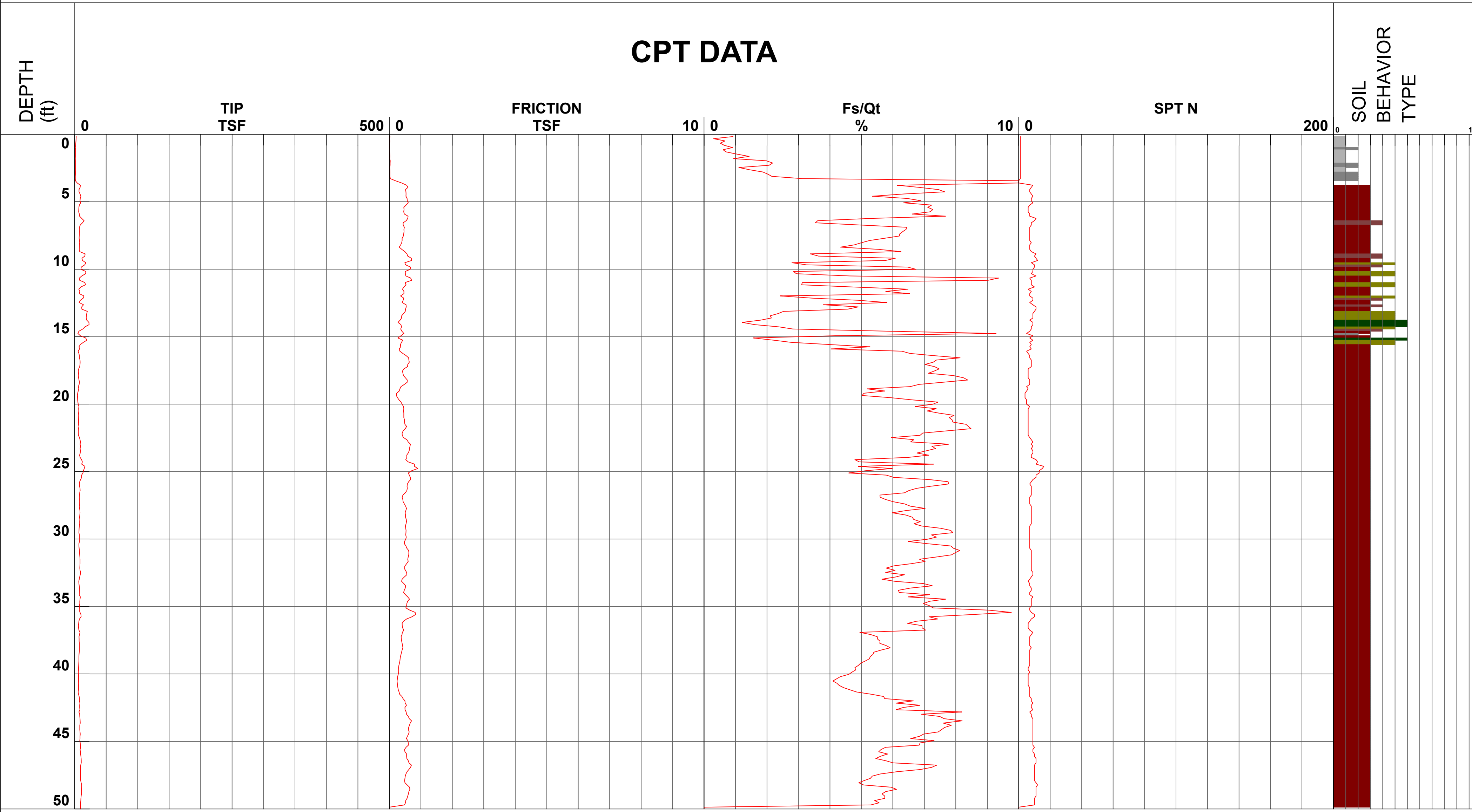
S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
Geo Testing Inc.

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(106).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-05	Date and Time	7/14/2017 12:21:38 PM	Maximum Depth	50.03 ft
EST GW Depth During Test		15.00 ft			

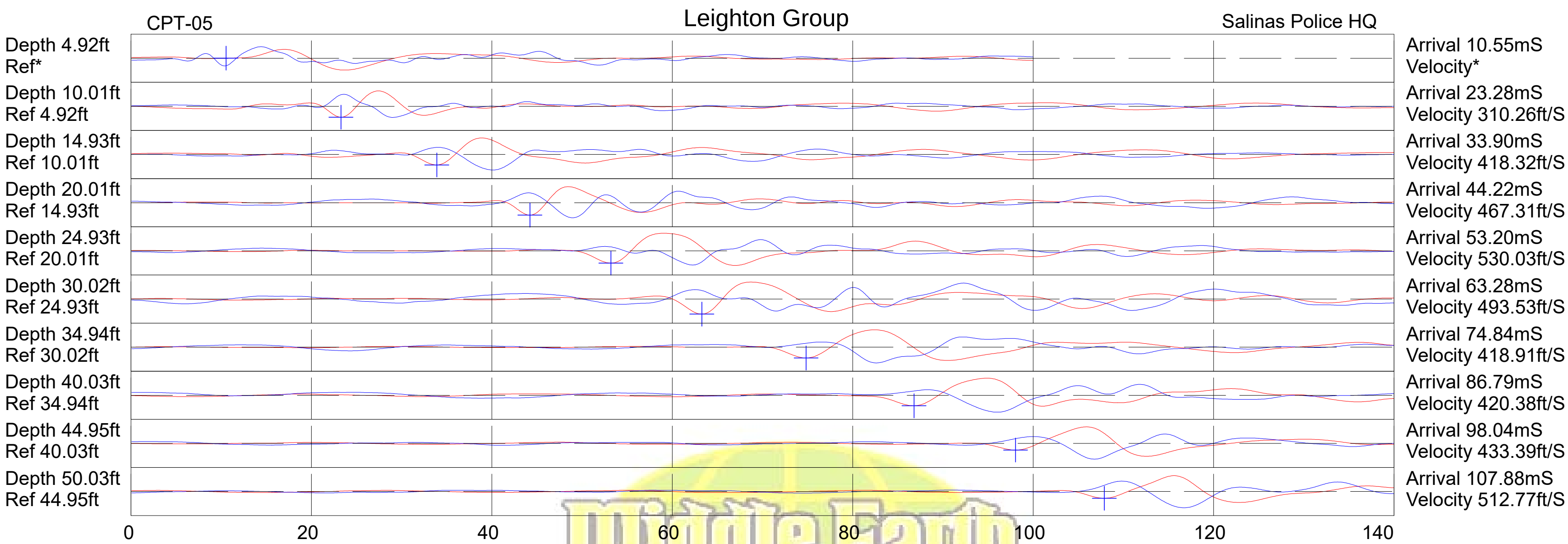
Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983



Hammer to Rod String Distance (ft): 5.83

* = Not Determined

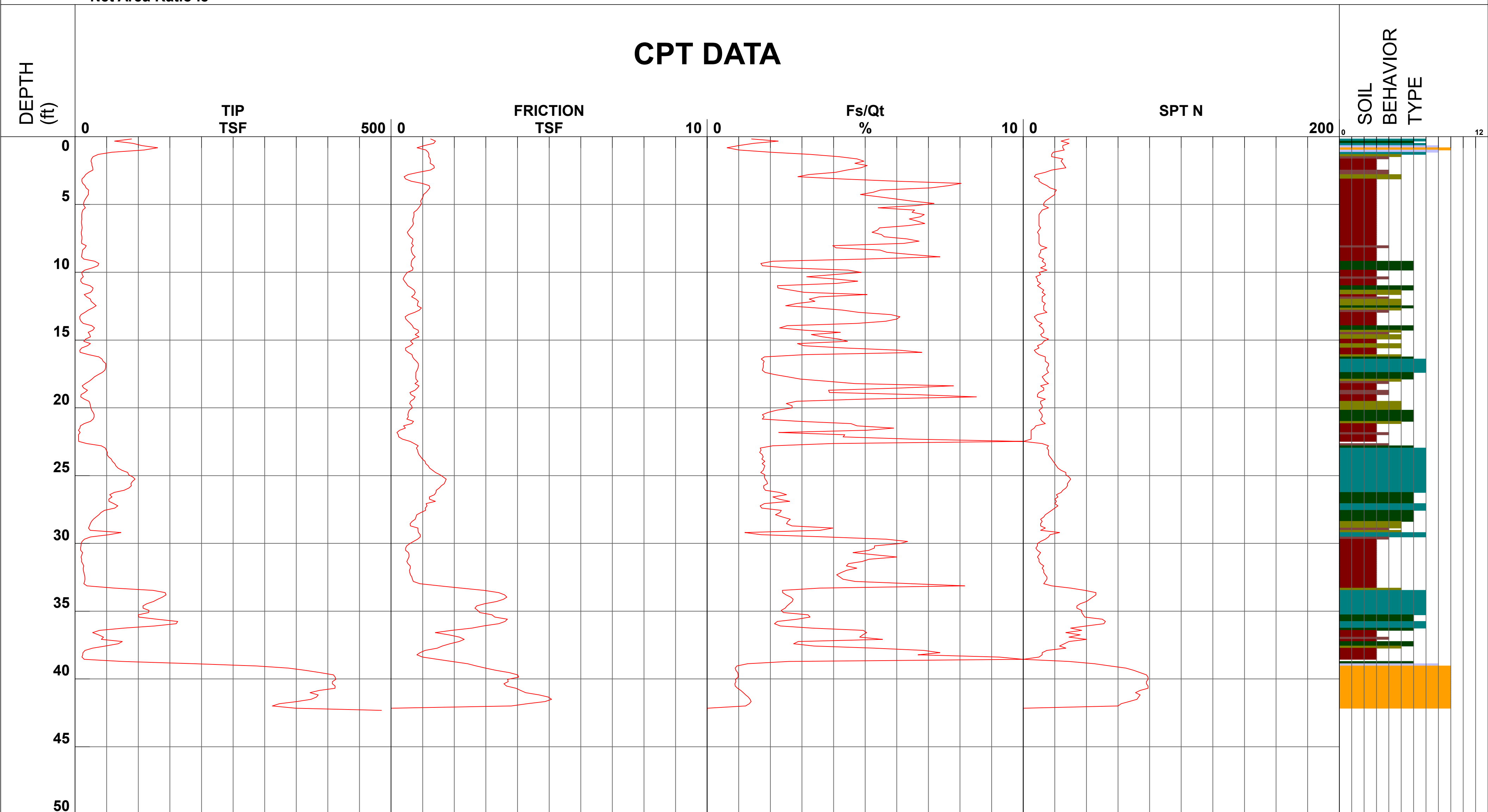
COMMENT:

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(102).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-06	Date and Time	7/14/2017 9:42:00 AM	Maximum Depth	42.32 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

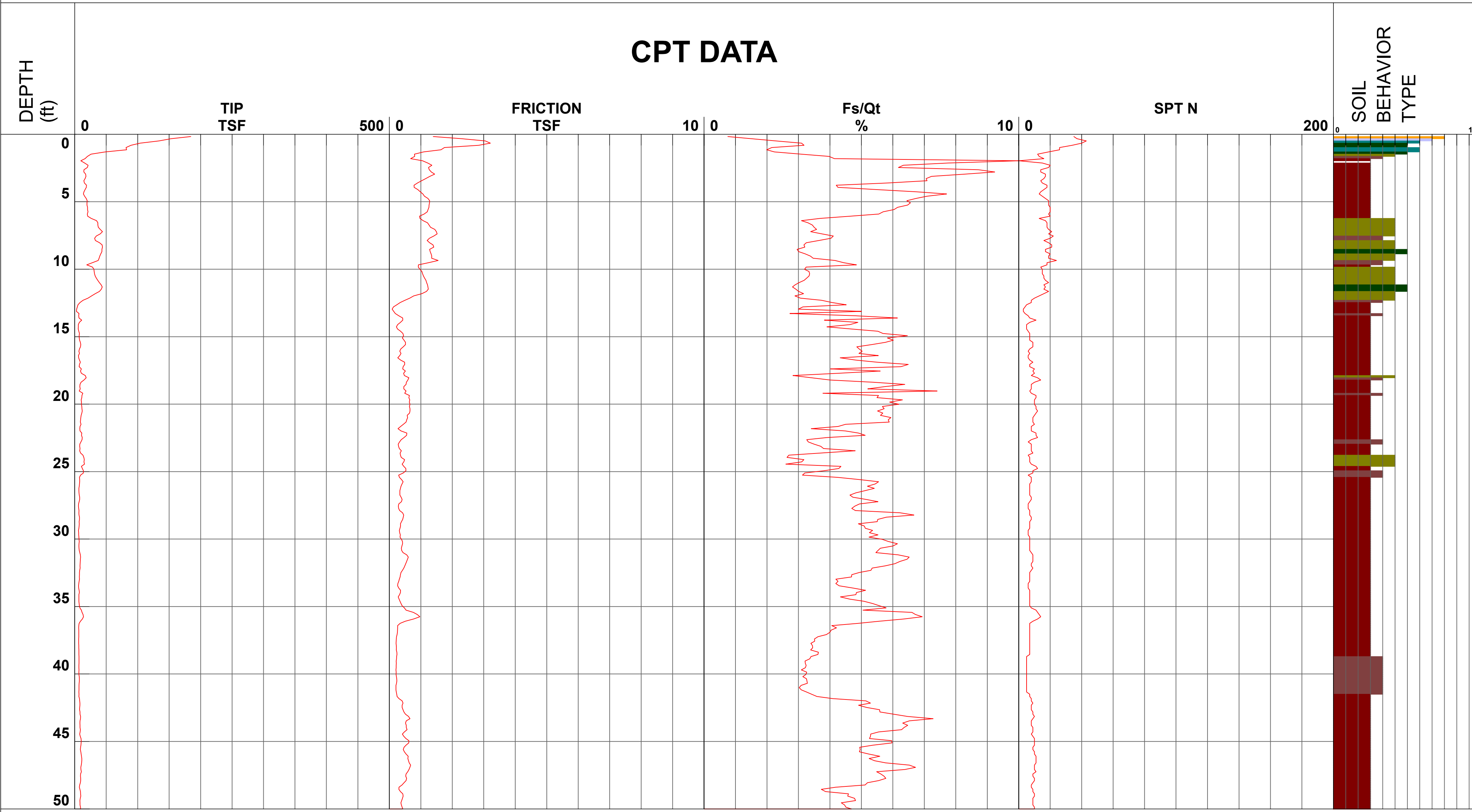
S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(101).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-07	Date and Time	7/14/2017 8:54:43 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

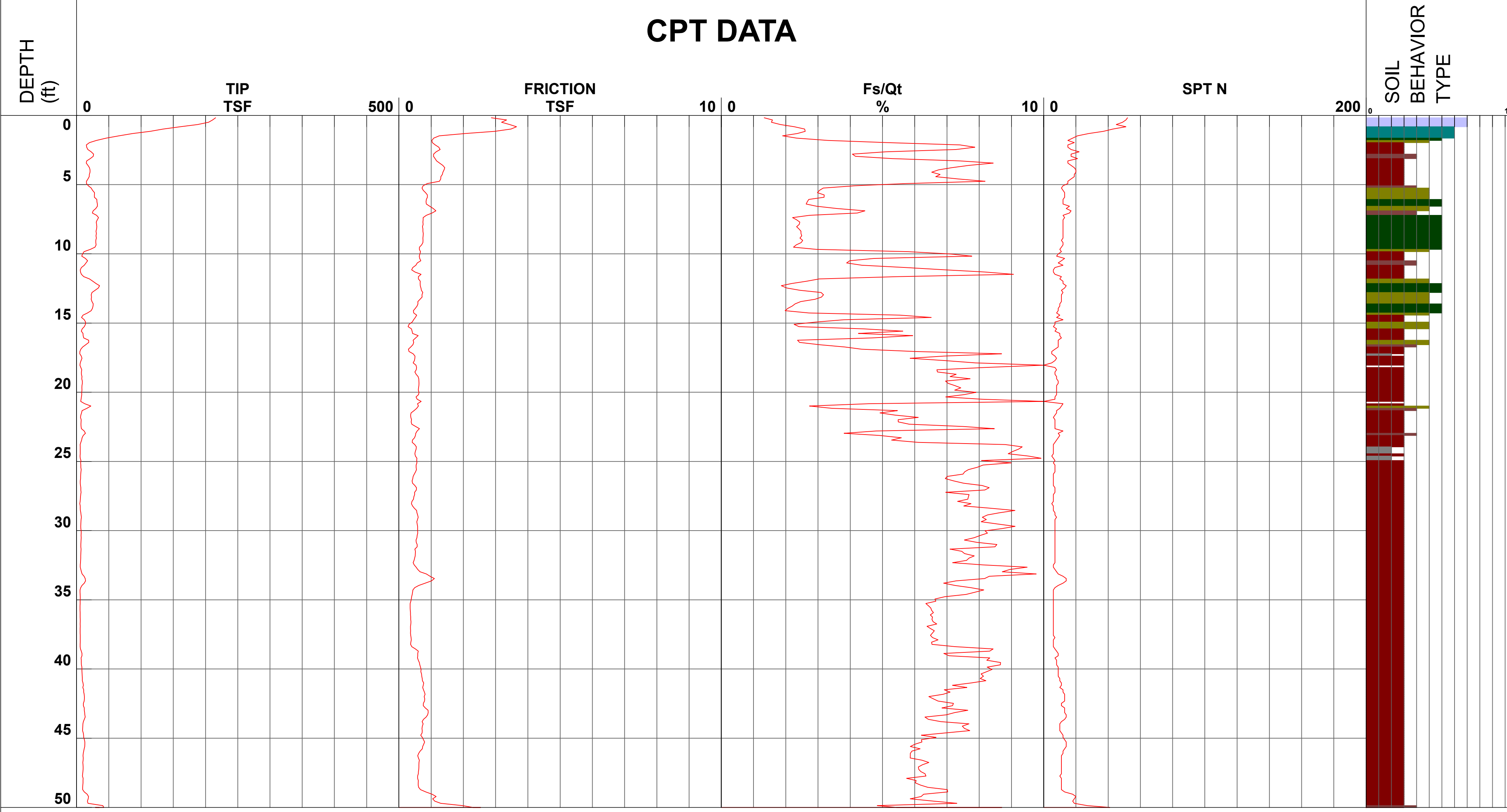
S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(104).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-08	Date and Time	7/14/2017 10:48:34 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

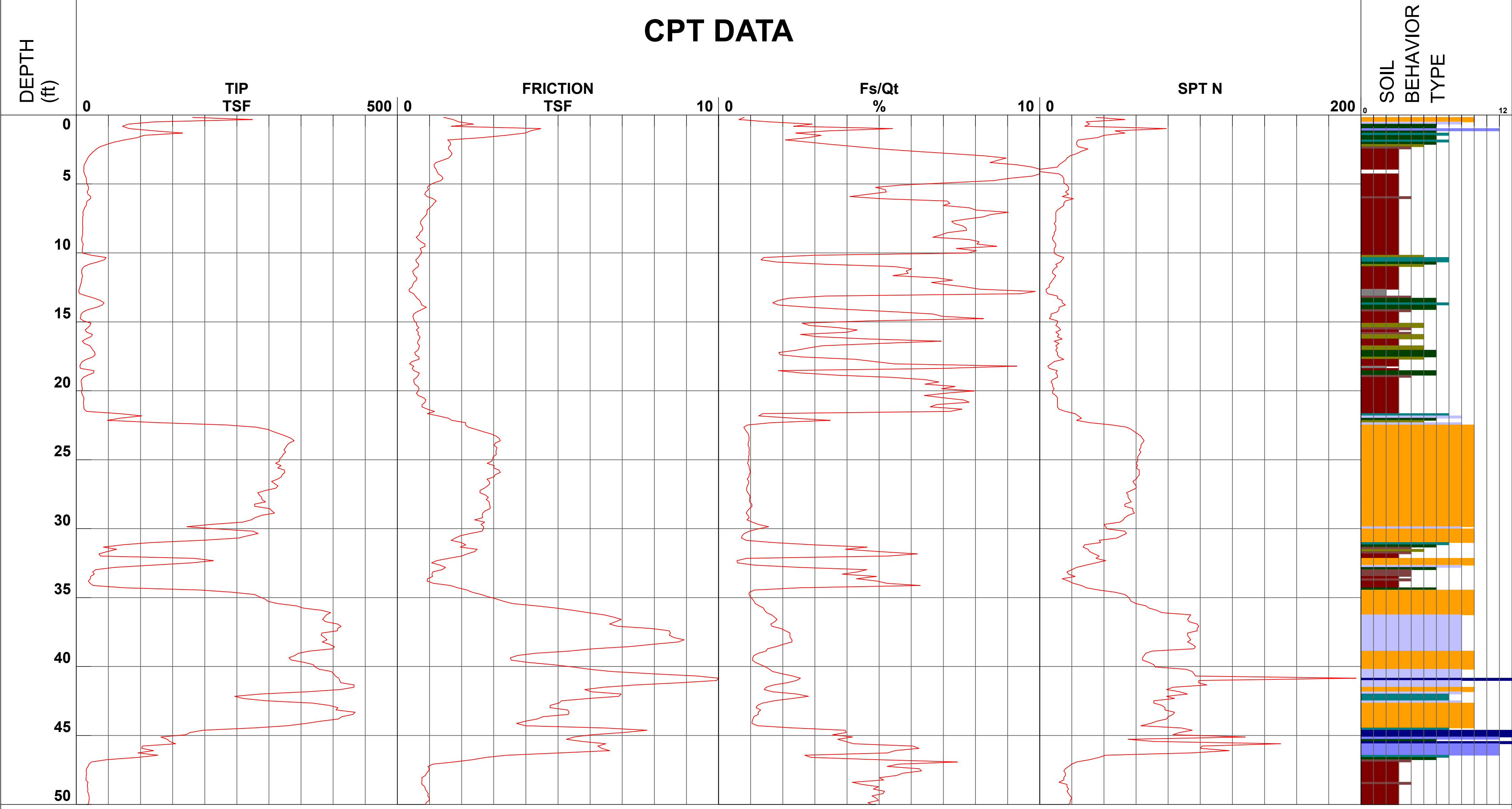
S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(103).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-09	Date and Time	7/14/2017 10:14:47 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

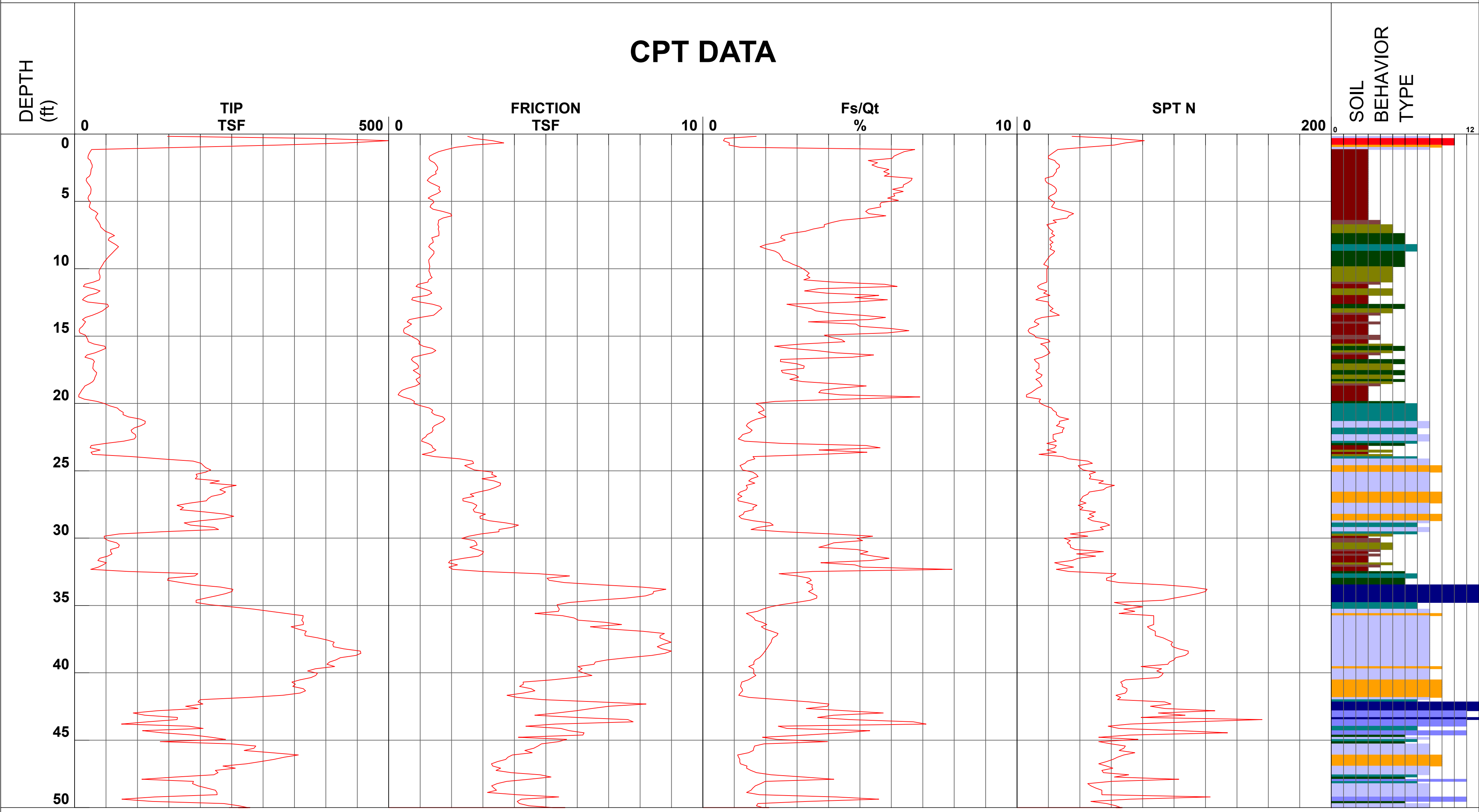
S*Soil behavior type and SPT based on data from UBC-1983

Leighton Group

Project	Salinas Police HQ	Operator	RB KK	Filename	SDF(100).cpt
Job Number	11693.001	Cone Number	DDG1333	GPS	
Hole Number	CPT-10	Date and Time	7/14/2017 8:09:25 AM	Maximum Depth	50.85 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

CPT DATA

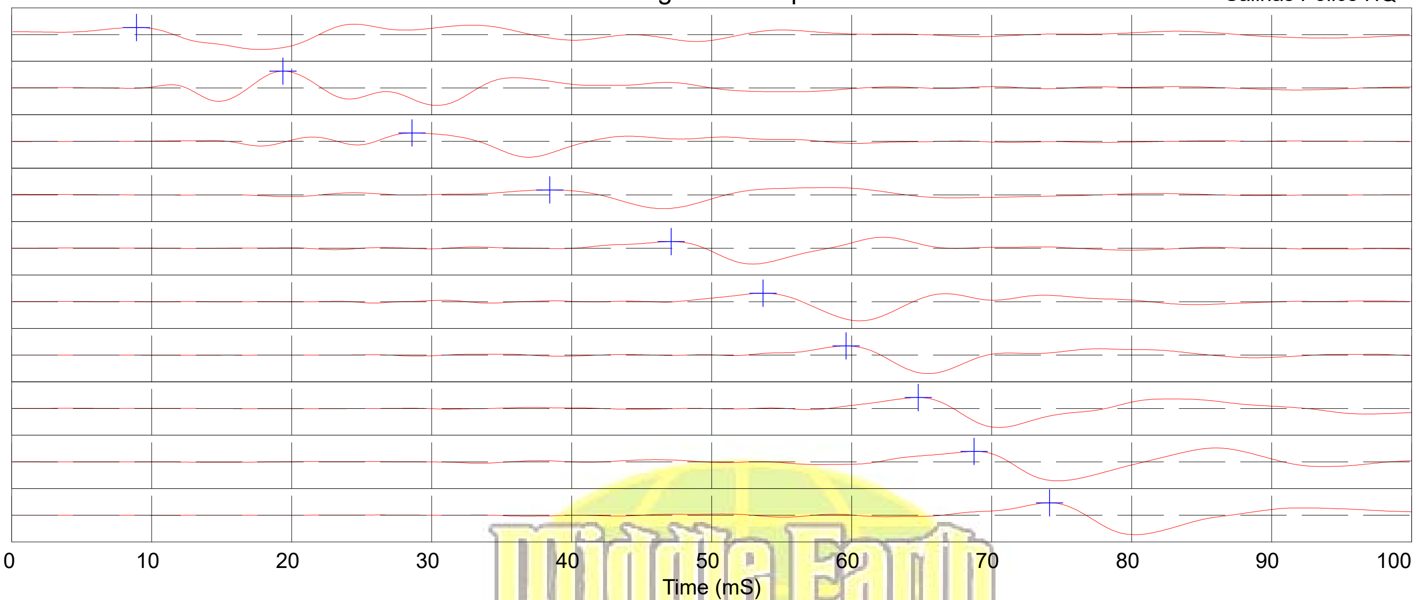


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

CPT-10 | Leighton Group | Salinas Police HQ



Depth 5.09ft	Ref*	Arrival 8.91mS	Velocity*
Depth 10.99ft	Ref 5.09ft	Arrival 19.37mS	Velocity 449.38ft/S
Depth 15.91ft	Ref 10.99ft	Arrival 28.59mS	Velocity 488.68ft/S
Depth 20.83ft	Ref 15.91ft	Arrival 38.44mS	Velocity 476.17ft/S
Depth 25.75ft	Ref 20.83ft	Arrival 47.11mS	Velocity 550.36ft/S
Depth 30.84ft	Ref 25.75ft	Arrival 53.67mS	Velocity 758.87ft/S
Depth 35.76ft	Ref 30.84ft	Arrival 59.61mS	Velocity 816.40ft/S
Depth 40.85ft	Ref 35.76ft	Arrival 64.76mS	Velocity 975.01ft/S
Depth 45.93ft	Ref 40.85ft	Arrival 68.75mS	Velocity 1264.98ft/S
Depth 50.85ft	Ref 45.93ft	Arrival 74.14mS	Velocity 906.41ft/S

Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:

APPENDIX B

Laboratory Test Results

Leighton

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	R. Manning	Date:	07/31/17
Project No. :	11693.001	Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	S1	Depth (ft.)	7.5		
Soil Identification:	Very dark gray fat clay (CH)				

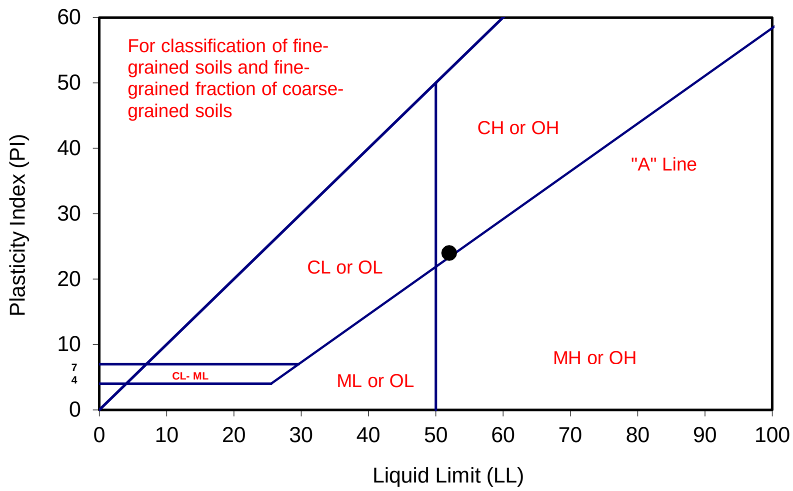
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			33	26	15	
Wet Wt. of Soil + Cont. (g)	13.83	13.37	20.86	22.15	23.71	
Dry Wt. of Soil + Cont. (g)	12.34	11.98	18.39	19.24	20.13	
Wt. of Container (g)	7.05	6.99	13.51	13.62	13.56	
Moisture Content (%) [Wn]	28.17	27.86	50.61	51.78	54.49	

Liquid Limit	**52**
Plastic Limit	**28**
Plasticity Index	**24**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 23.36

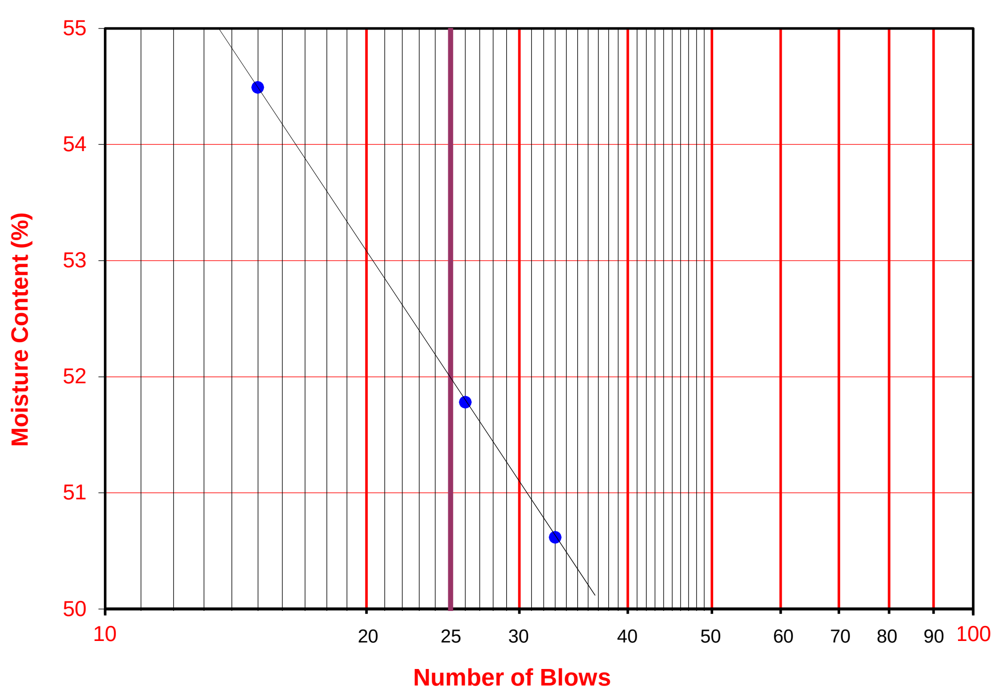
One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$



PROCEDURES USED

☐	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
☐	Procedure B One-point Test



Leighton

ATTERBERG LIMITS

ASTM D 4318

Project Name:	City of Salinas Public Safety Center	Tested By:	R. Manning	Date:	08/04/17
Project No. :	11693.001	Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	R3	Depth (ft.)	20.0		
Soil Identification:	Dark olive brown lean clay (CL)				

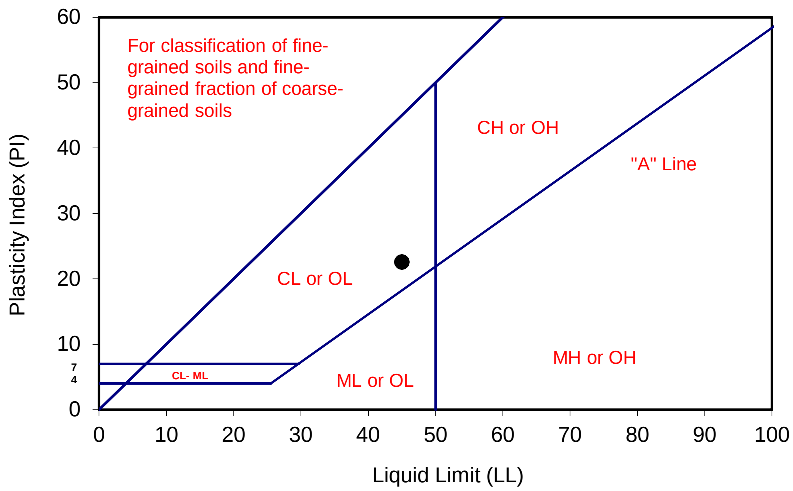
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	24	21	
Wet Wt. of Soil + Cont. (g)	13.37	12.50	21.85	19.51	21.90	
Dry Wt. of Soil + Cont. (g)	12.20	11.36	19.37	17.10	19.28	
Wt. of Container (g)	7.04	6.21	13.56	11.74	13.53	
Moisture Content (%) [Wn]	22.67	22.14	42.69	44.96	45.57	

Liquid Limit	**45**
Plastic Limit	**22**
Plasticity Index	**23**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 18.25

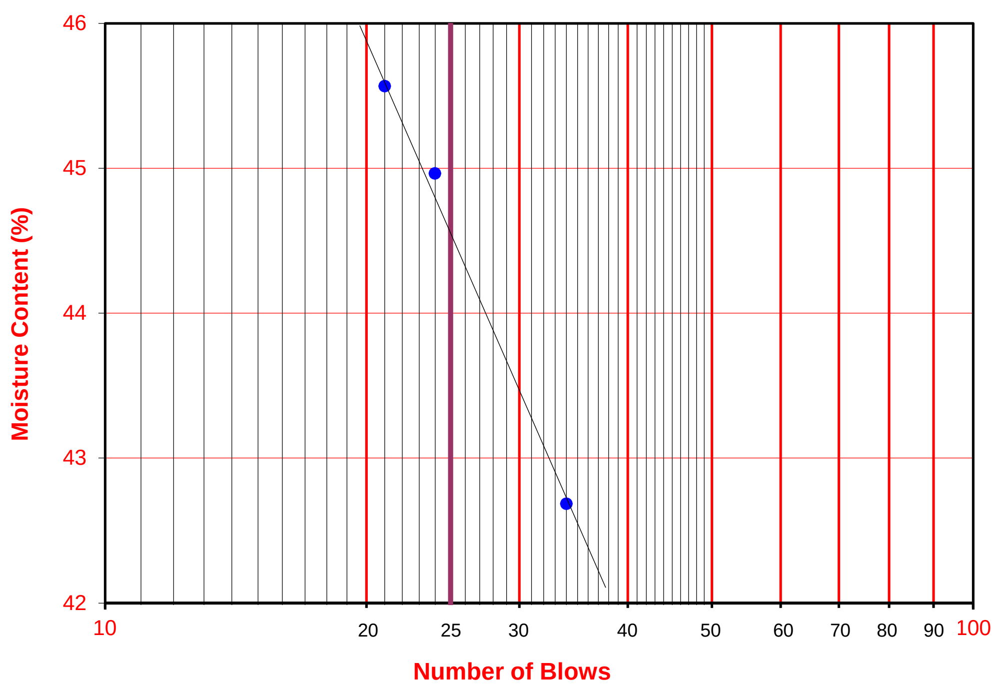
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
	Procedure B One-point Test



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	R. Manning	Date:	08/07/17
Project No. :	11693.001	Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	R4	Depth (ft.)	30.0		
Soil Identification:	Very dark gray fat clay (CH)				

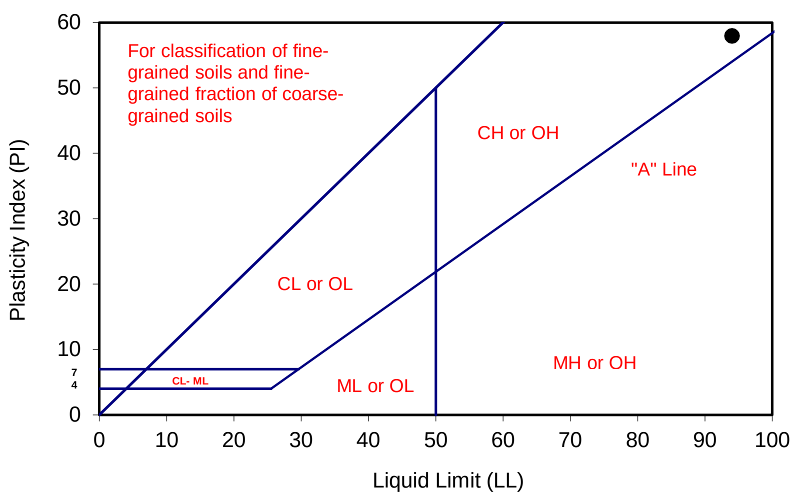
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			35	26	21	
Wet Wt. of Soil + Cont. (g)	13.69	13.37	21.50	21.76	21.64	
Dry Wt. of Soil + Cont. (g)	11.93	11.68	17.78	17.80	17.73	
Wt. of Container (g)	7.05	6.99	13.62	13.56	13.65	
Moisture Content (%) [Wn]	36.07	36.03	89.42	93.40	95.83	

Liquid Limit	**94**
Plastic Limit	**36**
Plasticity Index	**58**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 54.02

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



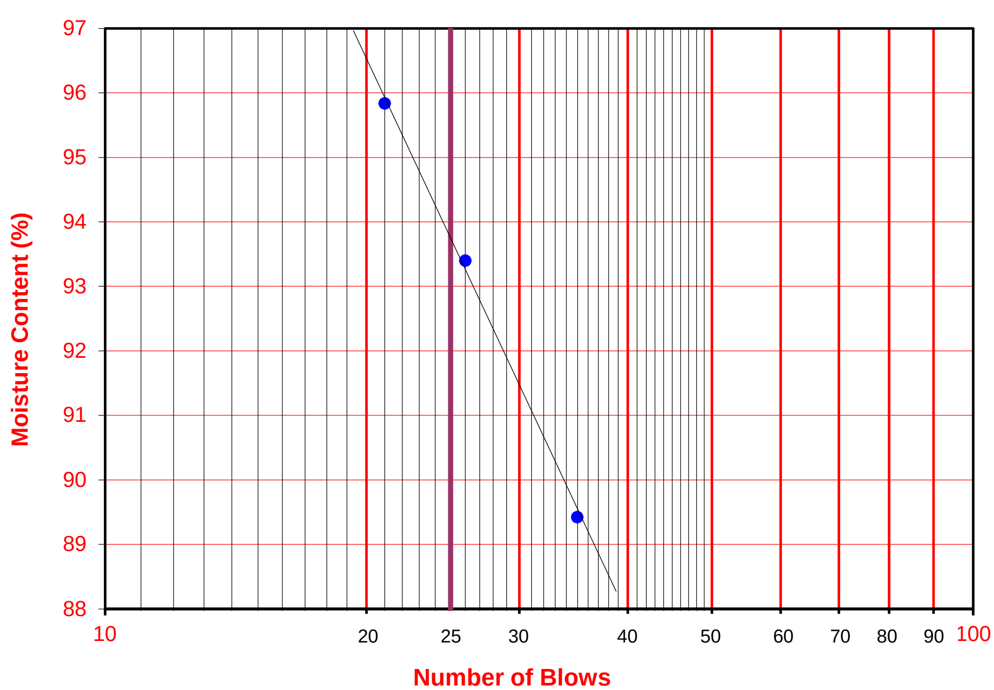
PROCEDURES USED

[ ] Wet Preparation
Multipoint - Wet

[X] Dry Preparation
Multipoint - Dry

[X] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	City of Salinas Public Safety Center	Tested By:	R. Manning	Date:	08/07/17
Project No. :	11693.001	Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	R5	Depth (ft.)	40.0		
Soil Identification:	Dark olive gray fat clay (CH)				

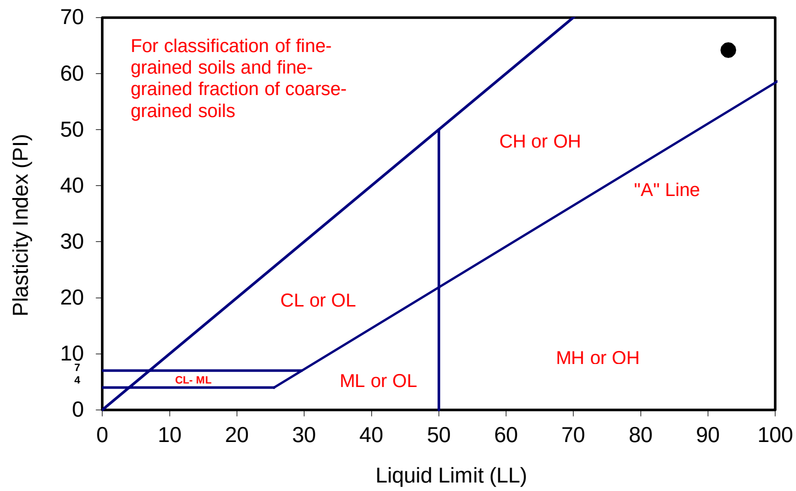
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	25	20	
Wet Wt. of Soil + Cont. (g)	13.09	12.93	21.26	21.46	22.85	
Dry Wt. of Soil + Cont. (g)	11.72	11.45	17.66	17.69	18.30	
Wt. of Container (g)	7.05	6.21	13.59	13.63	13.52	
Moisture Content (%) [Wn]	29.34	28.24	88.45	92.86	95.19	

Liquid Limit	**93**
Plastic Limit	**29**
Plasticity Index	**64**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 53.29

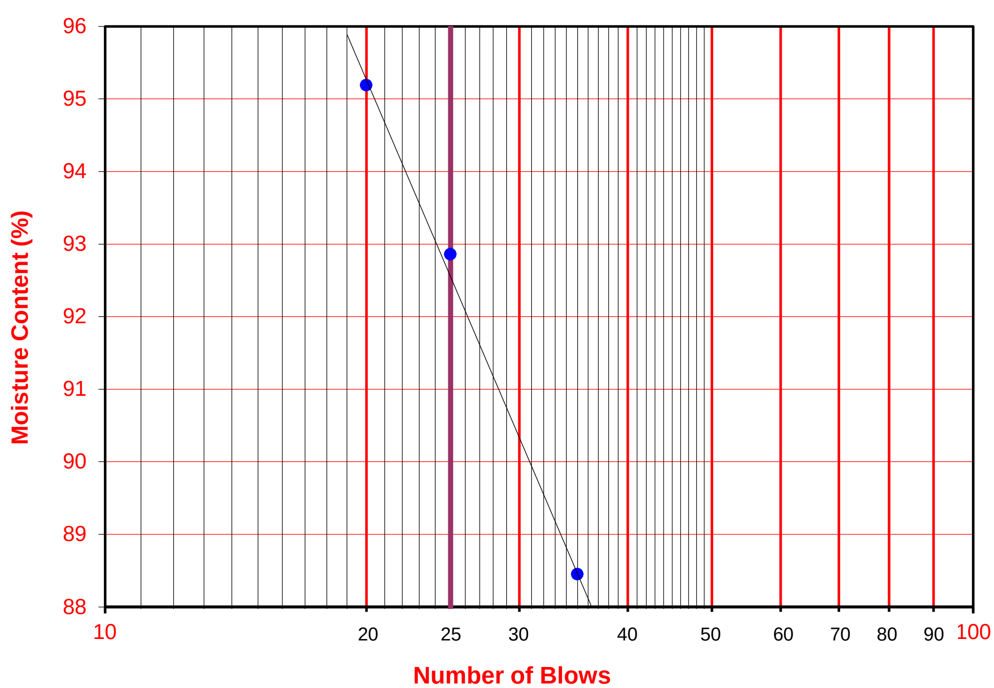
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test

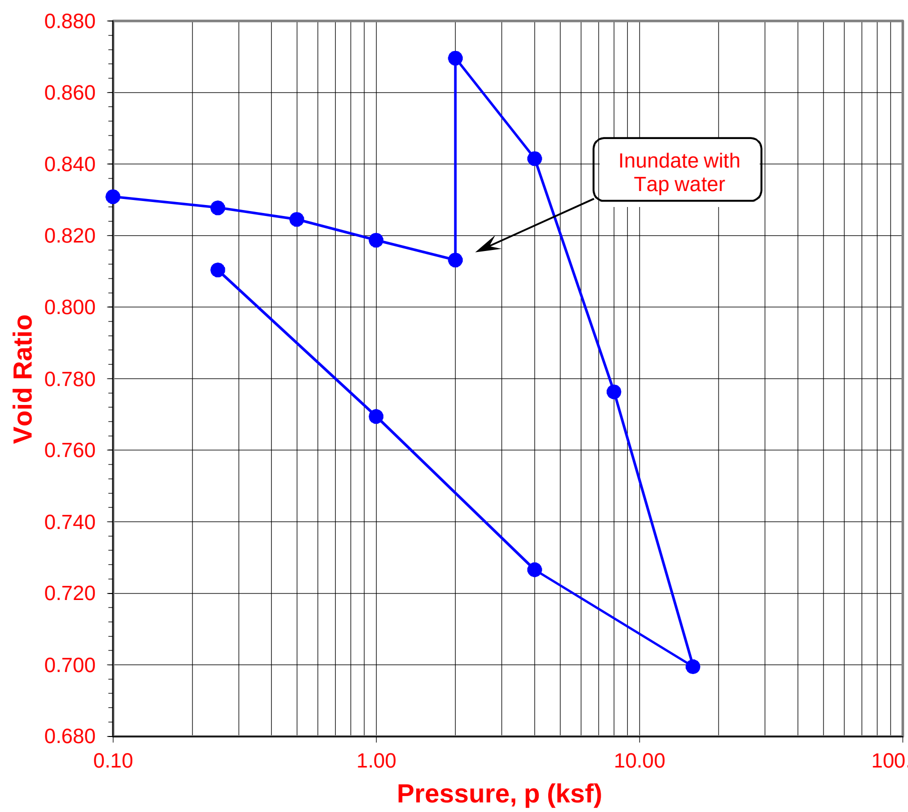


ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By: G. Bathala	Date: 08/02/17
Project No.:	11693.001	Checked By: J. Ward	Date: 08/11/17
Boring No.:	LB-1	Depth (ft.): 0-5	
Sample No.:	BB-1	Sample Type:	90% Remold
Soil Identification:	Dark olive gray fat clay (CH)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	174.03
Weight of Ring (g):	43.45
Height after consol. (in.):	0.9889
Before Test	
Wt. of Wet Sample+Cont. (g):	169.59
Wt. of Dry Sample+Cont. (g):	152.79
Weight of Container (g):	59.19
Initial Moisture Content (%)	17.9
Initial Dry Density (pcf)	92.1
Initial Saturation (%):	58
Initial Vertical Reading (in.)	0.1825
After Test	
Wt. of Wet Sample+Cont. (g):	262.26
Wt. of Dry Sample+Cont. (g):	231.08
Weight of Container (g):	76.60
Final Moisture Content (%)	28.08
Final Dry Density (pcf):	93.4
Final Saturation (%):	94
Final Vertical Reading (in.)	0.1939
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

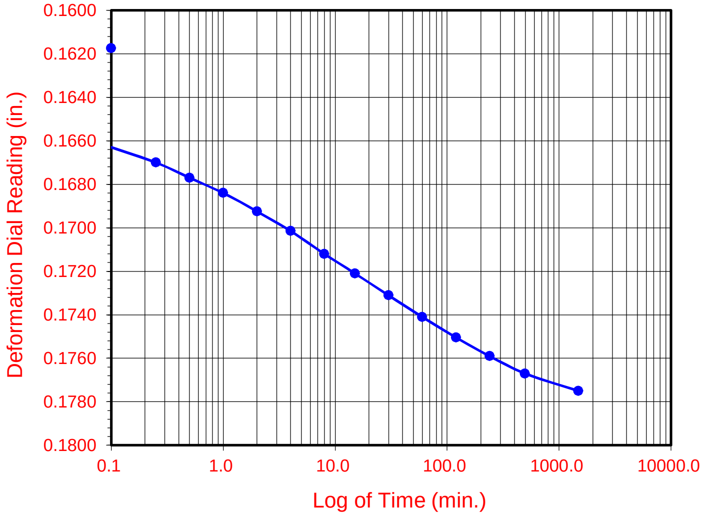


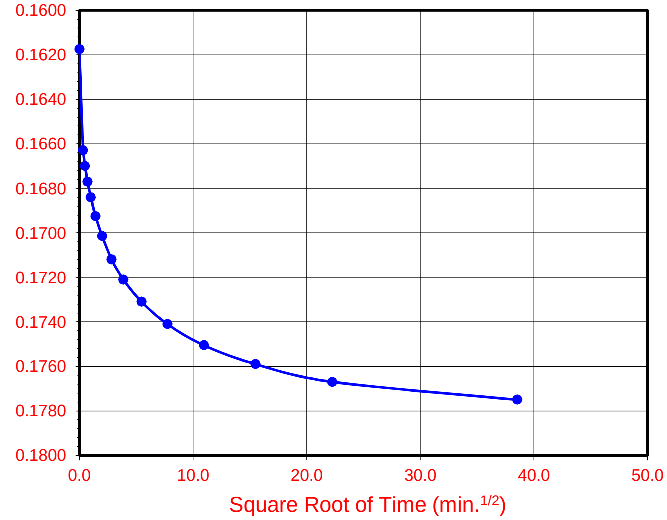
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.1824	1.0001	0.00	-0.01	0.831	-0.01
0.25	0.1842	0.9983	0.01	0.17	0.828	0.16
0.50	0.1861	0.9964	0.02	0.36	0.825	0.34
1.00	0.1895	0.9931	0.04	0.70	0.819	0.66
2.00	0.1926	0.9899	0.05	1.01	0.813	0.96
2.00	0.1618	1.0208	0.05	-2.08	0.870	-2.13
4.00	0.1775	1.0050	0.09	-0.50	0.842	-0.59
8.00	0.2135	0.9690	0.13	3.10	0.776	2.97
16.00	0.2563	0.9262	0.21	7.38	0.699	7.17
4.00	0.2405	0.9420	0.11	5.80	0.727	5.69
1.00	0.2166	0.9659	0.06	3.41	0.769	3.35
0.25	0.1939	0.9886	0.03	1.14	0.810	1.11

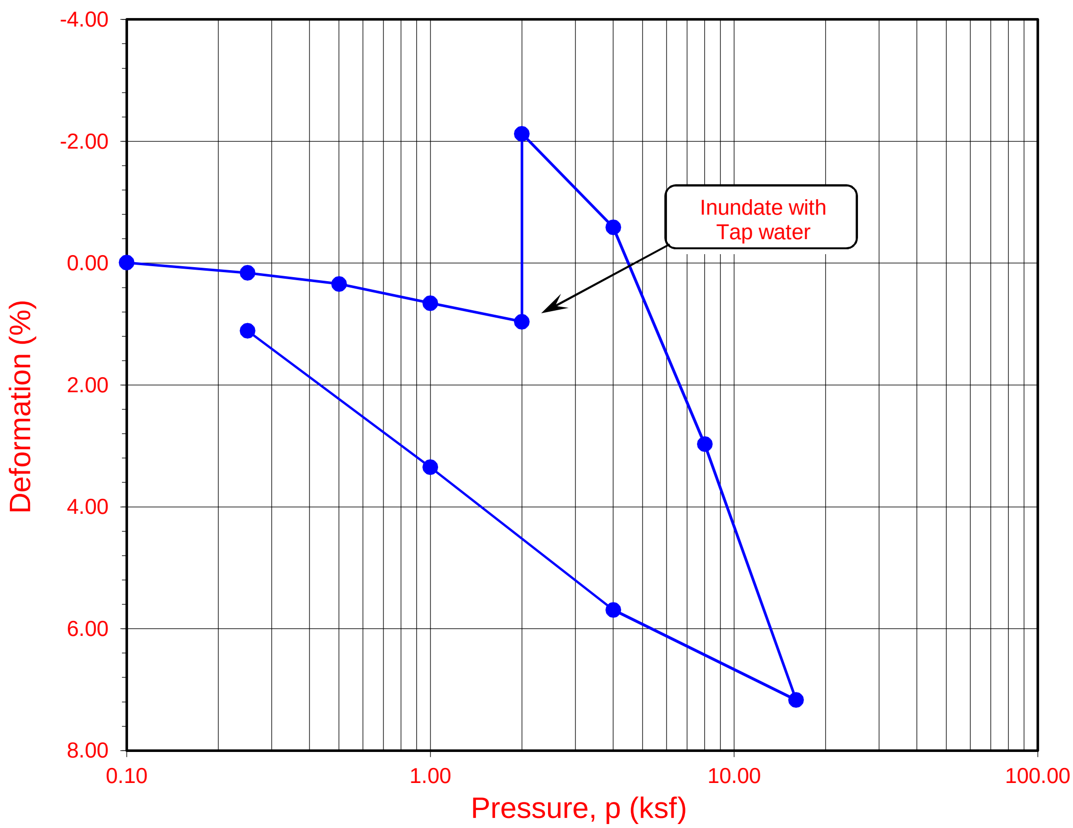
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/5/17	5:54:00	0.0	0.0	0.1618
8/5/17	5:54:06	0.1	0.3	0.1663
8/5/17	5:54:15	0.2	0.5	0.1670
8/5/17	5:54:30	0.5	0.7	0.1677
8/5/17	5:55:00	1.0	1.0	0.1684
8/5/17	5:56:00	2.0	1.4	0.1693
8/5/17	5:58:00	4.0	2.0	0.1702
8/5/17	6:02:00	8.0	2.8	0.1712
8/5/17	6:09:00	15.0	3.9	0.1721
8/5/17	6:24:00	30.0	5.5	0.1731
8/5/17	6:54:00	60.0	7.7	0.1741
8/5/17	7:54:00	120.0	11.0	0.1751
8/5/17	9:54:00	240.0	15.5	0.1759
8/5/17	14:10:00	496.0	22.3	0.1767
8/6/17	6:40:00	1486.0	38.5	0.1775

Time Readings @ 4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**BB-1**	**0-5**	**17.9**	**28.1**	**92.1**	**93.4**	**0.831**	**0.769**	**58**	**94**

Soil Identification: Dark olive gray fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

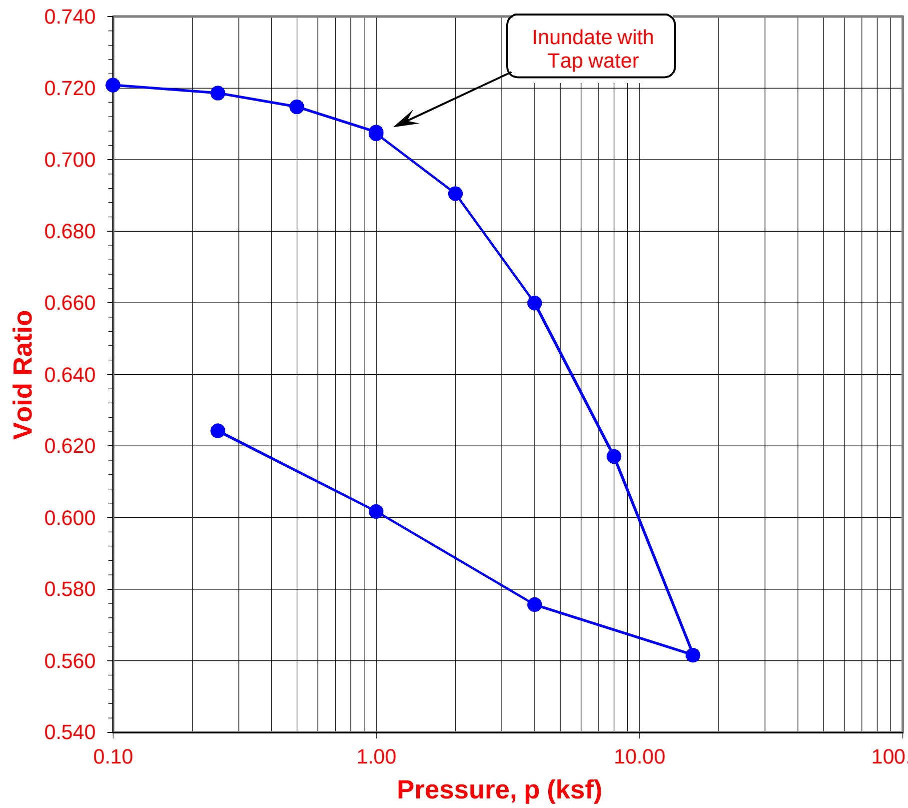
Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	City of Salinas Public Safety Center	Tested By:	G. Bathala	Date:	08/03/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Depth (ft.):	10.0		
Sample No.:	R2	Sample Type:	Ring		
Soil Identification:	Dark olive gray silty clay (CL-ML)				

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	184.57
Weight of Ring (g):	43.34
Height after consol. (in.):	0.9444
Before Test	
Wt. of Wet Sample+Cont. (g):	146.44
Wt. of Dry Sample+Cont. (g):	133.44
Weight of Container (g):	67.95
Initial Moisture Content (%)	19.9
Initial Dry Density (pcf)	98.0
Initial Saturation (%):	74
Initial Vertical Reading (in.)	0.1495
After Test	
Wt. of Wet Sample+Cont. (g):	259.44
Wt. of Dry Sample+Cont. (g):	231.28
Weight of Container (g):	77.30
Final Moisture Content (%)	25.45
Final Dry Density (pcf):	97.4
Final Saturation (%):	94
Final Vertical Reading (in.)	0.2106
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

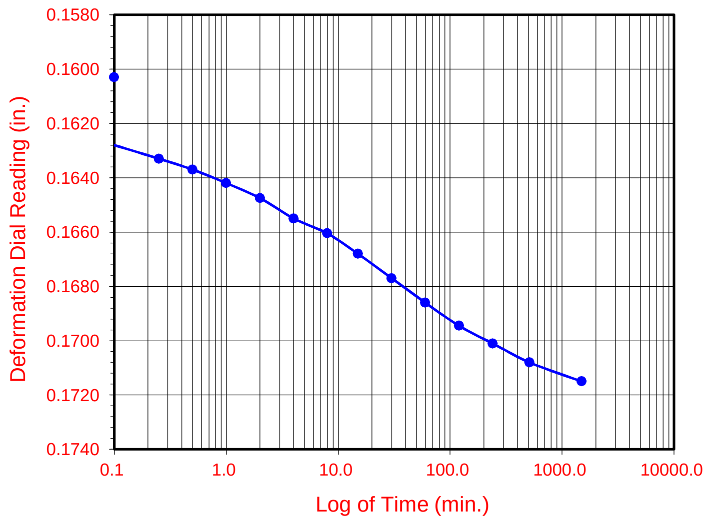


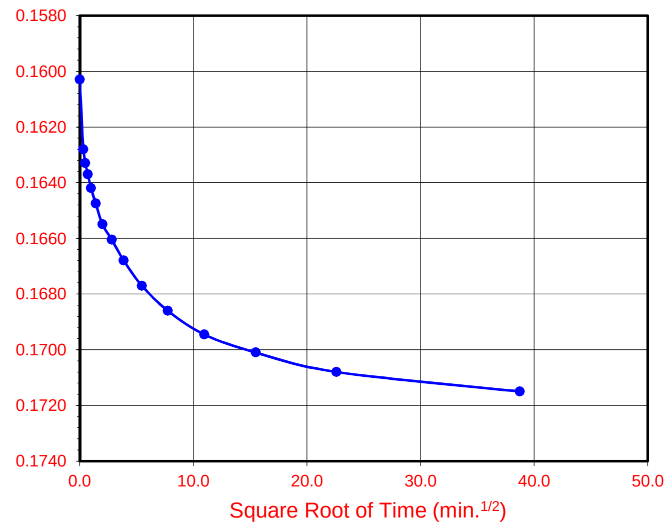
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.1490	1.0005	0.00	-0.05	0.721	-0.05
0.25	0.1512	0.9983	0.09	0.17	0.719	0.08
0.50	0.1544	0.9951	0.19	0.49	0.715	0.30
1.00	0.1601	0.9895	0.34	1.06	0.708	0.72
1.00	0.1603	0.9892	0.34	1.08	0.707	0.74
2.00	0.1715	0.9780	0.49	2.20	0.691	1.71
4.00	0.1907	0.9588	0.63	4.12	0.660	3.49
8.00	0.2168	0.9327	0.75	6.73	0.617	5.98
16.00	0.2505	0.8990	0.89	10.10	0.562	9.21
4.00	0.2409	0.9086	0.75	9.14	0.576	8.39
1.00	0.2246	0.9250	0.63	7.51	0.602	6.88
0.25	0.2106	0.9390	0.54	6.11	0.624	5.57

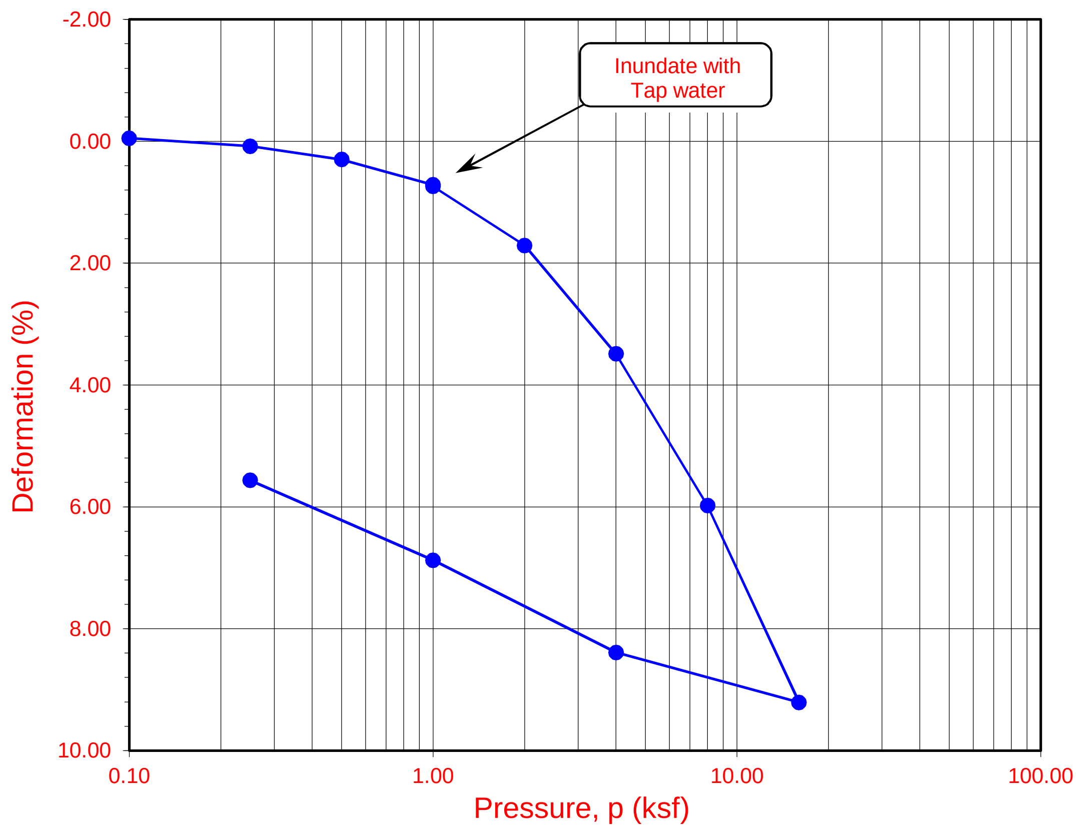
Time Readings @ 2 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/5/17	5:37:00	0.0	0.0	0.1603
8/5/17	5:37:06	0.1	0.3	0.1628
8/5/17	5:37:15	0.2	0.5	0.1633
8/5/17	5:37:30	0.5	0.7	0.1637
8/5/17	5:38:00	1.0	1.0	0.1642
8/5/17	5:39:00	2.0	1.4	0.1648
8/5/17	5:41:00	4.0	2.0	0.1655
8/5/17	5:45:00	8.0	2.8	0.1661
8/5/17	5:52:00	15.0	3.9	0.1668
8/5/17	6:07:00	30.0	5.5	0.1677
8/5/17	6:37:00	60.0	7.7	0.1686
8/5/17	7:37:00	120.0	11.0	0.1695
8/5/17	9:37:00	240.0	15.5	0.1701
8/5/17	14:08:00	511.0	22.6	0.1708
8/6/17	6:37:00	1500.0	38.7	0.1715

Time Readings @ 2 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R2**	**10**	**19.9**	**25.5**	**98.0**	**97.4**	**0.720**	**0.602**	**74**	**94**

Soil Identification: Dark olive gray silty clay (CL-ML)

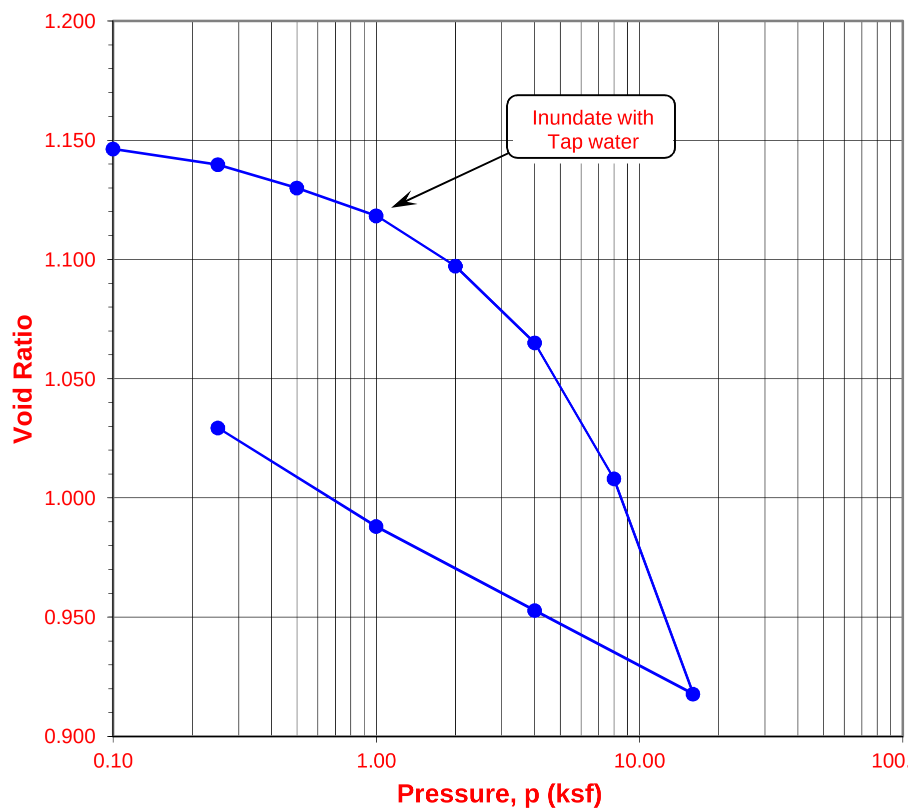
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	G. Bathala	Date:	08/03/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Depth (ft.):	20.0		
Sample No.:	R3	Sample Type:	Ring		
Soil Identification:	Dark olive brown lean clay (CL)				

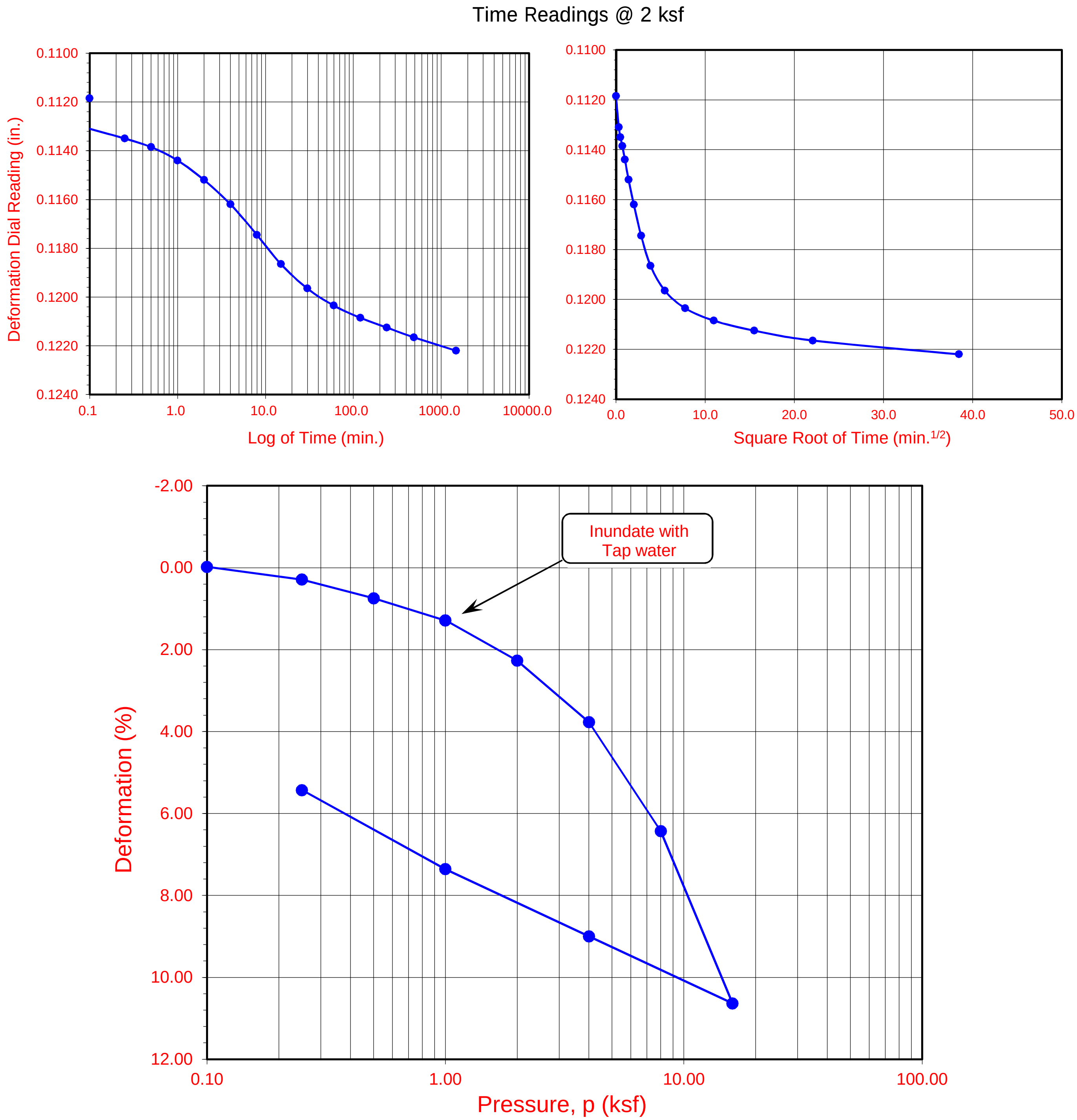
Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	175.19
Weight of Ring (g):	43.35
Height after consol. (in.):	0.9457
Before Test	
Wt. of Wet Sample+Cont. (g):	313.93
Wt. of Dry Sample+Cont. (g):	238.08
Weight of Container (g):	38.77
Initial Moisture Content (%)	38.1
Initial Dry Density (pcf)	79.4
Initial Saturation (%):	91
Initial Vertical Reading (in.)	0.0984
After Test	
Wt. of Wet Sample+Cont. (g):	247.19
Wt. of Dry Sample+Cont. (g):	208.84
Weight of Container (g):	75.98
Final Moisture Content (%)	42.84
Final Dry Density (pcf):	78.7
Final Saturation (%):	100
Final Vertical Reading (in.)	0.1541
Specific Gravity (assumed):	2.73
Water Density (pcf):	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.0982	1.0002	0.00	-0.02	1.146	-0.02
0.25	0.1014	0.9970	0.01	0.30	1.140	0.29
0.50	0.1062	0.9923	0.03	0.78	1.130	0.75
1.00	0.1119	0.9865	0.06	1.35	1.118	1.29
1.00	0.1119	0.9866	0.06	1.35	1.118	1.29
2.00	0.1222	0.9762	0.11	2.38	1.097	2.27
4.00	0.1381	0.9603	0.20	3.97	1.065	3.77
8.00	0.1655	0.9329	0.28	6.71	1.008	6.43
16.00	0.2086	0.8899	0.38	11.02	0.918	10.64
4.00	0.1910	0.9074	0.26	9.26	0.953	9.00
1.00	0.1738	0.9246	0.18	7.54	0.988	7.36
0.25	0.1541	0.9443	0.14	5.57	1.029	5.43

Time Readings @ 2 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/5/17	6:04:00	0.0	0.0	0.1119
8/5/17	6:04:06	0.1	0.3	0.1131
8/5/17	6:04:15	0.2	0.5	0.1135
8/5/17	6:04:30	0.5	0.7	0.1139
8/5/17	6:05:00	1.0	1.0	0.1144
8/5/17	6:06:00	2.0	1.4	0.1152
8/5/17	6:08:00	4.0	2.0	0.1162
8/5/17	6:12:00	8.0	2.8	0.1175
8/5/17	6:19:00	15.0	3.9	0.1187
8/5/17	6:34:00	30.0	5.5	0.1197
8/5/17	7:04:00	60.0	7.7	0.1204
8/5/17	8:04:00	120.0	11.0	0.1209
8/5/17	10:04:00	240.0	15.5	0.1213
8/5/17	14:11:00	487.0	22.1	0.1217
8/6/17	6:43:00	1479.0	38.5	0.1222



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R3**	**20**	**38.1**	**42.8**	**79.4**	**78.7**	**1.146**	**0.988**	**91**	**100**

Soil Identification: Dark olive brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

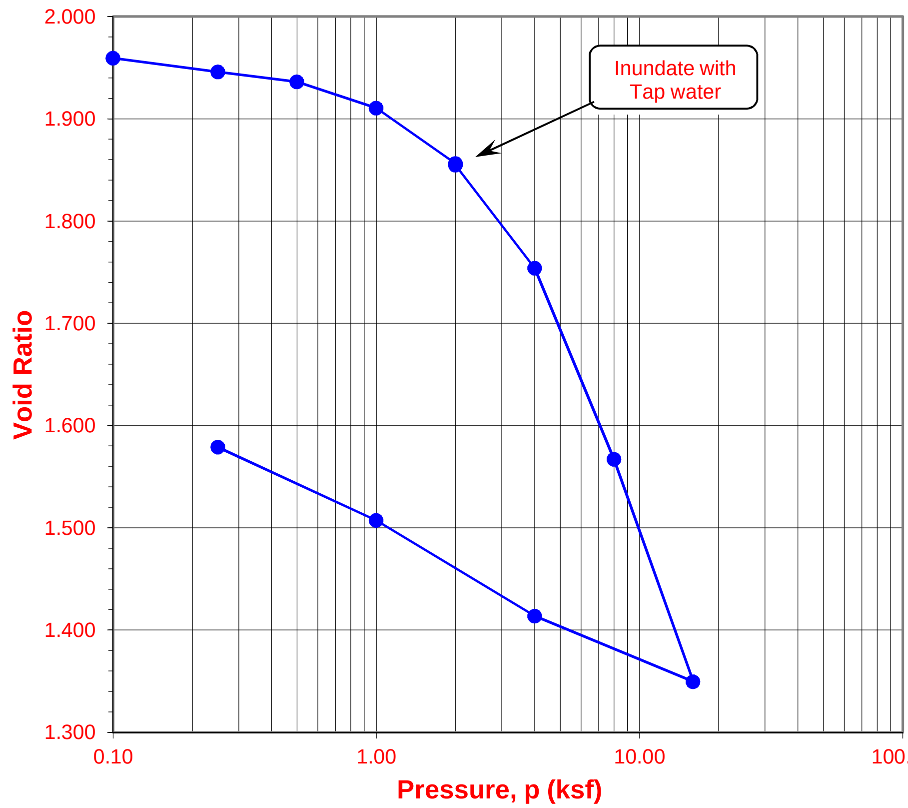
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By: G. Bathala	Date: 08/01/17
Project No.:	11693.001	Checked By: J. Ward	Date: 08/11/17
Boring No.:	LB-1	Depth (ft.): 30.0	
Sample No.:	R4	Sample Type: Ring	
Soil Identification:	Very dark gray fat clay (CH)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	151.58
Weight of Ring (g):	43.36
Height after consol. (in.):	0.8718
Before Test	
Wt. of Wet Sample+Cont. (g):	288.86
Wt. of Dry Sample+Cont. (g):	196.99
Weight of Container (g):	38.49
Initial Moisture Content (%)	58.0
Initial Dry Density (pcf)	57.0
Initial Saturation (%):	80
Initial Vertical Reading (in.)	0.1623
After Test	
Wt. of Wet Sample+Cont. (g):	224.58
Wt. of Dry Sample+Cont. (g):	182.57
Weight of Container (g):	80.27
Final Moisture Content (%)	71.28
Final Dry Density (pcf):	56.2
Final Saturation (%):	96
Final Vertical Reading (in.)	0.2914
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

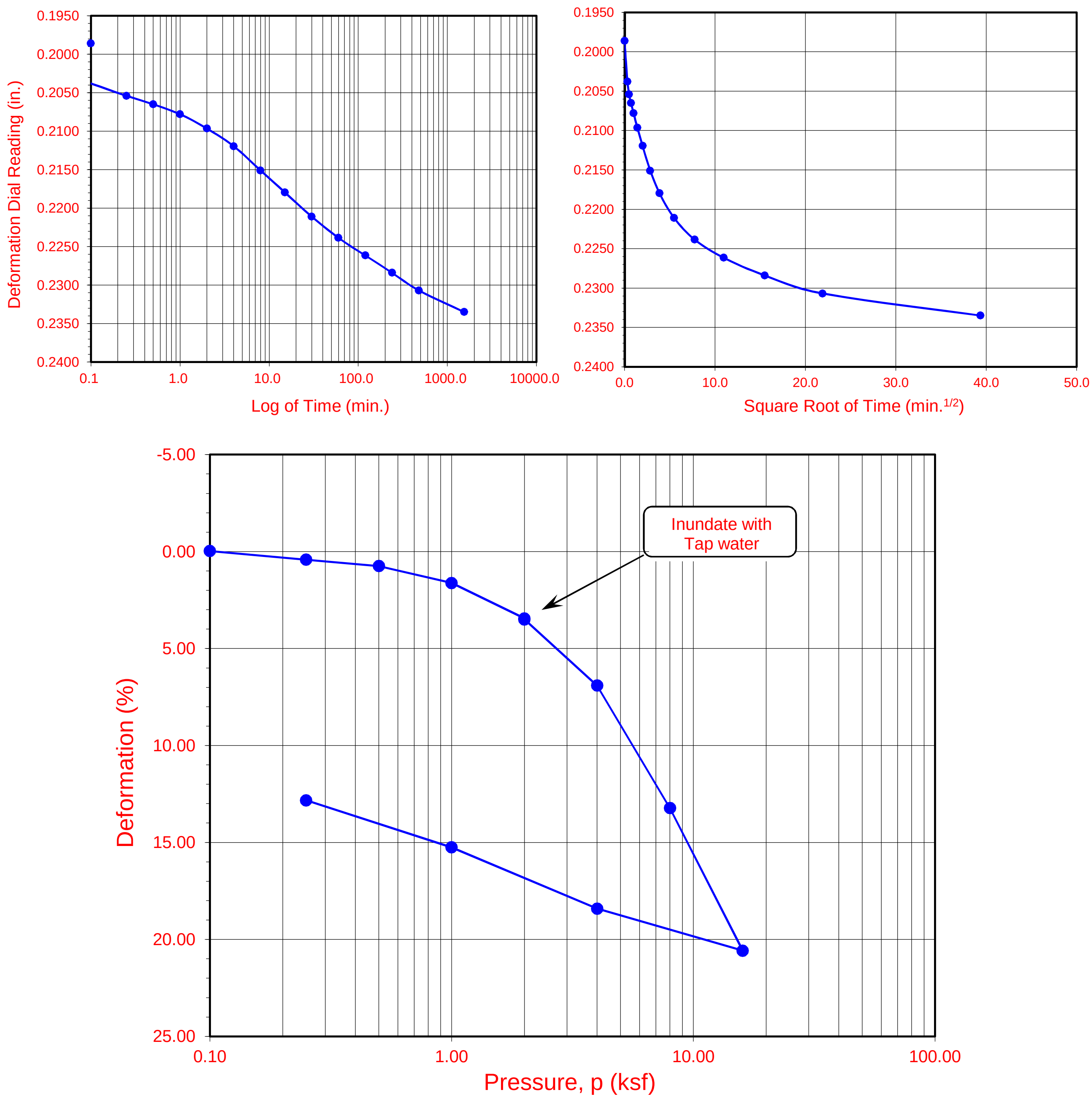


Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1620	1.0003	0.00	-0.03	1.959	-0.03
0.25	0.1666	0.9957	0.01	0.43	1.946	0.42
0.50	0.1703	0.9920	0.05	0.80	1.936	0.75
1.00	0.1793	0.9830	0.08	1.70	1.910	1.62
2.00	0.1981	0.9643	0.13	3.57	1.856	3.44
2.00	0.1986	0.9637	0.13	3.63	1.855	3.50
4.00	0.2335	0.9288	0.21	7.12	1.754	6.91
8.00	0.2979	0.8644	0.33	13.56	1.567	13.23
16.00	0.3733	0.7890	0.52	21.10	1.350	20.58
4.00	0.3489	0.8134	0.25	18.66	1.414	18.41
1.00	0.3162	0.8461	0.14	15.39	1.507	15.25
0.25	0.2914	0.8710	0.08	12.91	1.579	12.83

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/4/17	7:45:00	0.0	0.0	0.1986
8/4/17	7:45:06	0.1	0.3	0.2038
8/4/17	7:45:15	0.2	0.5	0.2054
8/4/17	7:45:30	0.5	0.7	0.2065
8/4/17	7:46:00	1.0	1.0	0.2078
8/4/17	7:47:00	2.0	1.4	0.2097
8/4/17	7:49:00	4.0	2.0	0.2120
8/4/17	7:53:00	8.0	2.8	0.2151
8/4/17	8:00:00	15.0	3.9	0.2180
8/4/17	8:15:00	30.0	5.5	0.2211
8/4/17	8:45:00	60.0	7.7	0.2239
8/4/17	9:45:00	120.0	11.0	0.2262
8/4/17	11:45:00	240.0	15.5	0.2284
8/4/17	15:45:00	480.0	21.9	0.2307
8/5/17	9:35:00	1550.0	39.4	0.2335

Time Readings @ 4 ksf



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R4**	**30**	**58.0**	**71.3**	**57.0**	**56.2**	**1.958**	**1.507**	**80**	**96**

Soil Identification: Very dark gray fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

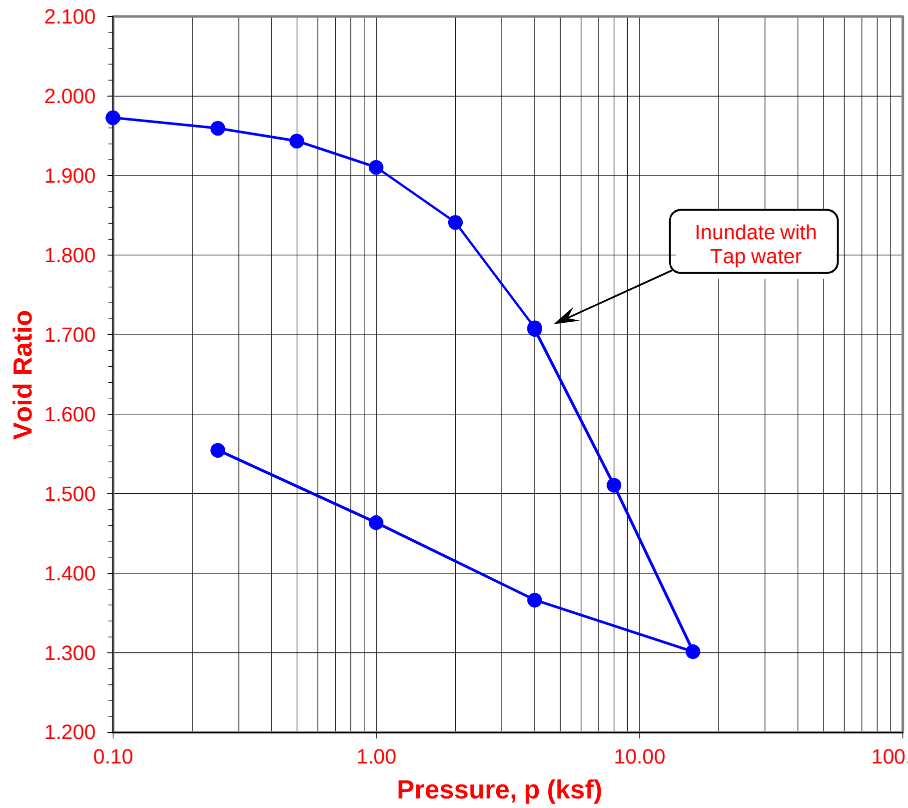
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By: G. Bathala	Date:	08/01/17
Project No.:	11693.001	Checked By: J. Ward	Date:	08/11/17
Boring No.:	LB-1	Depth (ft.): 40.0		
Sample No.:	R5	Sample Type:	Ring	
Soil Identification:	Dark olive gray fat clay (CH)			

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	168.78
Weight of Ring (g):	43.99
Height after consol. (in.):	0.8592
Before Test	
Wt. of Wet Sample+Cont. (g):	271.45
Wt. of Dry Sample+Cont. (g):	178.31
Weight of Container (g):	39.31
Initial Moisture Content (%)	67.0
Initial Dry Density (pcf)	62.1
Initial Saturation (%):	100
Initial Vertical Reading (in.)	0.1276
After Test	
Wt. of Wet Sample+Cont. (g):	233.64
Wt. of Dry Sample+Cont. (g):	197.34
Weight of Container (g):	75.01
Final Moisture Content (%)	46.34
Final Dry Density (pcf):	75.8
Final Saturation (%):	95
Final Vertical Reading (in.)	0.2690
Specific Gravity (assumed):	2.96
Water Density (pcf):	62.43

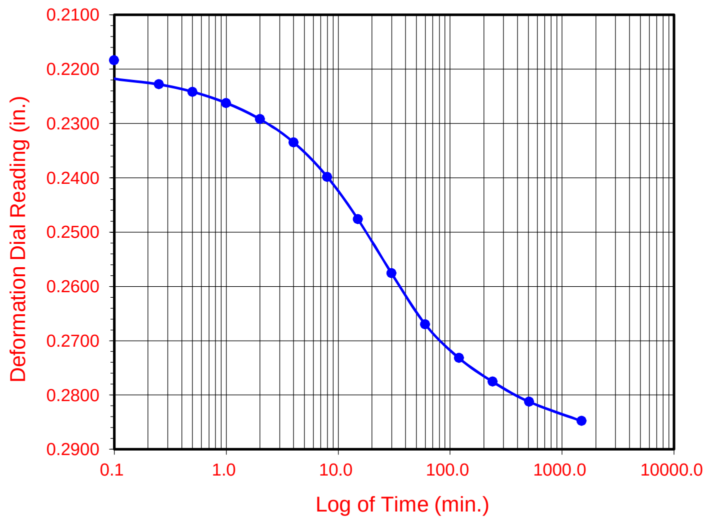


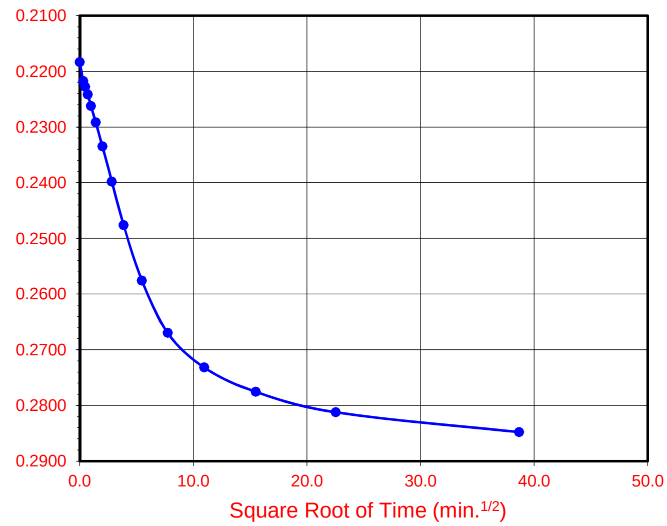
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1279	0.9997	0.00	0.03	1.973	0.03
0.25	0.1325	0.9952	0.01	0.49	1.960	0.48
0.50	0.1379	0.9897	0.02	1.03	1.944	1.01
1.00	0.1492	0.9784	0.03	2.16	1.910	2.13
2.00	0.1727	0.9549	0.06	4.51	1.841	4.45
4.00	0.2177	0.9100	0.10	9.01	1.709	8.91
4.00	0.2184	0.9092	0.10	9.08	1.707	8.98
8.00	0.2848	0.8428	0.15	15.72	1.511	15.57
16.00	0.3559	0.7717	0.22	22.83	1.301	22.61
4.00	0.3332	0.7944	0.14	20.56	1.366	20.42
1.00	0.2999	0.8277	0.08	17.23	1.464	17.15
0.25	0.2690	0.8587	0.05	14.14	1.555	14.09

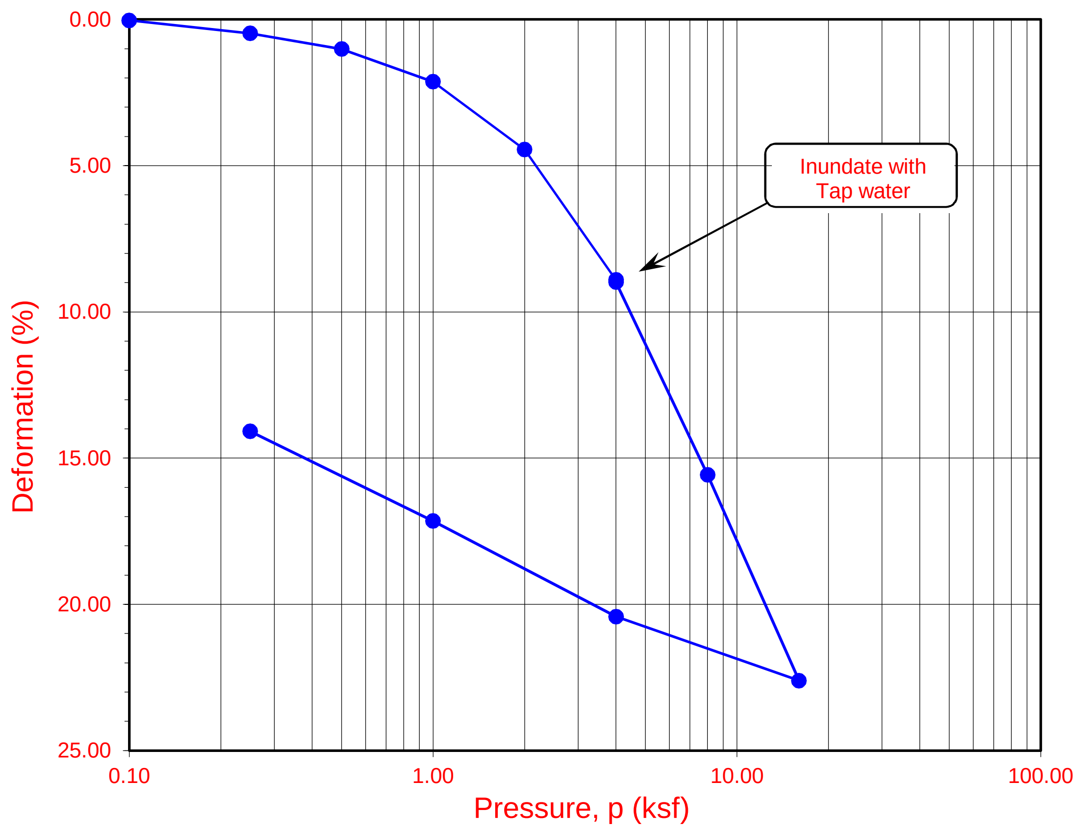
Time Readings @ 8 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/5/17	5:41:00	0.0	0.0	0.2184
8/5/17	5:41:06	0.1	0.3	0.2218
8/5/17	5:41:15	0.2	0.5	0.2228
8/5/17	5:41:30	0.5	0.7	0.2242
8/5/17	5:42:00	1.0	1.0	0.2263
8/5/17	5:43:00	2.0	1.4	0.2292
8/5/17	5:45:00	4.0	2.0	0.2335
8/5/17	5:49:00	8.0	2.8	0.2399
8/5/17	5:56:00	15.0	3.9	0.2477
8/5/17	6:11:00	30.0	5.5	0.2576
8/5/17	6:41:00	60.0	7.7	0.2670
8/5/17	7:41:00	120.0	11.0	0.2732
8/5/17	9:41:00	240.0	15.5	0.2776
8/5/17	14:09:00	508.0	22.5	0.2813
8/6/17	6:38:00	1497.0	38.7	0.2848

Time Readings @ 8 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R5**	**40**	**67.0**	**46.3**	**62.1**	**75.8**	**1.974**	**1.464**	**100**	**95**

Soil Identification: Dark olive gray fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

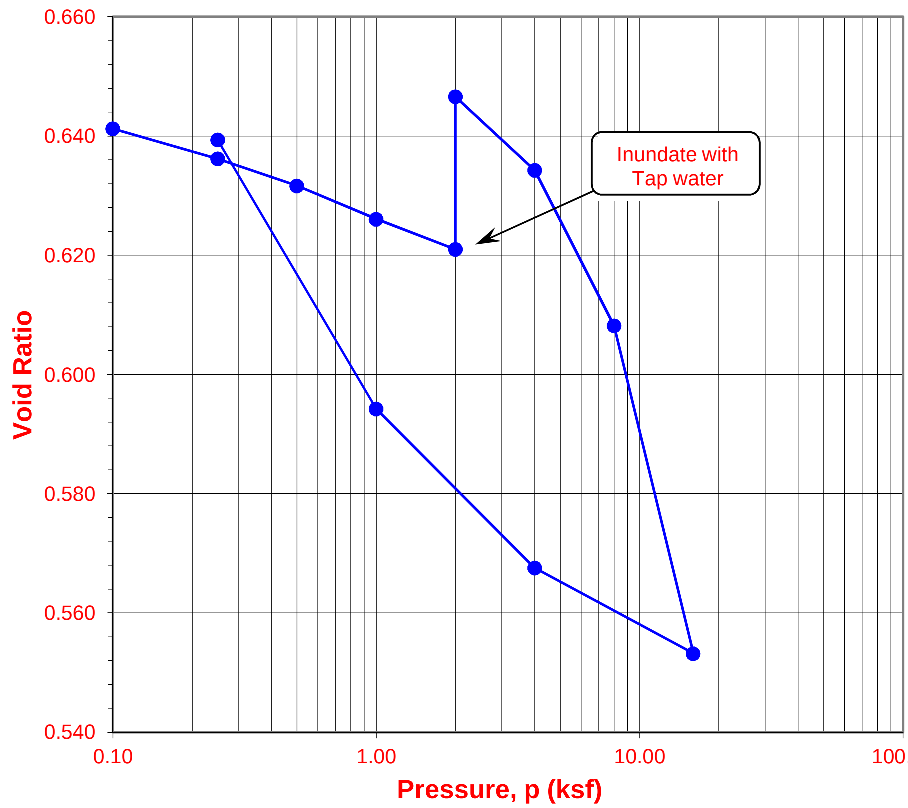
08-17

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	City of Salinas Public Safety Center	Tested By: G. Bathala	Date: 08/02/17
Project No.:	11693.001	Checked By: J. Ward	Date: 08/11/17
Boring No.:	LB-2	Depth (ft.): 0-5	
Sample No.:	BB-1	Sample Type:	90% Remold
Soil Identification:	Olive brown lean clay (CL)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	184.62
Weight of Ring (g):	43.50
Height after consol. (in.):	0.9985
Before Test	
Wt. of Wet Sample+Cont. (g):	179.89
Wt. of Dry Sample+Cont. (g):	165.94
Weight of Container (g):	68.49
Initial Moisture Content (%)	14.3
Initial Dry Density (pcf)	102.7
Initial Saturation (%):	60
Initial Vertical Reading (in.)	0.1518
After Test	
Wt. of Wet Sample+Cont. (g):	240.22
Wt. of Dry Sample+Cont. (g):	211.67
Weight of Container (g):	45.09
Final Moisture Content (%)	23.20
Final Dry Density (pcf):	102.5
Final Saturation (%):	97
Final Vertical Reading (in.)	0.1550
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

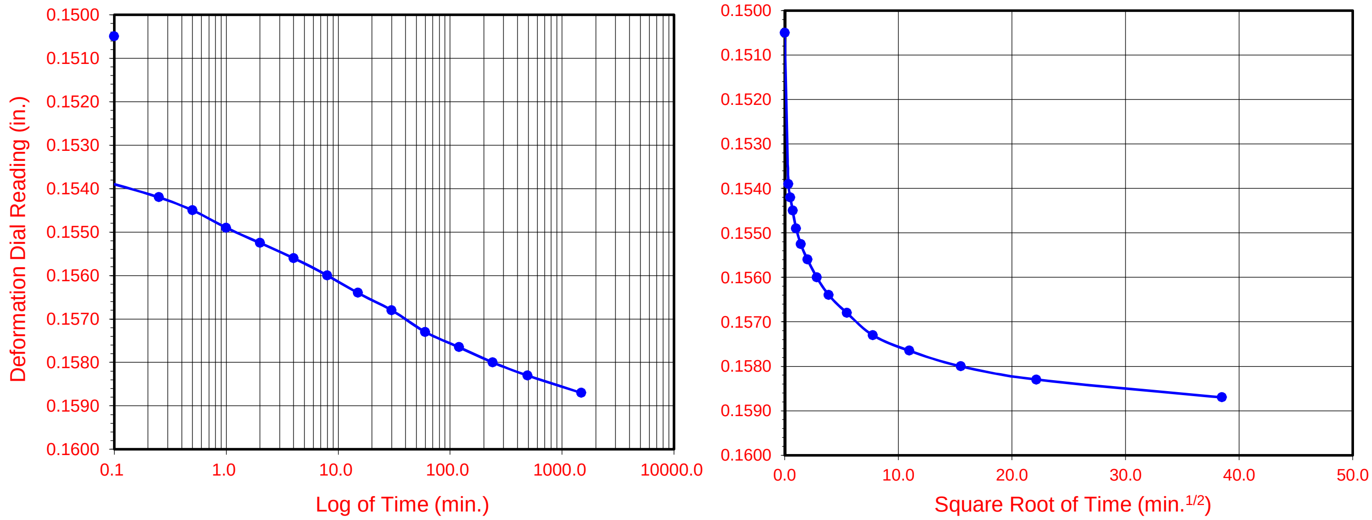


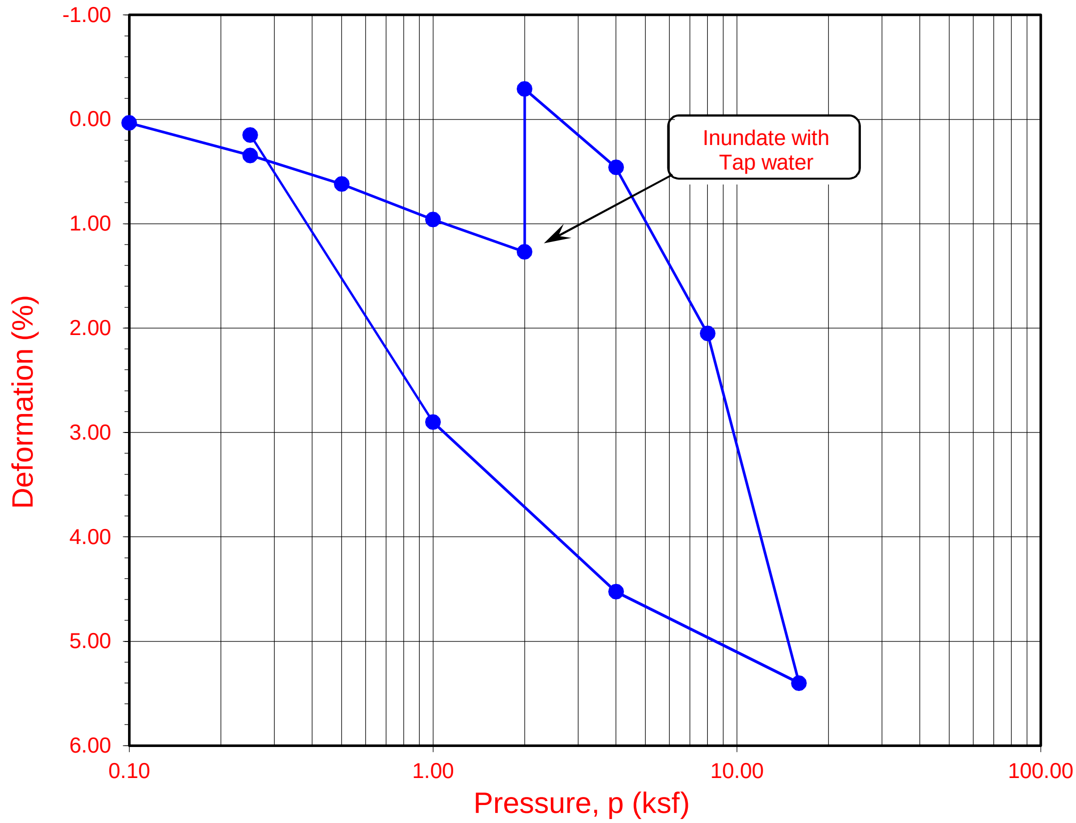
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1522	0.9997	0.00	0.04	0.641	0.04
0.25	0.1555	0.9964	0.02	0.37	0.636	0.35
0.50	0.1585	0.9933	0.05	0.67	0.632	0.62
1.00	0.1624	0.9894	0.10	1.06	0.626	0.96
2.00	0.1661	0.9857	0.16	1.43	0.621	1.27
2.00	0.1505	1.0013	0.16	-0.13	0.647	-0.29
4.00	0.1587	0.9931	0.23	0.69	0.634	0.46
8.00	0.1754	0.9764	0.31	2.36	0.608	2.05
16.00	0.2098	0.9420	0.40	5.80	0.553	5.40
4.00	0.2001	0.9518	0.30	4.83	0.568	4.53
1.00	0.1831	0.9687	0.23	3.13	0.594	2.90
0.25	0.1550	0.9968	0.17	0.32	0.639	0.15

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/5/17	5:59:00	0.0	0.0	0.1505
8/5/17	5:59:06	0.1	0.3	0.1539
8/5/17	5:59:15	0.2	0.5	0.1542
8/5/17	5:59:30	0.5	0.7	0.1545
8/5/17	6:00:00	1.0	1.0	0.1549
8/5/17	6:01:00	2.0	1.4	0.1553
8/5/17	6:03:00	4.0	2.0	0.1556
8/5/17	6:07:00	8.0	2.8	0.1560
8/5/17	6:14:00	15.0	3.9	0.1564
8/5/17	6:29:00	30.0	5.5	0.1568
8/5/17	6:59:00	60.0	7.7	0.1573
8/5/17	7:59:00	120.0	11.0	0.1577
8/5/17	9:59:00	240.0	15.5	0.1580
8/5/17	14:10:00	491.0	22.2	0.1583
8/6/17	6:41:00	1482.0	38.5	0.1587

Time Readings @ 4 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**BB-1**	**0-5**	**14.3**	**23.2**	**102.7**	**102.5**	**0.642**	**0.594**	**60**	**97**

Soil Identification: Olive brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

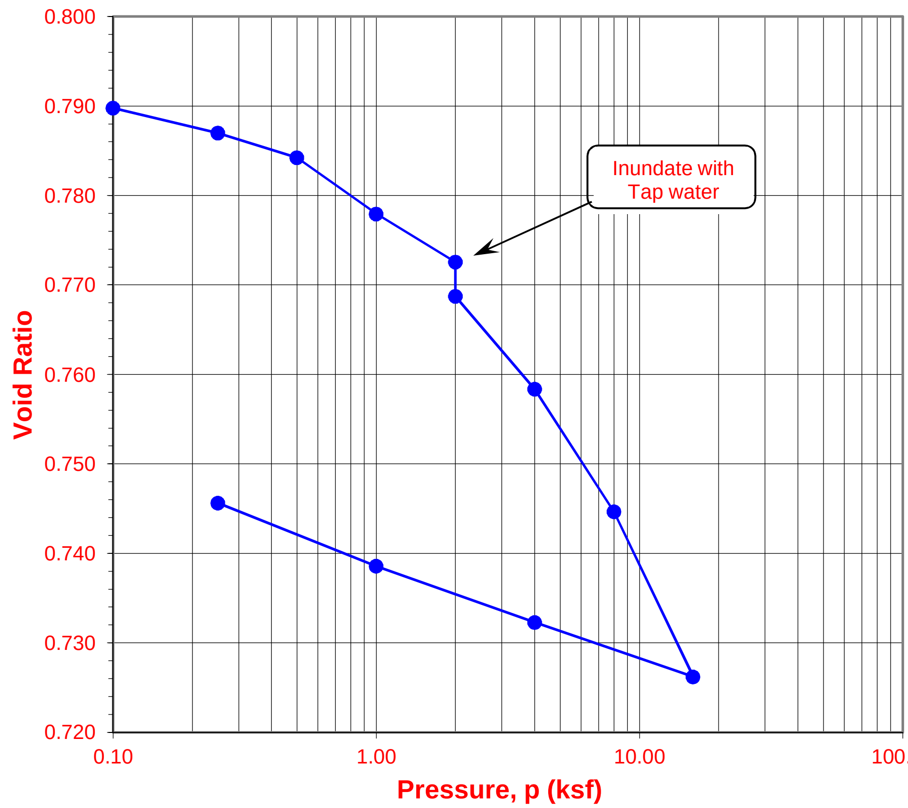
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By: G. Bathala	Date:	08/01/17
Project No.:	11693.001	Checked By: J. Ward	Date:	08/11/17
Boring No.:	LB-2	Depth (ft.): 10.0		
Sample No.:	R2	Sample Type:	Ring	
Soil Identification:	Yellowish brown silty sand (SM)			

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	164.35
Weight of Ring (g):	44.84
Height after consol. (in.):	0.9751
Before Test	
Wt. of Wet Sample+Cont. (g):	186.65
Wt. of Dry Sample+Cont. (g):	179.81
Weight of Container (g):	56.89
Initial Moisture Content (%)	5.6
Initial Dry Density (pcf)	94.2
Initial Saturation (%):	19
Initial Vertical Reading (in.)	0.1328
After Test	
Wt. of Wet Sample+Cont. (g):	254.30
Wt. of Dry Sample+Cont. (g):	227.30
Weight of Container (g):	70.35
Final Moisture Content (%)	24.08
Final Dry Density (pcf):	95.6
Final Saturation (%):	85
Final Vertical Reading (in.)	0.1582
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

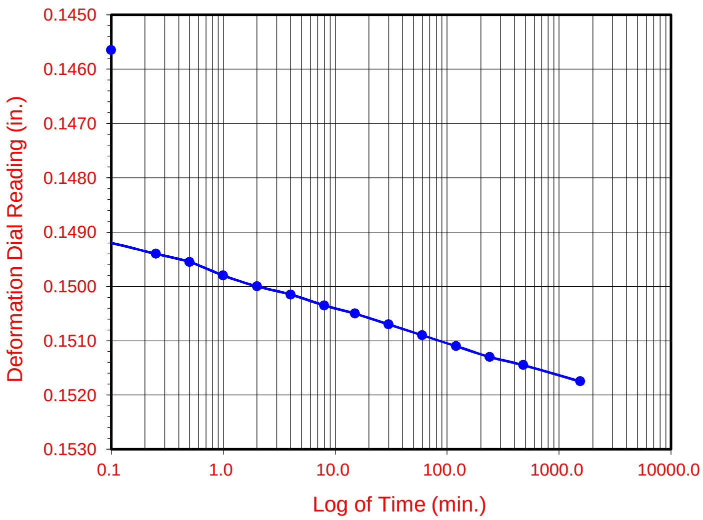


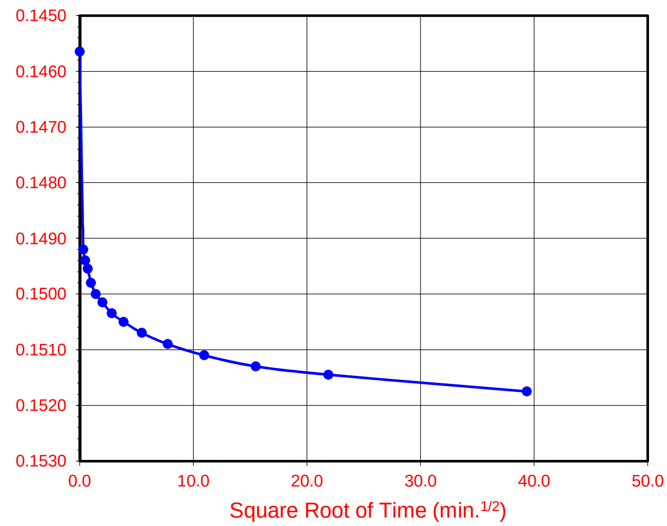
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1331	0.9997	0.00	0.03	0.790	0.03
0.25	0.1348	0.9981	0.01	0.20	0.787	0.19
0.50	0.1365	0.9963	0.03	0.37	0.784	0.34
1.00	0.1402	0.9926	0.05	0.74	0.778	0.69
2.00	0.1435	0.9893	0.08	1.07	0.773	0.99
2.00	0.1457	0.9872	0.08	1.29	0.769	1.21
4.00	0.1518	0.9811	0.11	1.90	0.758	1.79
8.00	0.1600	0.9728	0.17	2.72	0.745	2.55
16.00	0.1710	0.9618	0.24	3.82	0.726	3.58
4.00	0.1667	0.9661	0.15	3.39	0.732	3.24
1.00	0.1625	0.9703	0.08	2.97	0.739	2.89
0.25	0.1582	0.9747	0.04	2.54	0.746	2.50

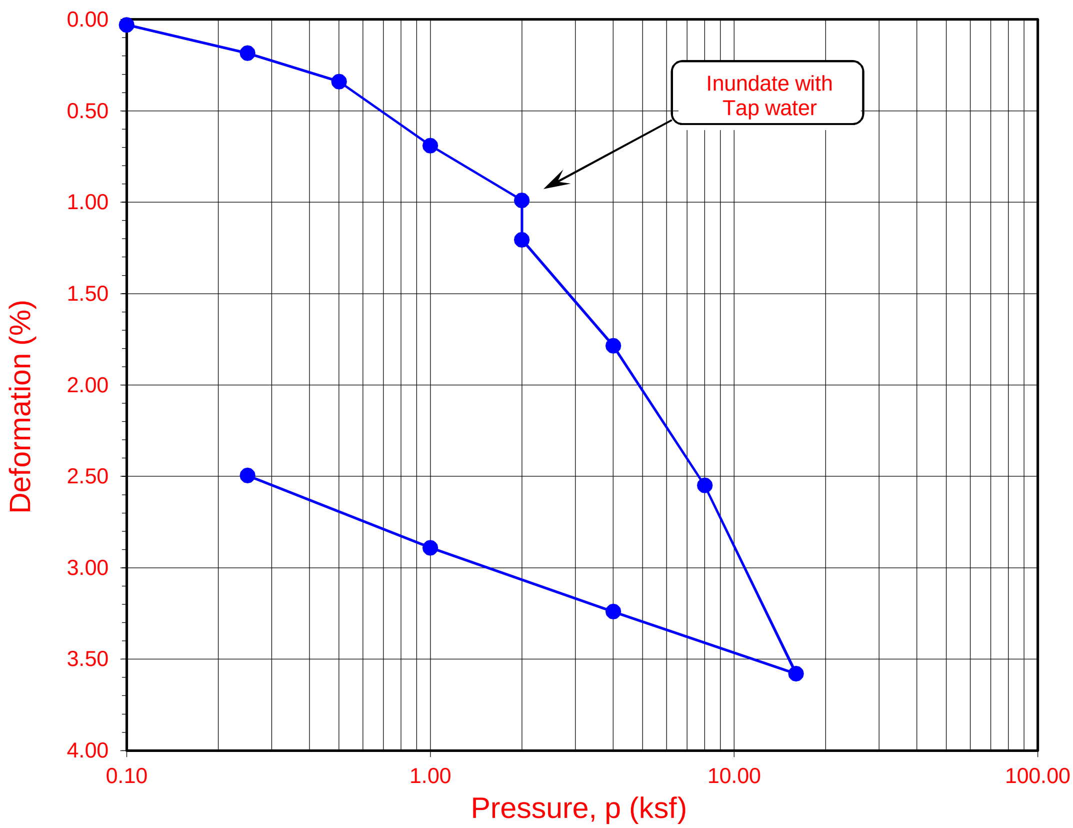
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/4/17	7:50:00	0.0	0.0	0.1457
8/4/17	7:50:06	0.1	0.3	0.1492
8/4/17	7:50:15	0.2	0.5	0.1494
8/4/17	7:50:30	0.5	0.7	0.1496
8/4/17	7:51:00	1.0	1.0	0.1498
8/4/17	7:52:00	2.0	1.4	0.1500
8/4/17	7:54:00	4.0	2.0	0.1502
8/4/17	7:58:00	8.0	2.8	0.1504
8/4/17	8:05:00	15.0	3.9	0.1505
8/4/17	8:20:00	30.0	5.5	0.1507
8/4/17	8:50:00	60.0	7.7	0.1509
8/4/17	9:50:00	120.0	11.0	0.1511
8/4/17	11:50:00	240.0	15.5	0.1513
8/4/17	15:50:00	480.0	21.9	0.1515
8/5/17	9:39:00	1549.0	39.4	0.1518

Time Readings @ 4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**R2**	**10**	**5.6**	**24.1**	**94.2**	**95.6**	**0.790**	**0.739**	**19**	**85**

Soil Identification: Yellowish brown silty sand (SM)

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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

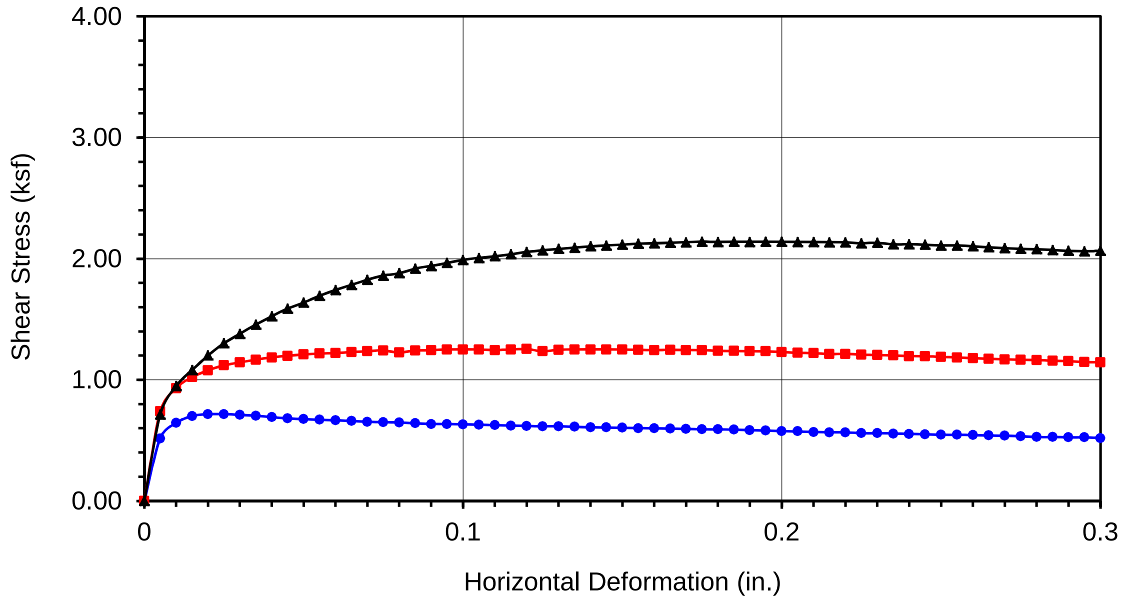
Leighton

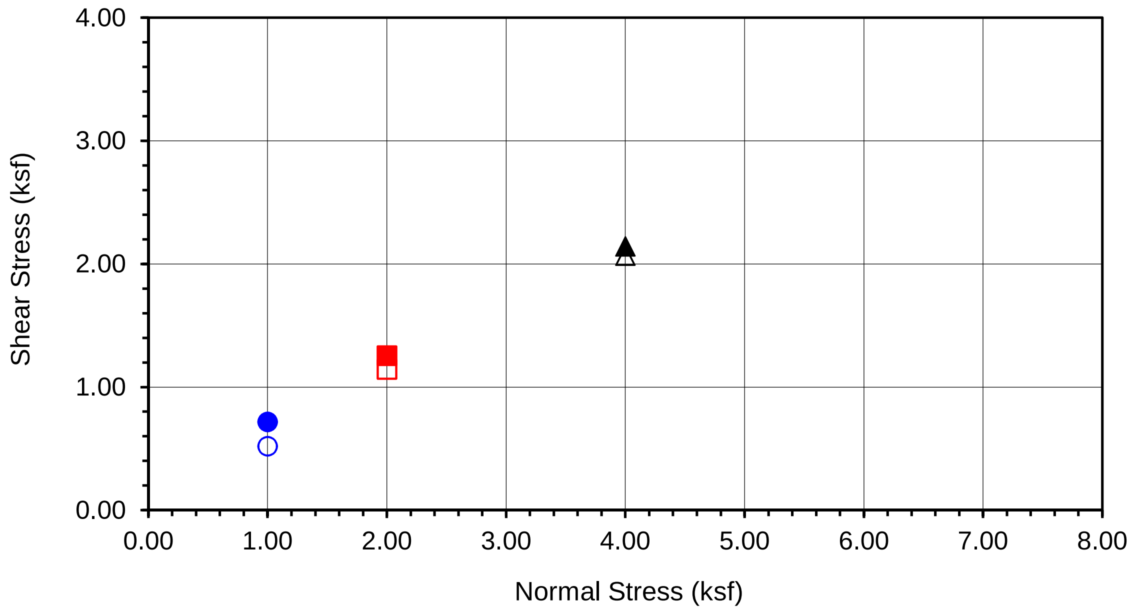
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: City of Salinas Public Safety Center
Tested By: G. Bathala
Date: 08/03/17
Project No.: 11693.001
Checked By: J. Ward
Date: 08/11/17
Boring No.: LB-1
Sample Type: 90% Remold
Sample No.: BB-1
Depth (ft.): 0-5
Soil Identification: Dark olive gray fat clay (CH)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	173.43	173.92	175.23
Weight of Ring(gm):	42.95	43.26	45.01
Before Shearing			
Weight of Wet Sample+Cont.(gm):	169.59	169.59	169.59
Weight of Dry Sample+Cont.(gm):	152.79	152.79	152.79
Weight of Container(gm):	59.19	59.19	59.19
Vertical Rdg.(in): Initial	0.0000	0.2820	0.2958
Vertical Rdg.(in): Final	0.0471	0.2600	0.2988
After Shearing			
Weight of Wet Sample+Cont.(gm):	203.88	182.95	180.63
Weight of Dry Sample+Cont.(gm):	168.55	149.38	149.19
Weight of Container(gm):	59.20	40.05	39.50
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**BB-1**
Depth (ft)	**0-5**
Sample Type: 90% Remold	
Soil Identification: Dark olive gray fat clay (CH)	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.717	■ 1.254	▲ 2.141
Shear Stress @ End of Test (ksf)	○ 0.519	□ 1.144	△ 2.065
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	17.95	17.95	17.95
Dry Density (pcf)	92.0	92.1	91.8
Saturation (%)	58.2	58.4	58.0
Soil Height Before Shearing (in.)	1.0471	1.0220	0.9970
Final Moisture Content (%)	32.3	30.7	28.7

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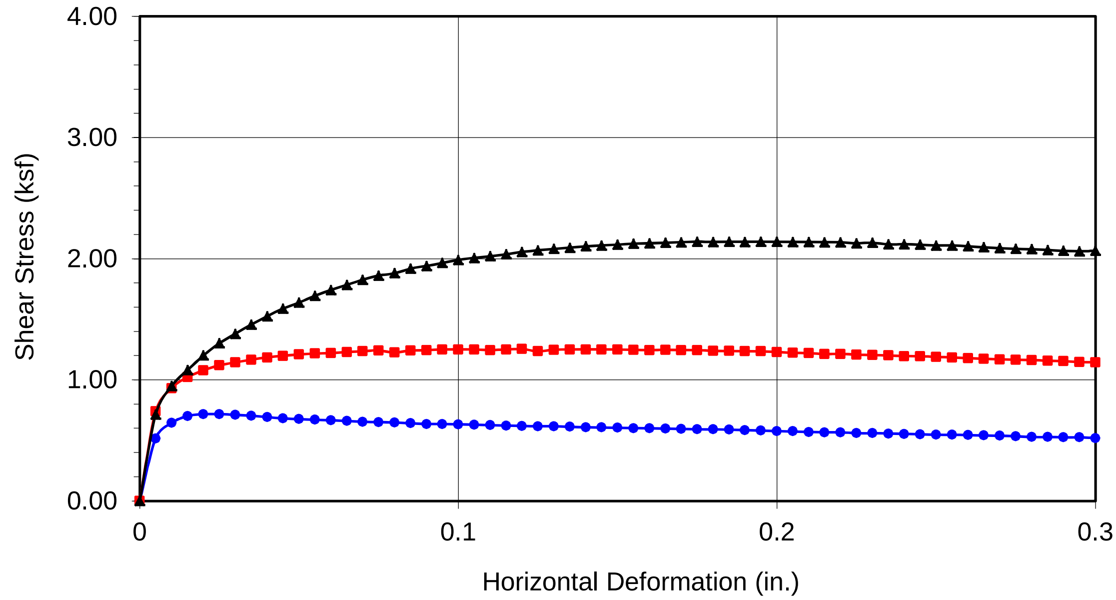
DIRECT SHEAR TEST RESULTS

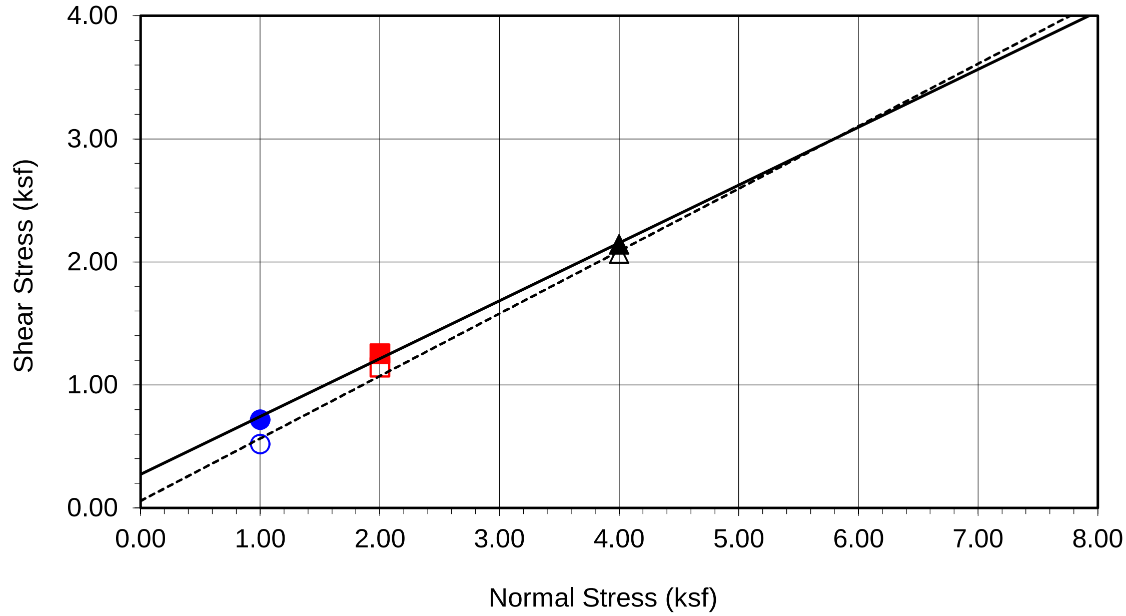
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17





Boring No.	**LB-1**
Sample No.	**BB-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold

Soil Identification:
Dark olive gray fat clay (CH)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	274	25
Ultimate	58	27

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.717	■ 1.254	▲ 2.141
Shear Stress @ End of Test (ksf)	○ 0.519	□ 1.144	△ 2.065
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	17.95	17.95	17.95
Dry Density (pcf)	92.0	92.1	91.8
Saturation (%)	58.2	58.4	58.0
Soil Height Before Shearing (in.)	1.0471	1.0220	0.9970
Final Moisture Content (%)	32.3	30.7	28.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

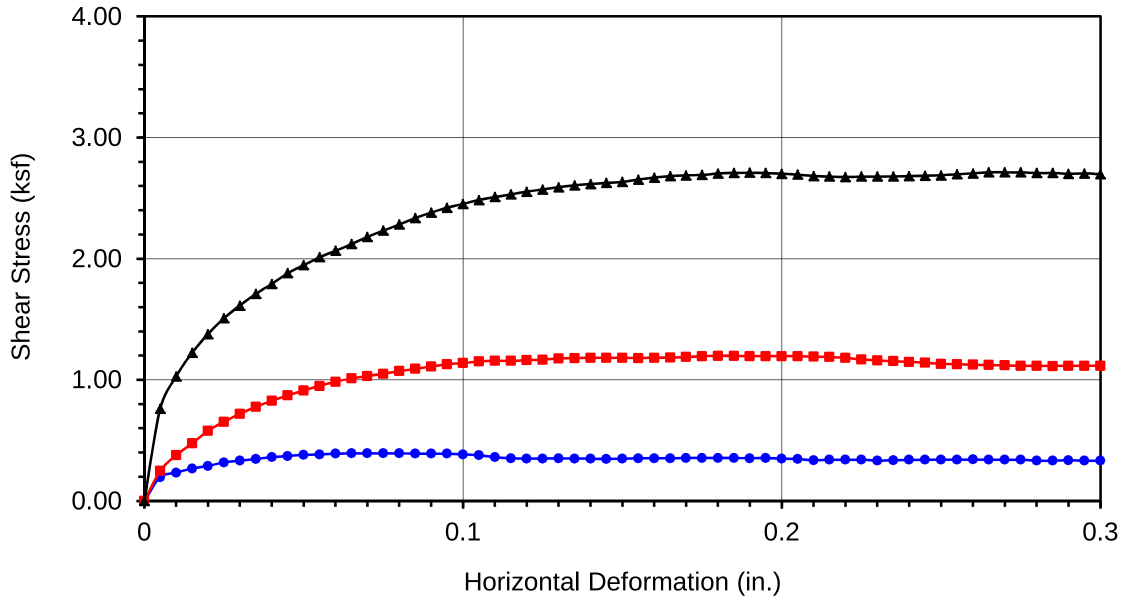
DIRECT SHEAR TEST

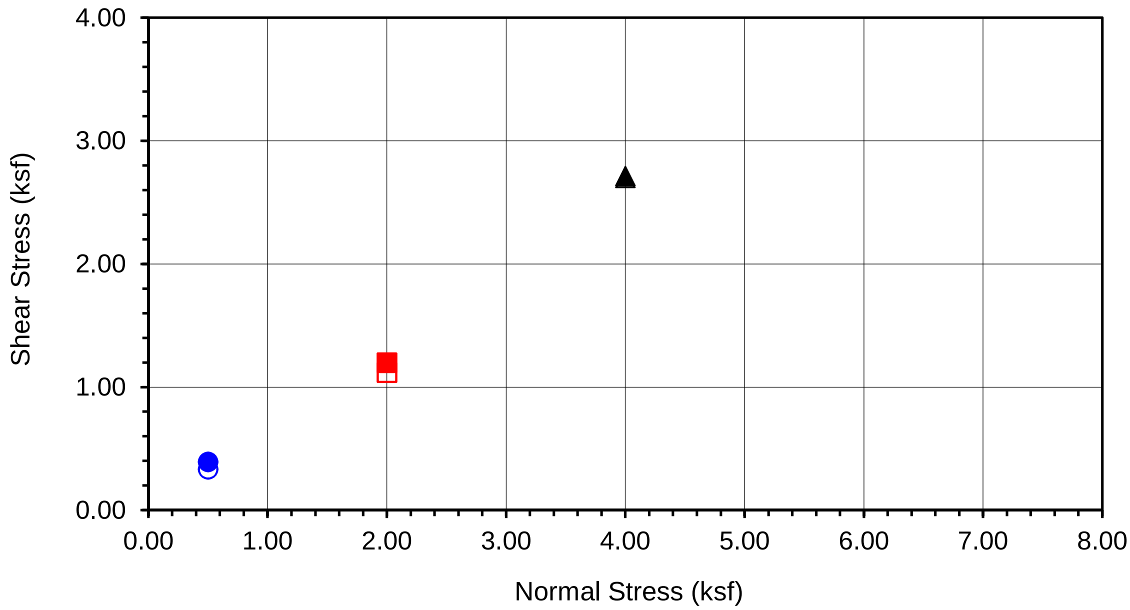
Consolidated Drained - ASTM D 3080

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	G. Bathala	Date:	08/09/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R1	Depth (ft.):	5.0		
Soil Identification:	Olive brown silty clay (CL-ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	186.94	187.26	189.80
Weight of Ring(gm):	43.95	43.06	44.80
Before Shearing			
Weight of Wet Sample+Cont.(gm):	201.31	201.31	201.31
Weight of Dry Sample+Cont.(gm):	171.85	171.85	171.85
Weight of Container(gm):	56.34	56.34	56.34
Vertical Rdg.(in): Initial	0.2713	0.0000	0.2483
Vertical Rdg.(in): Final	0.2796	-0.0393	0.3005
After Shearing			
Weight of Wet Sample+Cont.(gm):	195.22	193.16	208.59
Weight of Dry Sample+Cont.(gm):	165.50	164.49	180.94
Weight of Container(gm):	53.80	54.26	70.38
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**R1**
Depth (ft)	**5**

Sample Type:

Ring

Soil Identification:
Olive brown silty clay (CL-ML)

Normal Stress (kip/ft²)	0.500	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.393	■ 1.198	▲ 2.713
Shear Stress @ End of Test (ksf)	○ 0.333	□ 1.116	△ 2.697
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	25.50	25.50	25.50
Dry Density (pcf)	94.8	95.6	96.1
Saturation (%)	88.4	90.1	91.3
Soil Height Before Shearing (in.)	0.9917	0.9607	0.9478
Final Moisture Content (%)	26.6	26.0	25.0

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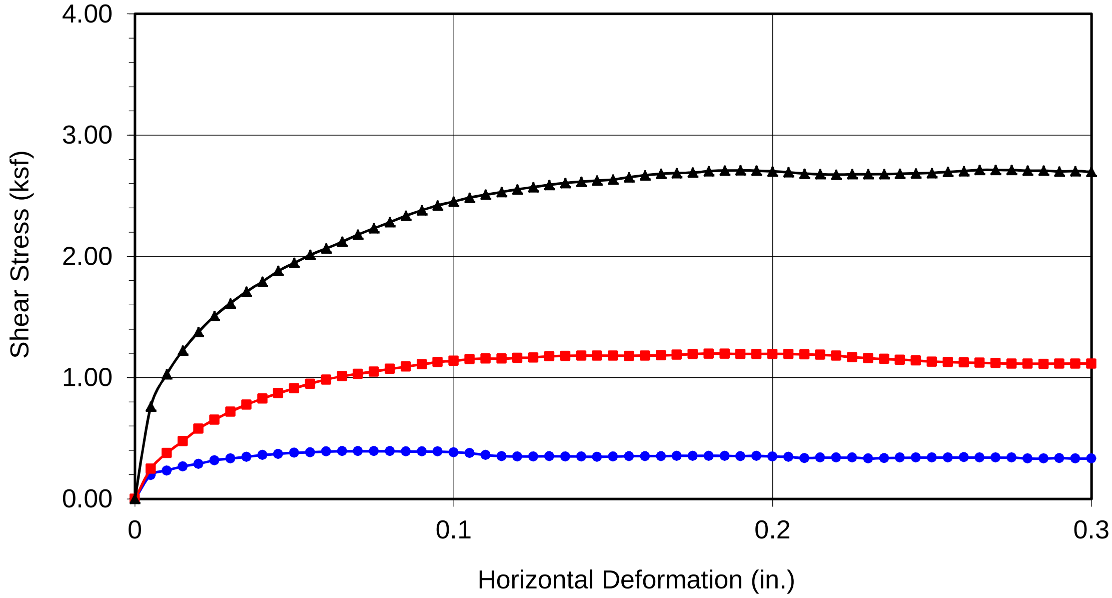
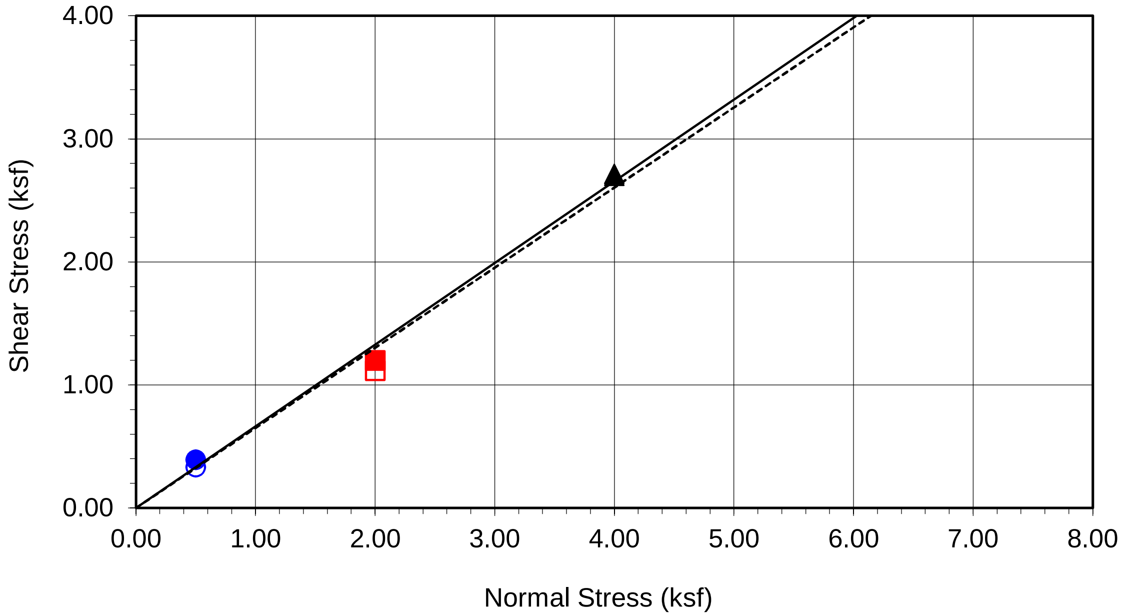
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17





Boring No.	**LB-1**
Sample No.	**R1**
Depth (ft)	**5**
Sample Type:	Ring
Soil Identification: Olive brown silty clay (CL-ML)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	34
Ultimate	0	33

Normal Stress (kip/ft²)	0.500	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.393	■ 1.198	▲ 2.713
Shear Stress @ End of Test (ksf)	○ 0.333	□ 1.116	△ 2.697
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	25.50	25.50	25.50
Dry Density (pcf)	94.8	95.6	96.1
Saturation (%)	88.4	90.1	91.3
Soil Height Before Shearing (in.)	0.9917	0.9607	0.9478
Final Moisture Content (%)	26.6	26.0	25.0

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

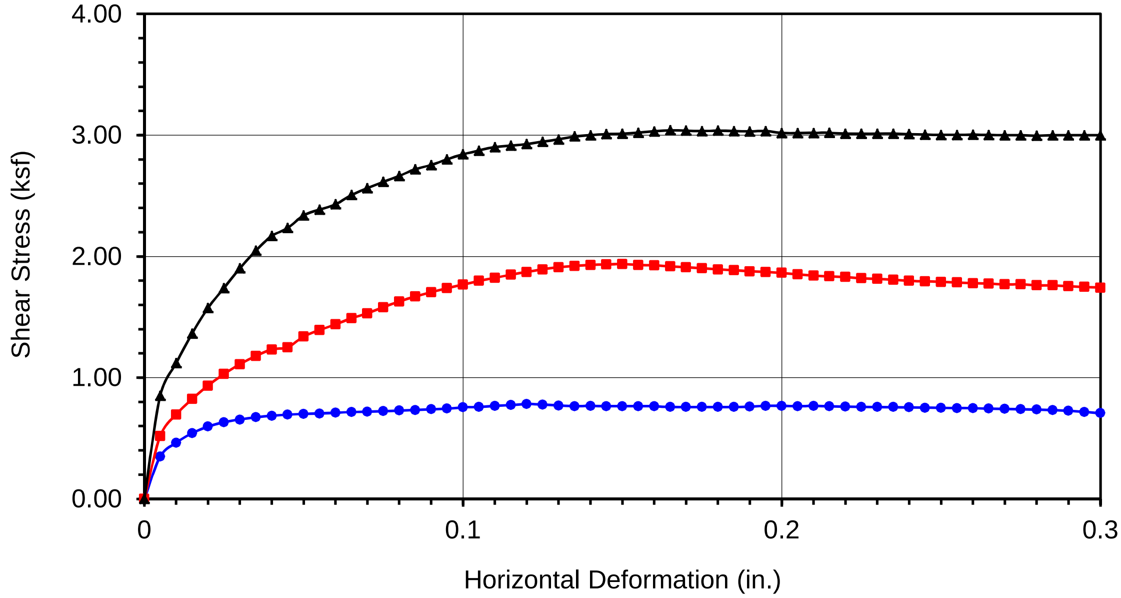
Leighton

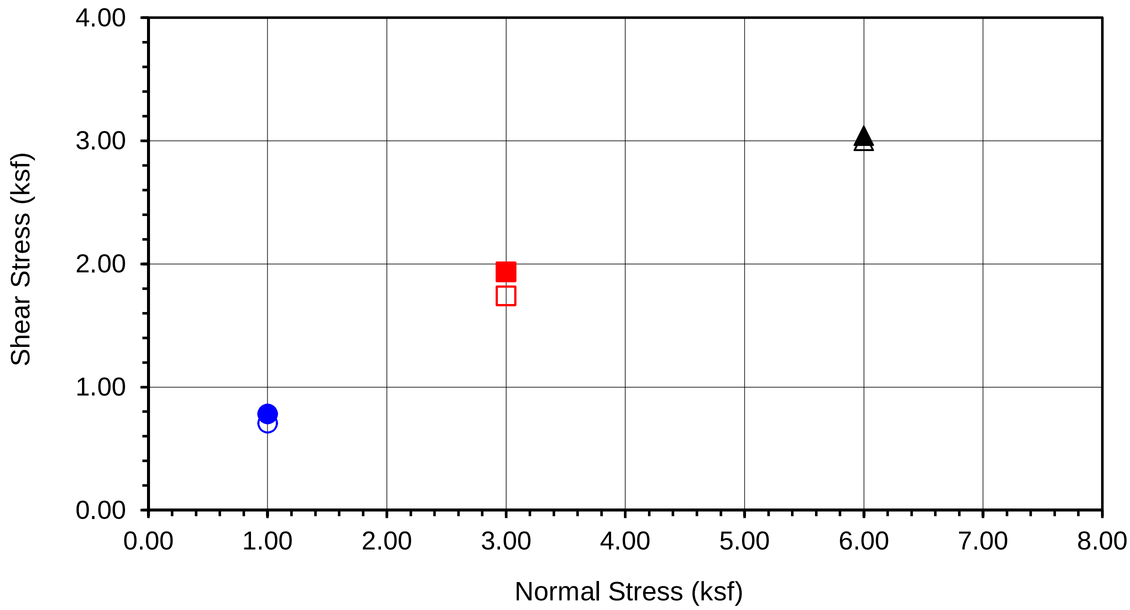
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	City of Salinas Public Safety Center	Tested By:	G. Bathala	Date:	08/09/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Sample Type:	Ring		
Sample No.:	R2	Depth (ft.):	10.0		
Soil Identification:	Dark olive gray silty clay (CL-ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	177.14	179.29	193.47
Weight of Ring(gm):	45.67	45.68	59.70
Before Shearing			
Weight of Wet Sample+Cont.(gm):	146.44	146.44	146.44
Weight of Dry Sample+Cont.(gm):	133.44	133.44	133.44
Weight of Container(gm):	67.95	67.95	67.95
Vertical Rdg.(in): Initial	0.2509	0.2774	0.0000
Vertical Rdg.(in): Final	0.2689	0.3162	-0.0583
After Shearing			
Weight of Wet Sample+Cont.(gm):	193.31	186.65	190.02
Weight of Dry Sample+Cont.(gm):	162.30	156.61	161.98
Weight of Container(gm):	58.21	52.04	57.53
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**R2**
Depth (ft)	**10**

Sample Type:

Ring

Soil Identification:

Dark olive gray silty clay (CL-ML)

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.783	■ 1.937	▲ 3.040
Shear Stress @ End of Test (ksf)	○ 0.707	□ 1.742	△ 2.999
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	19.85	19.85	19.85
Dry Density (pcf)	91.2	92.7	92.8
Saturation (%)	63.2	65.5	65.7
Soil Height Before Shearing (in.)	0.9820	0.9612	0.9417
Final Moisture Content (%)	29.8	28.7	26.8

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DIRECT SHEAR TEST RESULTS

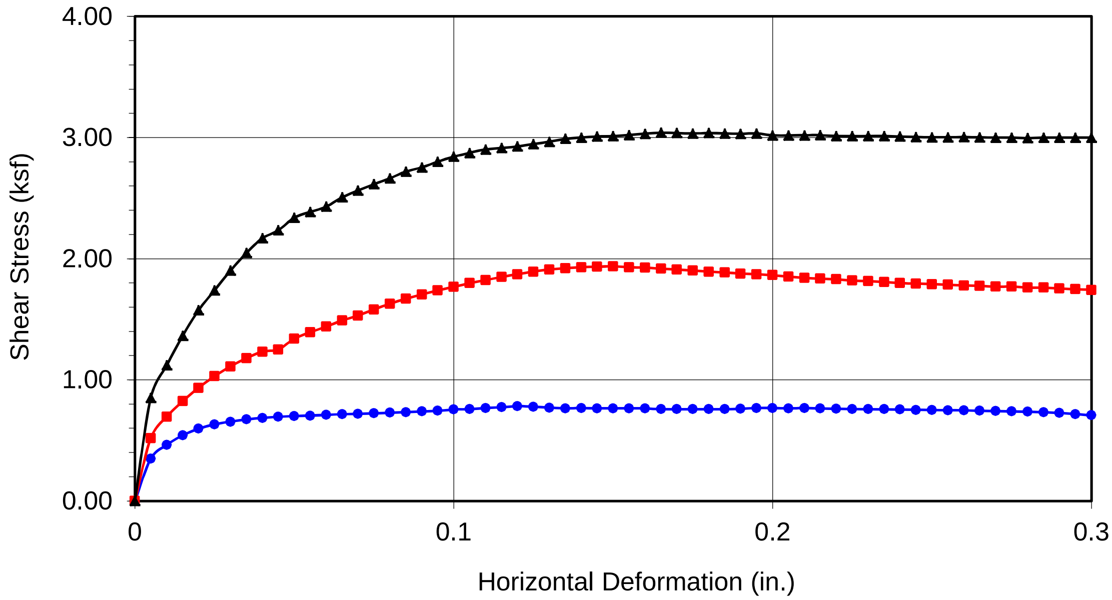
Consolidated Drained - ASTM D 3080

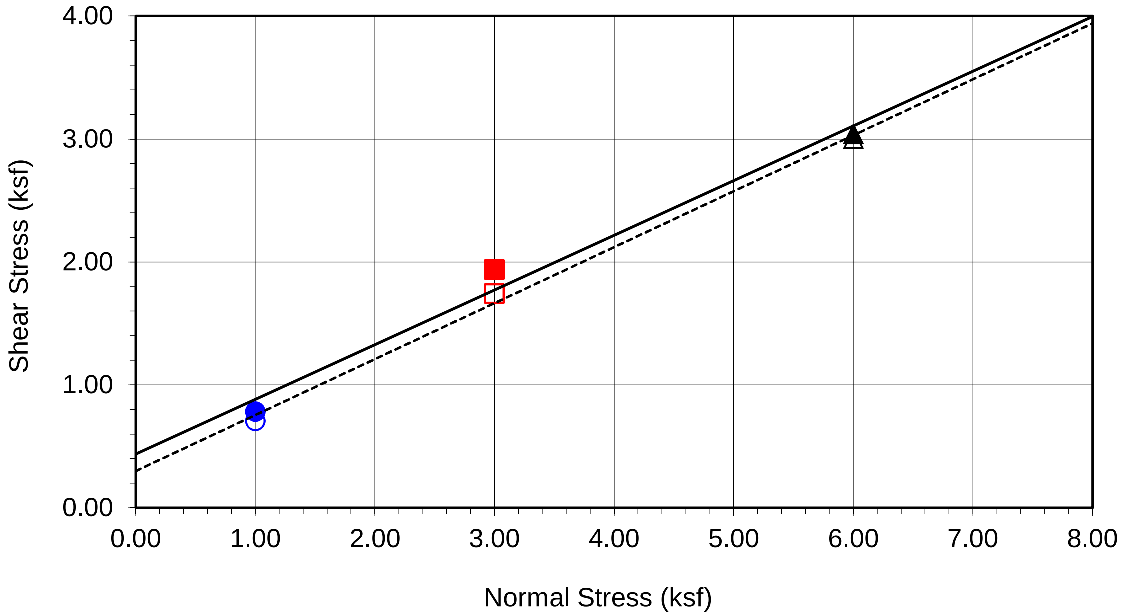
Project No.: 11693.001

City of Salinas Public Safety Center

08-17

DS LB-1, R2 @ 10





Boring No.	**LB-1**
Sample No.	**R2**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Dark olive gray silty clay (CL-ML)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	437	24
Ultimate	298	24

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.783	■ 1.937	▲ 3.040
Shear Stress @ End of Test (ksf)	○ 0.707	□ 1.742	△ 2.999
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	19.85	19.85	19.85
Dry Density (pcf)	91.2	92.7	92.8
Saturation (%)	63.2	65.5	65.7
Soil Height Before Shearing (in.)	0.9820	0.9612	0.9417
Final Moisture Content (%)	29.8	28.7	26.8

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

DS LB-1, R2 @ 10

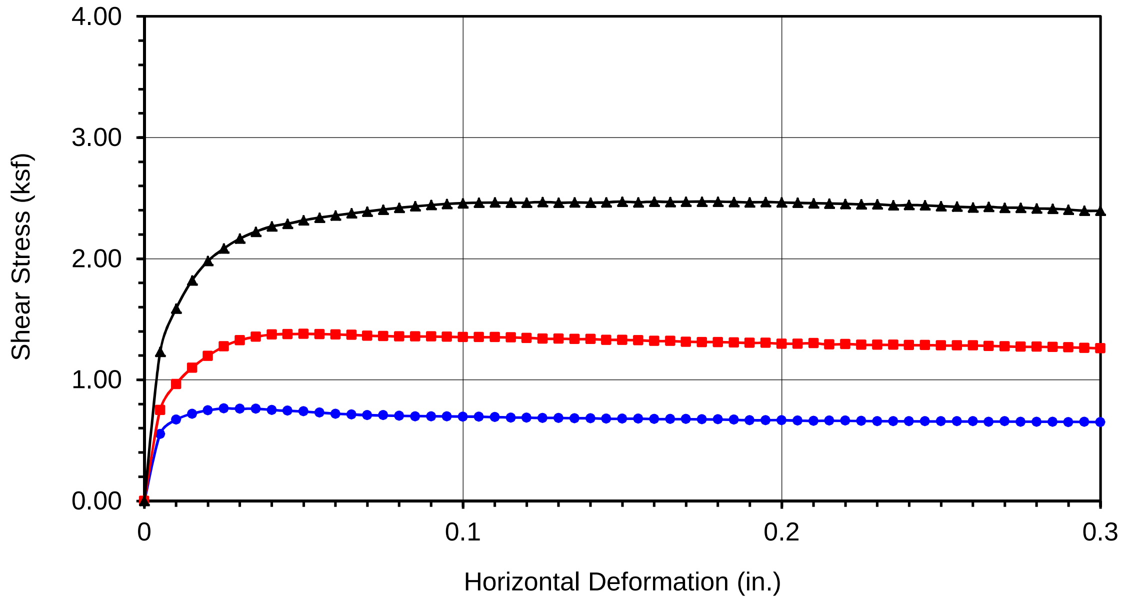
Leighton

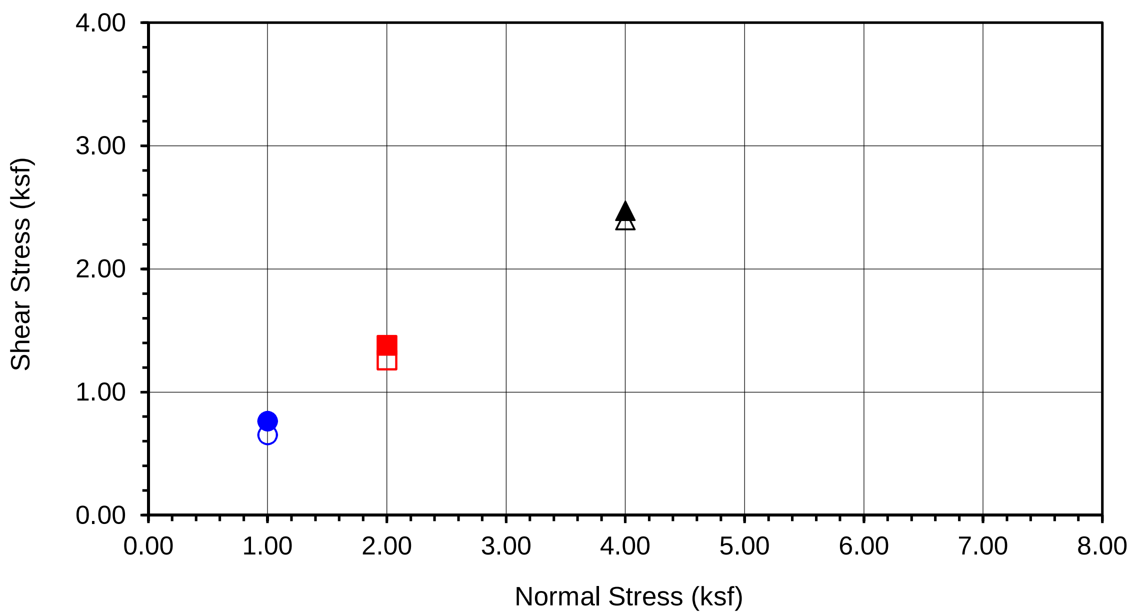
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	City of Salinas Public Safety Center	Tested By:	G. Bathala	Date:	08/03/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-2	Sample Type:	90% Remold		
Sample No.:	BB-1	Depth (ft.):	0-5		
Soil Identification:	Olive brown lean clay (CL)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	186.02	184.83	186.42
Weight of Ring(gm):	45.02	43.26	44.93
Before Shearing			
Weight of Wet Sample+Cont.(gm):	179.89	179.89	179.89
Weight of Dry Sample+Cont.(gm):	165.94	165.94	165.94
Weight of Container(gm):	68.49	68.49	68.49
Vertical Rdg.(in): Initial	0.0000	0.2804	0.0000
Vertical Rdg.(in): Final	0.0179	0.2764	-0.0113
After Shearing			
Weight of Wet Sample+Cont.(gm):	217.16	209.21	217.58
Weight of Dry Sample+Cont.(gm):	186.63	179.91	189.98
Weight of Container(gm):	64.61	57.88	67.37
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**BB-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:
Olive brown lean clay (CL)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.764	■ 1.380	▲ 2.471
Shear Stress @ End of Test (ksf)	○ 0.651	□ 1.261	△ 2.396
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	14.32	14.32	14.32
Dry Density (pcf)	102.6	103.0	102.9
Saturation (%)	60.1	60.7	60.6
Soil Height Before Shearing (in.)	1.0179	1.0040	0.9887
Final Moisture Content (%)	25.0	24.0	22.5

Leighton

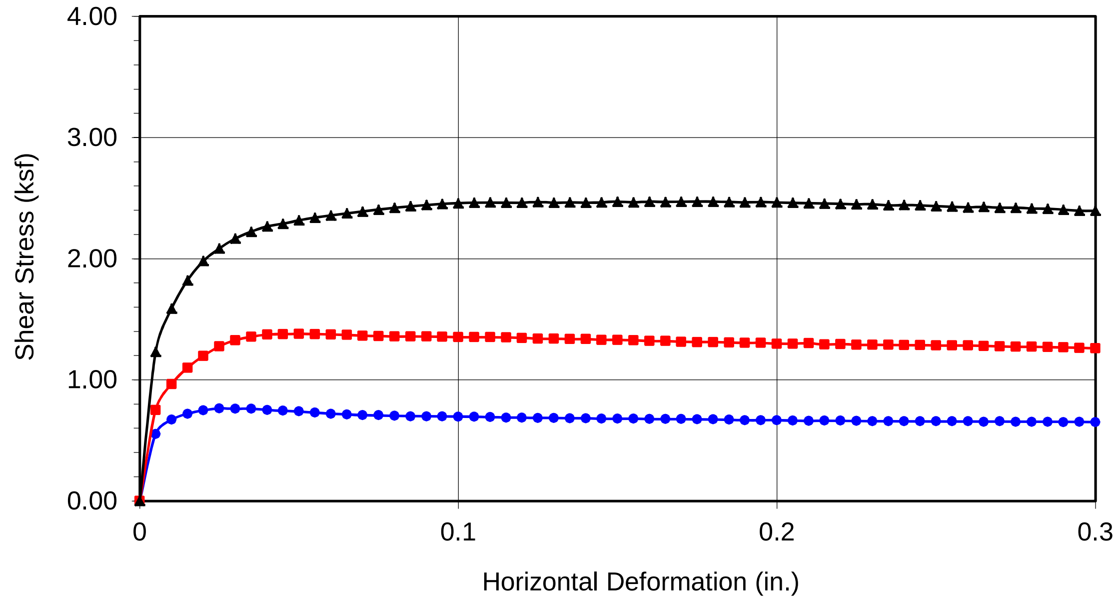
DIRECT SHEAR TEST RESULTS

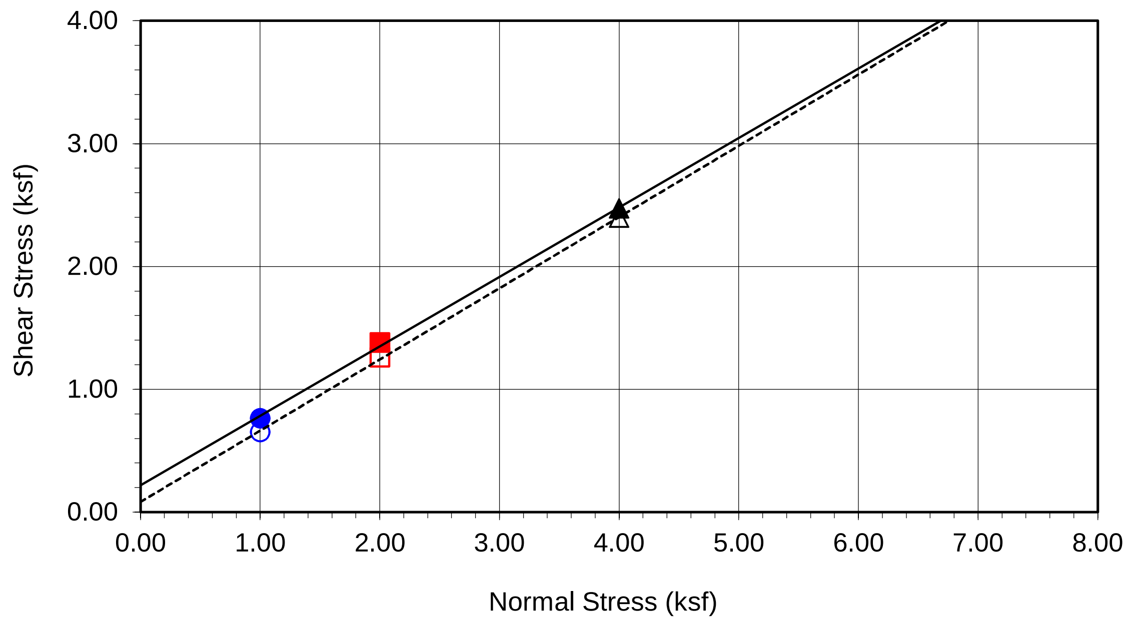
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17





Boring No.	**LB-2**
Sample No.	**BB-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold

Soil Identification:
Olive brown lean clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	219	29
Ultimate	84	30

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.764	■ 1.380	▲ 2.471
Shear Stress @ End of Test (ksf)	○ 0.651	□ 1.261	△ 2.396
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	14.32	14.32	14.32
Dry Density (pcf)	102.6	103.0	102.9
Saturation (%)	60.1	60.7	60.6
Soil Height Before Shearing (in.)	1.0179	1.0040	0.9887
Final Moisture Content (%)	25.0	24.0	22.5

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11693.001

City of Salinas Public Safety Center

08-17

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	O. Figueroa	Date:	08/01/17
Project No.:	11693.001	Input By:	J. Ward	Date:	08/02/17
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Dark olive gray fat clay (CH)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3611	3675	3699			
Weight of Mold (g)	1857	1857	1857			
Net Weight of Soil (g)	1754	1818	1842			
Wet Weight of Soil + Cont. (g)	395.1	403.1	409.7			
Dry Weight of Soil + Cont. (g)	348.9	347.8	345.9			
Weight of Container (g)	38.7	39.5	39.1			
Moisture Content (%)	14.89	17.94	20.80			
Wet Density (pcf)	116.1	120.4	121.9			
Dry Density (pcf)	101.1	102.1	101.0			

Maximum Dry Density (pcf) 102.0 **Optimum Moisture Content (%)** 18.0

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

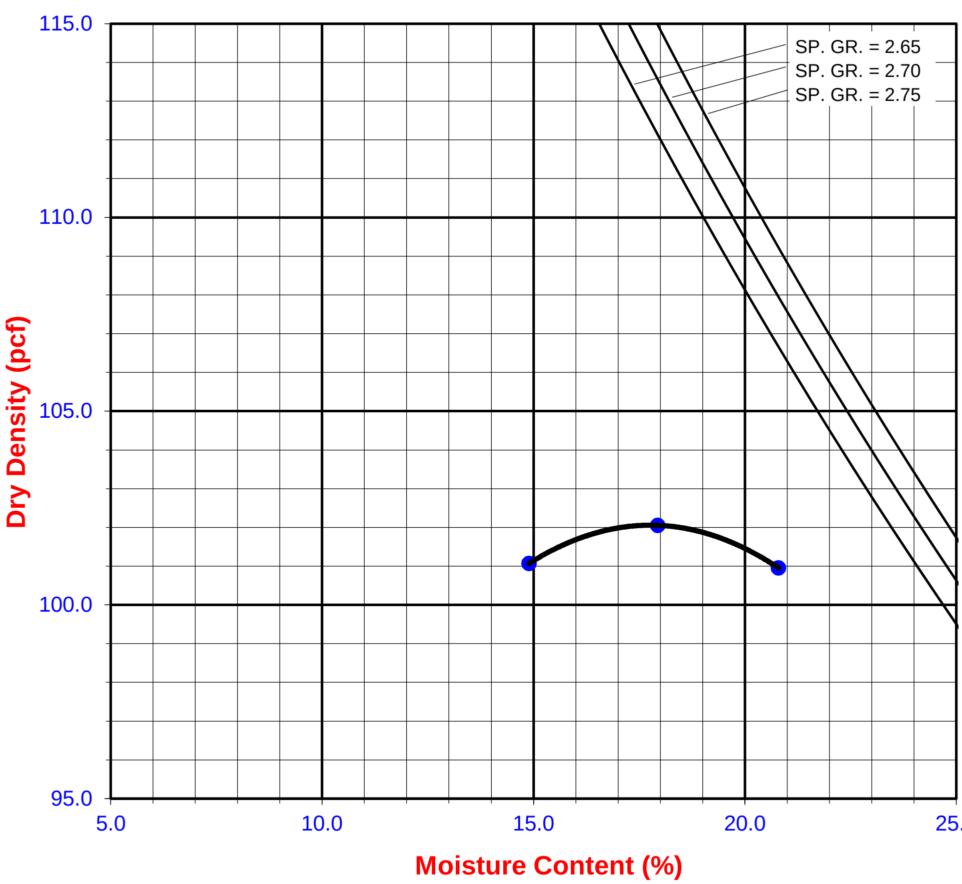
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	O. Figueroa	Date:	08/01/17
Project No.:	11693.001	Input By:	J. Ward	Date:	08/02/17
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Olive brown lean clay (CL)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3698	3774	3837	3817		
Weight of Mold (g)	1857	1857	1857	1857		
Net Weight of Soil (g)	1841	1917	1980	1960		
Wet Weight of Soil + Cont. (g)	390.0	382.4	406.1	430.8		
Dry Weight of Soil + Cont. (g)	358.0	344.3	357.5	371.9		
Weight of Container (g)	39.7	39.7	38.4	38.9		
Moisture Content (%)	10.05	12.51	15.23	17.69		
Wet Density (pcf)	121.9	126.9	131.1	129.8		
Dry Density (pcf)	110.7	112.8	113.8	110.3		

Maximum Dry Density (pcf) 114.0 **Optimum Moisture Content (%)** 14.5

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

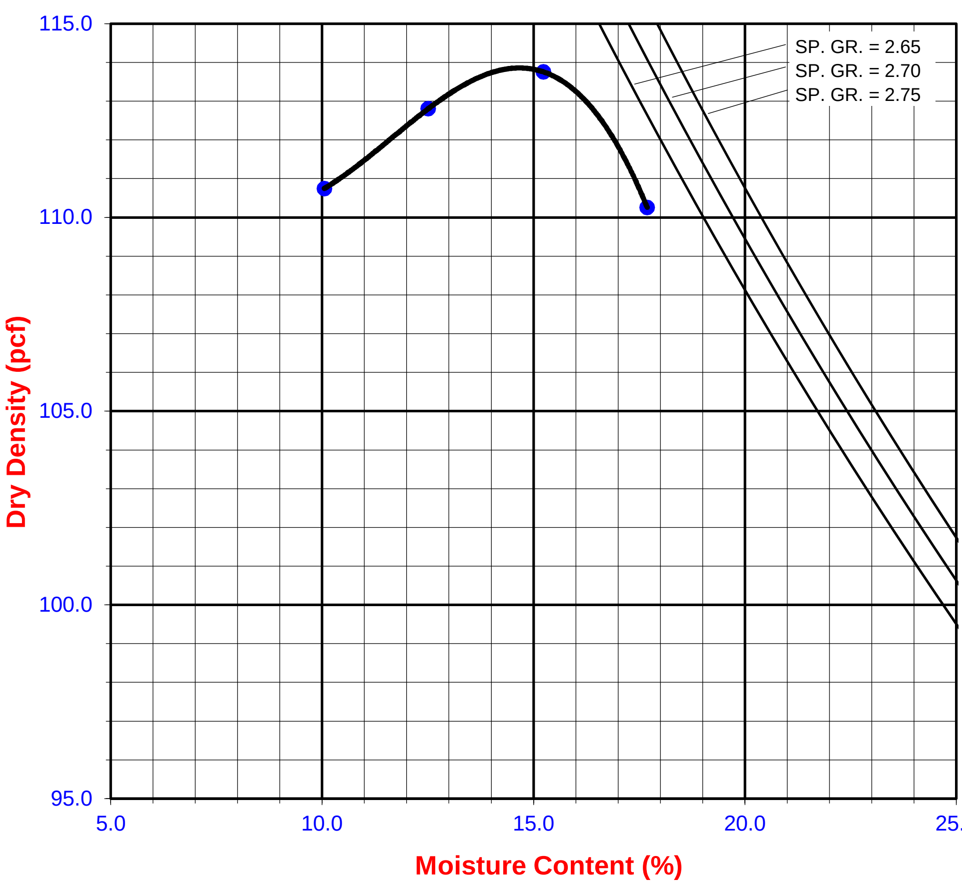
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	City of Salinas Public Safety Center	Tested By:	A. Santos	Date:	08/03/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Dark olive gray fat clay (CH)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.1005
Wt. Comp. Soil + Mold	(g)	555.00	409.20
Wt. of Mold	(g)	202.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	717.50	611.50
Dry Wt. of Soil + Cont.	(g)	626.70	510.33
Wt. of Container	(g)	0.00	202.30
Moisture Content	(%)	14.49	32.84
Wet Density	(pcf)	106.4	112.2
Dry Density	(pcf)	92.9	84.4
Void Ratio		0.814	0.997
Total Porosity		0.449	0.499
Pore Volume	(cc)	92.9	113.7
Degree of Saturation (%) [S_{meas}]		**48.0**	89.0

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
08/03/17	14:00	1.0	0	0.2155
08/03/17	14:10	1.0	10	0.2150
Add Distilled Water to the Specimen				
08/03/17	15:00	1.0	50	0.2210
08/04/17	7:30	1.0	1040	0.3160
08/04/17	8:45	1.0	1115	0.3160

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**101**

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	City of Salinas Public Safety Center	Tested By:	S. Felter	Date:	07/31/17
Project No.:	11693.001	Checked By:	J. Ward	Date:	08/11/17
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Olive brown lean clay (CL)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0780
Wt. Comp. Soil + Mold	(g)	531.20	417.02
Wt. of Mold	(g)	164.60	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	722.20	581.62
Dry Wt. of Soil + Cont.	(g)	632.40	485.62
Wt. of Container	(g)	0.00	164.60
Moisture Content	(%)	14.20	29.90
Wet Density	(pcf)	110.6	116.7
Dry Density	(pcf)	96.8	89.8
Void Ratio		0.741	0.877
Total Porosity		0.426	0.467
Pore Volume	(cc)	88.1	104.2
Degree of Saturation (%) [S_{meas}]		**51.7**	92.1

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
07/31/17	8:35	1.0	0	0.1580
07/31/17	8:45	1.0	10	0.1580
Add Distilled Water to the Specimen				
07/31/17	11:32	1.0	167	0.2290
08/01/17	6:30	1.0	1305	0.2360
08/01/17	7:40	1.0	1375	0.2360

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**78**

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: City of Salinas Public Safety Center   Tested By : G. Berdy   Date: 07/28/17

Project No. : 11693.001   Data Input By: J. Ward   Date: 08/11/17

Boring No.	LB-1	LB-2		
Sample No.	BB-1	BB-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Dark olive gray CH	Olive brown CL		
Wet Weight of Soil + Container (g)	146.12	191.89		
Dry Weight of Soil + Container (g)	141.95	185.86		
Weight of Container (g)	72.02	58.69		
Moisture Content (%)	5.96	4.74		
Weight of Soaked Soil (g)	100.41	100.43		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	0	315		
Crucible No.	26	6		
Furnace Temperature (°C)	860	860		
Time In / Time Out	8:30/9:15	8:30/9:15		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	20.9376	23.3558		
Wt. of Crucible (g)	20.9350	23.3525		
Wt. of Residue (g) (A)	0.0026	0.0033		
PPM of Sulfate (A) x 41150	106.99	135.79		
PPM of Sulfate, Dry Weight Basis	**114**	**143**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.6	0.9		
PPM of Chloride (C -0.2) * 100 * 30 / B	80	140		
PPM of Chloride, Dry Wt. Basis	**85**	**147**		

pH TEST, DOT California Test 643

pH Value	**6.94**	**8.15**		
Temperature °C	20.5	20.5		

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

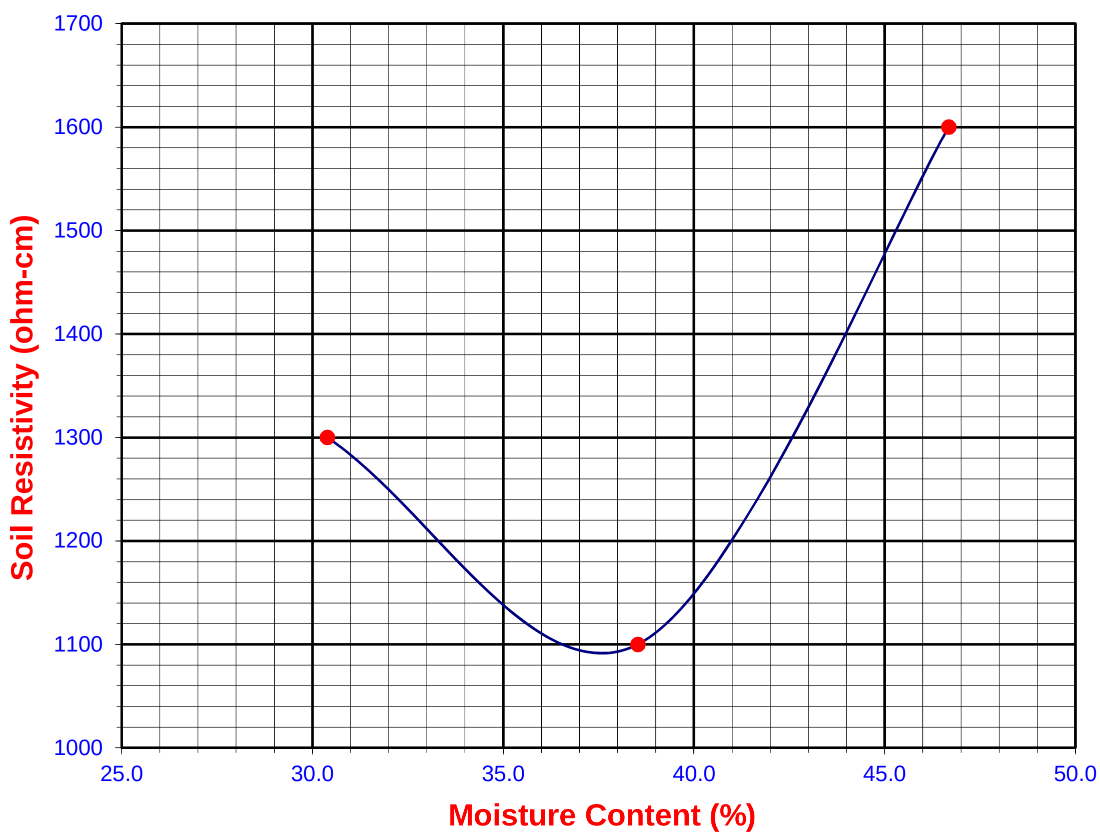
Project Name:	City of Salinas Public Safety Center	Tested By :	G. Berdy	Date:	07/31/17
Project No. :	11693.001	Data Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-1	Depth (ft.) :	0-5		
Sample No. :	BB-1				
Soil Identification:*	Dark olive gray CH				

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	30.39	1300	1300
2	40	38.53	1100	1100
3	50	46.68	1600	1600
4				
5				

Moisture Content (%) (MCi)	5.96
Wet Wt. of Soil + Cont. (g)	146.12
Dry Wt. of Soil + Cont. (g)	141.95
Wt. of Container (g)	72.02
Container No.	
Initial Soil Wt. (g) (Wt)	130.13
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1090	**37.6**	**114**	**85**	**6.94**	**20.5**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	City of Salinas Public Safety Center	Tested By :	G. Berdy	Date:	07/28/17
Project No. :	11693.001	Data Input By:	J. Ward	Date:	08/11/17
Boring No.:	LB-2	Depth (ft.) :	0-5		
Sample No. :	BB-1				

Soil Identification:* Olive brown CL

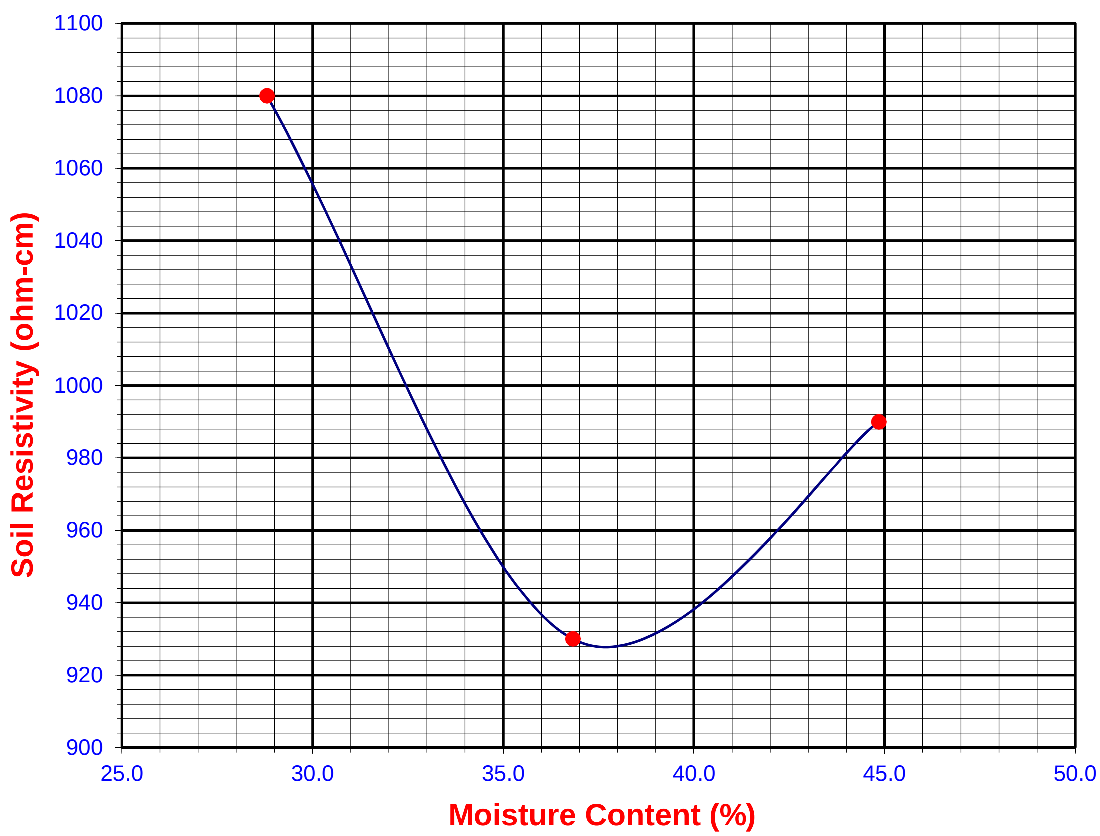
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	28.81	1080	1080
2	40	36.83	930	930
3	50	44.85	990	990
4				
5				

Moisture Content (%) (MCi)	4.74
Wet Wt. of Soil + Cont. (g)	191.89
Dry Wt. of Soil + Cont. (g)	185.86
Wt. of Container (g)	58.69
Container No.	
Initial Soil Wt. (g) (Wt)	130.58
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
928	**37.7**	**143**	**147**	**8.15**	**20.5**



APPENDIX C

Percolation Test Results

Boring Percolation Test Data Sheet

Project Number:	11693.001	**Test Hole Number:**	LB-3
Project Name:	Salinas Police HQ	**Date Excavated:**	7/13/2017
Earth Description:	Alluvium	**Date Tested:**	7/14/2017
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	JMP	**Diameter of boring (in):**	8
Time Interval Standard		**Diameter of casing (in):**	2
Start Time for Pre-Soak:	11:10	**Length of slotted of casing (ft):**	5
Start Time for Standard:	12:13	**Depth to Initial Water Depth (ft):**	5
Standard Time Interval Between Readings, mins:	30	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
1	7:00	30	4.98	60.2	0.0	#DIV/0!	0.00
	7:30		4.98	60.2			
2	7:30	30	4.98	60.2	0.4	83.33	0.01
	8:00		5.01	59.9			
3	8:00	30	5.01	59.9	0.2	125.00	0.01
	8:30		5.03	59.6			
4	8:30	30	5.03	59.6	0.4	83.33	0.01
	9:00		5.06	59.3			
5	9:00	30	5.06	59.3	0.5	62.50	0.01
	9:30		5.10	58.8			
6	9:30	30	5.00	60.0	0.5	62.50	0.01
	10:00		5.04	59.5			
7	10:00	30	5.04	59.5	0.6	50.00	0.02
	10:30		5.09	58.9			
8	10:30	30	5.00	60.0	0.5	62.50	0.01
	11:00		5.04	59.5			
9	11:00	30	5.04	59.5	0.5	62.50	0.01
	11:30		5.08	59.0			
10	11:30	30	5.00	60.0	0.5	62.50	0.01
	12:00		5.04	59.5			
11	12:00	30	5.04	59.5	0.6	50.00	0.02
	12:30		5.09	58.9			
12	12:30	30	5.00	60.0	0.5	62.50	0.01
	13:00		5.04	59.5			

Infiltration Rate (I) = Flow Volume/Flow Area/Δt

Infiltration Rate, I (Last Reading) = $\boxed{\mathbf{0.01}}$ **in./hr.**

APPENDIX D1

Seismicity Data

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*************************
*                       *
*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
*                       *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 11693.001

DATE: 08-10-2017

JOB NAME: 11693.001

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 4.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 36.6736
SITE LONGITUDE: 121.6450

SEARCH DATES:
START DATE: 1800
END DATE: 2016

SEARCH RADIUS:
62.0 mi
99.8 km

ATTENUATION RELATION: 3) Boore et al. (1997) Horiz. - NEHRP D (250)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	36.6700	121.6700	12/28/1924	421 0.0	0.0	4.00	0.145	VIII	1.4(2.3)
DMG	36.6700	121.6700	10/09/1931	1950 0.0	0.0	4.00	0.145	VIII	1.4(2.3)
MGI	36.6700	121.6700	08/06/1916	2056 0.0	0.0	4.00	0.145	VIII	1.4(2.3)
DMG	36.7200	121.6200	05/26/1959	1558 1.0	0.0	4.60	0.161	VIII	3.5(5.6)
DMG	36.7000	121.7000	03/05/1937	1247 0.0	0.0	4.50	0.152	VIII	3.5(5.7)
DMG	36.6800	121.5600	09/20/1939	24529.0	0.0	4.00	0.102	VII	4.7(7.6)
MGI	36.5800	121.6700	10/05/1920	19 4 0.0	0.0	4.00	0.085	VII	6.6(10.6)
MGI	36.5800	121.6700	03/09/1924	1133 0.0	0.0	4.00	0.085	VII	6.6(10.6)
MGI	36.5800	121.6700	08/25/1923	1121 0.0	0.0	4.00	0.085	VII	6.6(10.6)
MGI	36.5800	121.6700	06/16/1920	12 9 0.0	0.0	4.00	0.085	VII	6.6(10.6)
DMG	36.7600	121.5800	08/31/1963	163114.2	0.0	4.20	0.091	VII	7.0(11.2)
GSB	36.7400	121.5070	03/26/1984	075839.7	5.0	4.00	0.070	VI	8.9(14.3)
BRK	36.7900	121.5600	09/28/1981	73439.0	0.0	4.00	0.068	VI	9.3(15.0)
DMG	36.6000	121.5000	10/27/1937	1553 0.0	0.0	4.50	0.087	VII	9.5(15.3)
BRK	36.7700	121.5200	04/13/1980	61556.0	0.0	4.70	0.096	VII	9.6(15.4)
DMG	36.6000	121.8000	04/25/1908	1133 0.0	0.0	4.50	0.084	VII	10.0(16.1)
GSB	36.7710	121.4990	01/12/2011	085103.6	8.0	4.50	0.081	VII	10.5(16.9)
DMG	36.7930	121.5260	09/23/1972	15 748.2	6.8	4.11	0.066	VI	10.5(17.0)
USG	36.8060	121.5440	08/02/1979	214316.4	5.0	4.00	0.062	VI	10.7(17.2)
PAS	36.6840	121.4480	10/22/1984	4 628.6	6.0	4.00	0.061	VI	10.9(17.6)
GSG	36.8058	121.5360	11/20/2014	062649.3	6.6	4.23	0.069	VI	10.9(17.6)
DMG	36.8020	121.5290	10/03/1972	630 2.3	5.0	4.76	0.091	VII	10.9(17.6)
GSB	36.8100	121.5410	05/28/1998	203133.0	8.0	4.20	0.067	VI	11.0(17.8)
GSB	36.8010	121.5240	11/29/1995	231233.6	7.0	4.40	0.074	VII	11.0(17.8)
GSB	36.7940	121.5120	02/11/2001	230406.6	6.0	4.10	0.063	VI	11.1(17.9)
DMG	36.8140	121.5450	10/03/1972	111013.6	5.1	4.05	0.061	VI	11.2(18.0)
GSB	36.8010	121.5200	08/26/2001	043232.9	6.0	4.00	0.060	VI	11.2(18.0)
GSB	36.8110	121.5330	08/26/2001	050302.4	6.0	4.00	0.059	VI	11.3(18.2)
GSB	36.8050	121.5200	03/16/2004	063832.6	6.0	4.30	0.069	VI	11.4(18.4)
GSB	36.7550	121.4640	08/12/1998	141025.1	8.0	5.40	0.123	VII	11.5(18.5)
BRK	36.8400	121.6300	01/27/1981	221053.0	0.0	4.10	0.062	VI	11.5(18.5)
T-A	36.8300	121.5700	10/18/1800	0 0 0.0	0.0	7.00	0.284	IX	11.6(18.6)
DMG	36.8000	121.5000	11/13/1892	1245 0.0	0.0	5.60	0.133	VIII	11.8(19.1)
DMG	36.8000	121.5000	01/09/1930	956 0.0	0.0	4.00	0.057	VI	11.8(19.1)
DMG	36.8000	121.5000	01/09/1930	816 0.0	0.0	4.50	0.075	VII	11.8(19.1)
DMG	36.8000	121.5000	01/09/1930	8 6 0.0	0.0	4.50	0.075	VII	11.8(19.1)
DMG	36.5000	121.6700	12/04/1931	053 0.0	0.0	4.00	0.057	VI	12.1(19.4)
DMG	36.8200	121.5200	11/23/1950	135824.0	0.0	4.10	0.059	VI	12.2(19.7)
DMG	36.8500	121.6800	07/17/1939	925 0.0	0.0	4.50	0.073	VII	12.3(19.8)
GSB	36.8550	121.6380	08/05/1990	065213.6	7.0	4.00	0.055	VI	12.5(20.2)
DMG	36.8500	121.5700	03/28/1948	2236 0.0	0.0	4.00	0.054	VI	12.9(20.7)
DMG	36.8500	121.5700	03/28/1948	2238 3.0	0.0	4.60	0.074	VII	12.9(20.7)
DMG	36.8500	121.5700	03/28/1948	2245 0.0	0.0	4.50	0.070	VI	12.9(20.7)

DMG	36.8600	121.6500	05/07/1963	7 748.0	0.0	4.40	0.067	VI	12.9(20.7)
BRK	36.8600	121.6200	01/07/1981	114233.0	0.0	4.50	0.070	VI	12.9(20.8)
T-A	36.8300	121.5000	04/18/1865	2131 0.0	0.0	4.30	0.061	VI	13.4(21.6)
DMG	36.8700	121.6300	09/14/1963	194617.0	0.0	5.40	0.109	VII	13.6(21.9)
DMG	36.8700	121.6700	01/04/1961	03017.0	0.0	4.10	0.055	VI	13.6(21.9)
DMG	36.7000	121.4000	11/22/1909	1521 0.0	0.0	4.50	0.067	VI	13.7(22.0)
DMG	36.8700	121.6000	12/01/1956	141125.0	0.0	4.40	0.063	VI	13.8(22.2)
DMG	36.8000	121.4500	06/24/1939	13 2 0.0	0.0	5.50	0.113	VII	13.9(22.3)
DMG	36.8000	121.4500	03/02/1940	1327 0.0	0.0	4.00	0.051	VI	13.9(22.3)
DMG	36.7800	121.4300	01/20/1960	32553.0	0.0	5.00	0.086	VII	14.0(22.5)

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	36.8200	121.4700	05/25/1953	02330.0	0.0	4.00	0.051	VI	14.0(22.5)
DMG	36.7500	121.4100	11/15/1969	2058 3.6	0.0	4.20	0.056	VI	14.0(22.6)
GSB	36.8750	121.6070	04/18/1996	232152.0	4.0	4.40	0.063	VI	14.1(22.6)
DMG	36.7700	121.4200	07/07/1947	440 0.0	0.0	4.30	0.059	VI	14.1(22.7)
DMG	36.7700	121.4200	05/29/1946	1751 3.0	0.0	4.50	0.066	VI	14.1(22.7)
GSB	36.8800	121.6470	04/22/1990	020015.5	5.0	4.00	0.050	VI	14.2(22.9)
GSB	36.8800	121.6520	10/25/1989	130042.3	9.0	4.10	0.053	VI	14.3(22.9)
DMG	36.8500	121.7800	08/05/1946	4 845.0	0.0	4.10	0.053	VI	14.3(23.0)
GSB	36.8800	121.6220	07/02/2007	195853.2	8.0	4.30	0.059	VI	14.3(23.0)
DMG	36.5700	121.4200	06/22/1954	1250 4.0	0.0	4.30	0.058	VI	14.4(23.1)
DMG	36.5700	121.4200	06/22/1954	114929.0	0.0	4.40	0.062	VI	14.4(23.1)
DMG	36.5000	121.5000	09/07/1940	13 2 6.0	0.0	4.50	0.065	VI	14.4(23.2)
GSB	36.8830	121.6630	09/19/1991	090756.2	1.0	4.50	0.065	VI	14.5(23.3)
MGI	36.7500	121.4000	04/28/1948	730 0.0	0.0	4.30	0.058	VI	14.5(23.4)
MGI	36.8500	121.5000	08/16/1957	18 2 0.0	0.0	4.30	0.058	VI	14.6(23.5)
GSB	36.8840	121.6160	04/19/1996	005241.4	5.0	4.70	0.071	VI	14.6(23.5)
GSB	36.8870	121.6430	04/07/1990	200859.5	4.0	4.20	0.054	VI	14.7(23.7)
DMG	36.5800	121.4000	10/14/1938	1531 0.0	0.0	4.00	0.048	VI	15.0(24.2)
MGI	36.7800	121.4000	05/11/1948	1045 0.0	0.0	4.30	0.055	VI	15.4(24.8)
BRK	36.9000	121.6400	06/18/1980	45226.0	0.0	4.20	0.052	VI	15.6(25.2)
GSB	36.9000	121.6500	09/19/1991	090618.5	6.0	4.50	0.061	VI	15.6(25.2)
DMG	36.9000	121.6200	01/01/1949	11754.0	0.0	4.50	0.061	VI	15.7(25.2)
DMG	36.9000	121.6800	04/22/1954	185013.0	0.0	4.30	0.055	VI	15.7(25.3)
BRK	36.6600	121.3600	03/06/1980	11 5 9.0	0.0	4.00	0.046	VI	15.8(25.4)
MGI	36.9000	121.6000	10/11/1800	0 0 0.0	0.0	5.70	0.114	VII	15.8(25.5)
DMG	36.9000	121.6000	04/30/1900	224114.0	0.0	4.50	0.060	VI	15.8(25.5)
DMG	36.9000	121.6000	04/24/1890	1136 0.0	0.0	6.00	0.133	VIII	15.8(25.5)
DMG	36.9000	121.6000	03/30/1883	1545 0.0	0.0	5.60	0.108	VII	15.8(25.5)
DMG	36.9000	121.7000	04/30/1899	2241 0.0	0.0	5.60	0.107	VII	15.9(25.6)
DMG	36.6000	121.3700	04/28/1961	1 252.0	0.0	4.20	0.051	VI	16.1(25.8)
DMG	36.6300	121.3600	09/14/1965	9 924.2	0.0	4.00	0.046	VI	16.1(25.9)
DMG	36.8000	121.4000	04/02/1885	1525 0.0	0.0	5.40	0.096	VII	16.1(25.9)
MGI	36.8000	121.4000	10/18/1908	821 0.0	0.0	4.60	0.063	VI	16.1(25.9)
GSG	36.7530	121.3710	06/16/1995	042036.2	9.0	4.10	0.048	VI	16.1(25.9)
DMG	36.7900	121.3900	10/27/1969	105942.8	0.0	4.60	0.062	VI	16.2(26.1)
GSB	36.9100	121.6520	04/18/1990	133812.0	8.0	4.90	0.073	VII	16.3(26.3)
GSB	36.9100	121.6520	04/18/1990	133757.2	8.0	4.40	0.056	VI	16.3(26.3)
GSB	36.6770	121.3500	12/01/1989	123743.5	6.0	4.40	0.056	VI	16.3(26.3)
GSB	36.7780	121.3790	12/05/2000	225454.5	9.0	4.00	0.045	VI	16.4(26.4)

BRK	36.7400	121.3600	06/08/1981	3 9 5.0	0.0	4.30	0.053	VI	16.4(26.4)
MGI	36.8300	121.4200	10/25/1912	316 0.0	0.0	4.00	0.045	VI	16.5(26.5)
DMG	36.8300	121.4200	12/31/1910	1211 0.0	0.0	5.00	0.076	VII	16.5(26.5)
DMG	36.8300	121.4200	12/27/1915	723 0.0	0.0	4.00	0.045	VI	16.5(26.5)
MGI	36.8300	121.4200	10/17/1922	530 0.0	0.0	4.00	0.045	VI	16.5(26.5)
DMG	36.8300	121.4200	09/24/1917	2121 0.0	0.0	4.00	0.045	VI	16.5(26.5)
DMG	36.8300	121.4200	03/11/1911	2130 0.0	0.0	4.50	0.059	VI	16.5(26.5)
GSB	36.9120	121.6630	04/18/1990	145223.8	6.0	4.30	0.053	VI	16.5(26.5)
T-A	36.5800	121.9200	06/14/1891	027 0.0	0.0	4.30	0.053	VI	16.5(26.6)
T-A	36.5800	121.9200	08/13/1887	1117 0.0	0.0	4.30	0.053	VI	16.5(26.6)
MGI	36.5800	121.9200	02/28/1915	1044 0.0	0.0	4.00	0.045	VI	16.5(26.6)
DMG	36.9000	121.7500	10/25/1935	2056 0.0	0.0	4.00	0.045	VI	16.7(26.8)
GSB	36.8050	121.3900	01/07/2003	222926.9	8.0	4.30	0.052	VI	16.8(27.0)
GSB	36.9170	121.6750	04/18/1990	135351.4	5.0	5.40	0.093	VII	16.9(27.2)

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSB	36.9130	121.7100	10/18/1989	004539.6	27.0	4.00	0.044	VI	16.9(27.2)
BRK	36.6600	121.3400	09/24/1982	8 555.0	0.0	4.00	0.044	VI	16.9(27.2)
DMG	36.4500	121.5200	11/19/1969	62850.0	0.0	4.20	0.049	VI	16.9(27.2)
GSB	36.9180	121.6700	04/18/1990	134138.8	6.0	5.00	0.075	VII	16.9(27.2)
DMG	36.9200	121.6500	09/14/1963	202811.2	0.0	4.60	0.060	VI	17.0(27.4)
DMG	36.9200	121.6700	12/17/1953	51312.0	0.0	4.20	0.049	VI	17.1(27.5)
DMG	36.9200	121.6700	07/25/1921	5 5 0.0	0.0	4.00	0.044	VI	17.1(27.5)
GSB	36.9200	121.6780	04/18/1990	161913.2	9.0	4.20	0.049	VI	17.1(27.5)
GSB	36.6650	121.3350	09/08/1990	124822.3	8.0	4.00	0.044	VI	17.2(27.6)
DMG	36.6720	121.3340	01/15/1973	94329.9	6.3	4.07	0.045	VI	17.2(27.7)
GSB	36.8380	121.4090	03/22/1997	113149.3	9.0	4.20	0.048	VI	17.3(27.8)
DMG	36.6700	121.3300	01/08/1932	181659.0	0.0	4.00	0.043	VI	17.4(28.1)
DMG	36.9200	121.5700	05/13/1966	172555.9	0.0	4.50	0.056	VI	17.5(28.2)
MGI	36.9200	121.5700	02/07/1931	740 0.0	0.0	4.30	0.050	VI	17.5(28.2)
BRK	36.6400	121.3300	08/31/1982	311 7.0	0.0	4.00	0.043	VI	17.6(28.3)
GSB	36.6950	121.3280	07/03/2001	190250.8	6.0	4.20	0.048	VI	17.6(28.3)
GSB	36.6960	121.3280	07/02/2001	173353.7	4.0	4.10	0.045	VI	17.6(28.3)
GSB	36.9280	121.6800	04/18/1990	152816.5	6.0	4.70	0.062	VI	17.7(28.4)
DMG	36.9300	121.6800	04/25/1954	203328.0	0.0	5.30	0.084	VII	17.8(28.7)
DMG	36.9000	121.8000	03/11/1910	652 0.0	0.0	5.50	0.094	VII	17.8(28.7)
GSB	36.9280	121.7020	02/07/1990	141214.9	8.0	4.00	0.042	VI	17.8(28.7)
GSB	36.6920	121.3230	07/03/2001	190716.3	4.0	4.00	0.042	VI	17.9(28.8)
MGI	36.9200	121.7500	04/06/1915	1628 0.0	0.0	4.00	0.042	VI	18.0(28.9)
GSB	36.9320	121.6880	04/18/1990	153651.5	9.0	4.20	0.047	VI	18.0(29.0)
DMG	36.6700	121.3200	02/18/1956	235830.0	0.0	4.20	0.047	VI	18.0(29.0)
GSB	36.9320	121.6900	10/19/1989	095350.4	12.0	4.50	0.055	VI	18.0(29.0)
GSB	36.9320	121.6950	04/18/1990	154603.7	9.0	5.20	0.079	VII	18.0(29.0)
DMG	36.9000	121.4800	12/29/1959	23253.0	0.0	4.70	0.061	VI	18.1(29.1)
DMG	36.5000	121.4000	01/06/1931	232840.0	0.0	4.00	0.042	VI	18.1(29.1)
DMG	36.8500	121.4000	07/04/1939	1049 0.0	0.0	4.00	0.042	VI	18.2(29.3)
GSB	36.9320	121.7120	10/18/1989	063910.1	12.0	4.30	0.049	VI	18.2(29.3)
DMG	36.8200	121.3700	05/17/1945	15 647.0	0.0	4.60	0.057	VI	18.3(29.4)
DMG	36.5800	121.3300	06/09/1917	334 0.0	0.0	4.00	0.041	V	18.6(29.9)
DMG	36.9100	121.4800	11/28/1974	23 124.7	0.0	5.20	0.077	VII	18.7(30.1)
DMG	36.7500	121.3200	04/11/1942	84059.0	0.0	4.00	0.041	V	18.7(30.2)

DMG	36.8680	121.4080	03/31/1970	7 228.3	11.3	4.54	0.054	VI	18.8(30.2)
DMG	36.8700	121.8800	11/07/1958	213324.0	0.0	4.30	0.048	VI	18.8(30.2)
DMG	36.8800	121.4200	08/10/1947	2158 0.0	0.0	4.40	0.050	VI	18.9(30.4)
GSB	36.5700	121.3270	05/31/1986	084756.1	5.0	4.80	0.062	VI	19.0(30.6)
DMG	36.4800	121.4000	10/15/1942	135356.0	0.0	4.30	0.047	VI	19.1(30.7)
GSB	36.6750	121.3000	12/21/2008	173536.6	7.0	4.00	0.040	V	19.1(30.7)
DMG	36.6800	121.3000	04/09/1961	72316.0	0.0	5.60	0.094	VII	19.1(30.7)
DMG	36.9500	121.6700	03/16/1953	852 6.0	0.0	4.00	0.040	V	19.1(30.8)
DMG	36.4300	121.4800	01/11/1948	53728.0	0.0	4.30	0.047	VI	19.1(30.8)
GSB	36.6760	121.2990	03/23/1999	183640.4	6.0	4.20	0.045	VI	19.2(30.8)
DMG	36.7000	121.3000	04/09/1961	72541.0	0.0	5.50	0.089	VII	19.2(30.9)
DMG	36.7000	121.3000	09/11/1959	18 5 3.0	0.0	4.00	0.040	V	19.2(30.9)
DMG	36.7000	121.3000	03/31/1885	756 0.0	0.0	5.50	0.089	VII	19.2(30.9)
MGI	36.9500	121.6000	05/26/1937	211 0.0	0.0	4.30	0.047	VI	19.2(31.0)
DMG	36.9200	121.4800	11/29/1974	1 4 7.0	0.0	4.00	0.040	V	19.3(31.1)
DMG	36.9500	121.5900	01/10/1974	112224.8	0.0	4.40	0.049	VI	19.3(31.1)
BRK	36.6300	121.3000	08/11/1982	74643.0	0.0	4.60	0.055	VI	19.3(31.1)
DMG	36.9200	121.4700	12/31/1974	2022 1.1	0.0	4.30	0.046	VI	19.6(31.5)

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	36.6000	121.3000	11/22/1956	164358.0	0.0	4.20	0.044	VI	19.8(31.8)
DMG	36.6000	121.3000	04/08/1942	142014.0	0.0	4.00	0.039	V	19.8(31.8)
BRK	36.9300	121.4800	03/03/1975	113455.0	0.0	4.20	0.043	VI	19.9(32.0)
DMG	36.9000	121.4200	10/31/1951	205819.0	0.0	4.80	0.059	VI	20.0(32.1)
DMG	36.9000	121.4200	10/30/1951	195918.0	0.0	4.20	0.043	VI	20.0(32.1)
DMG	36.9000	121.4200	10/30/1951	195514.0	0.0	4.00	0.039	V	20.0(32.1)
GSB	36.9590	121.5760	10/10/1998	065031.4	6.0	4.10	0.041	V	20.1(32.3)
GSB	36.9650	121.5970	07/17/1997	194637.2	7.0	4.10	0.041	V	20.3(32.7)
MGI	36.6000	122.0000	07/03/1841	22 7 0.0	0.0	5.00	0.065	VI	20.3(32.7)
DMG	36.7500	122.0000	06/12/1921	342 0.0	0.0	4.00	0.038	V	20.3(32.7)
GSB	36.9670	121.6000	05/14/2002	050029.5	7.0	4.90	0.062	VI	20.4(32.8)
GSB	36.9630	121.7380	03/24/1991	034208.8	13.0	4.50	0.050	VI	20.6(33.2)
DMG	36.6250	121.2740	09/04/1972	18 440.8	5.7	4.63	0.053	VI	20.8(33.5)
GSB	36.6470	121.2680	01/18/1999	084836.7	11.0	4.30	0.044	VI	21.0(33.7)
GSB	36.8030	121.3020	02/20/1988	083957.5	9.0	5.30	0.075	VII	21.0(33.7)
DMG	36.9800	121.6700	06/05/1942	123325.0	0.0	4.20	0.041	V	21.2(34.1)
DMG	36.5800	121.2800	06/07/1944	123538.0	0.0	4.00	0.037	V	21.2(34.2)
DMG	36.9800	121.6000	03/02/1959	232717.0	30.0	5.30	0.074	VII	21.3(34.3)
DMG	36.9800	121.6000	03/03/1959	183213.0	0.0	4.00	0.037	V	21.3(34.3)
DMG	36.9000	121.3800	09/30/1950	212633.0	0.0	4.10	0.039	V	21.4(34.5)
BRK	36.7800	121.2800	08/14/1981	124959.0	0.0	4.20	0.041	V	21.5(34.6)
DMG	36.7700	121.2700	04/27/1948	202225.0	0.0	4.40	0.045	VI	21.8(35.1)
DMG	36.7700	121.2700	04/27/1948	1641 8.0	0.0	4.00	0.037	V	21.8(35.1)
DMG	36.6700	121.2500	08/06/1916	1938 0.0	0.0	5.50	0.080	VII	21.9(35.2)
T-A	36.6700	121.2500	04/01/1857	1135 0.0	0.0	5.00	0.062	VI	21.9(35.2)
DMG	36.6700	121.2500	06/27/1916	1343 0.0	0.0	4.00	0.036	V	21.9(35.2)
DMG	36.9800	121.7500	10/14/1966	203428.9	0.0	4.20	0.040	V	21.9(35.3)
GSB	36.6410	121.2510	12/28/2001	211402.0	6.0	4.60	0.050	VI	21.9(35.3)
DMG	36.6300	121.2500	11/03/1945	155022.0	0.0	4.20	0.040	V	22.1(35.5)
USG	36.8230	121.2900	01/26/1986	234654.5	4.8	4.20	0.040	V	22.2(35.7)
GSB	36.8390	121.2980	09/02/1998	112407.2	7.0	4.00	0.036	V	22.3(35.9)

GSB	36.9880	121.7470	08/06/1984	000639.6	10.0	4.00	0.036	V	22.4(36.1)
GSB	36.9920	121.7320	01/11/1994	105351.4	12.0	4.30	0.042	VI	22.5(36.2)
GSB	36.8100	121.2750	01/26/1986	192051.2	7.0	5.50	0.078	VII	22.5(36.3)
DMG	37.0000	121.6000	03/03/1959	72346.0	0.0	4.40	0.044	VI	22.7(36.5)
MGI	37.0000	121.6000	09/28/1900	1217 0.0	0.0	4.00	0.035	V	22.7(36.5)
GSB	36.9820	121.7880	10/25/1989	220149.8	14.0	4.00	0.035	V	22.7(36.5)
GSB	36.9630	121.8430	10/19/1989	101435.1	13.0	4.60	0.048	VI	22.8(36.7)
DMG	36.9800	121.4900	01/17/1966	2 320.0	0.0	4.10	0.037	V	22.8(36.7)
GSB	36.9800	121.8020	10/22/1989	142437.2	15.0	4.10	0.037	V	22.9(36.8)
GSB	36.9760	121.4750	12/13/1995	054512.8	7.0	4.00	0.035	V	22.9(36.8)
MGI	37.0000	121.5700	01/09/1928	250 0.0	0.0	5.30	0.070	VI	22.9(36.9)
T-A	37.0000	121.5700	03/25/1859	0 0 0.0	0.0	5.00	0.060	VI	22.9(36.9)
DMG	36.6500	121.2300	10/19/1956	1232 3.0	0.0	4.10	0.037	V	23.0(37.1)
GSB	36.9630	121.8530	10/19/1989	084549.9	12.0	4.00	0.035	V	23.0(37.1)
GSB	36.8400	121.2830	04/30/1987	192422.2	7.0	4.10	0.037	V	23.1(37.1)
GSB	36.9950	121.7630	10/18/1989	021549.9	4.0	4.50	0.045	VI	23.1(37.2)
BRK	36.5900	121.2400	08/10/1982	21129.0	0.0	4.50	0.045	VI	23.2(37.3)
GSB	36.9790	121.4670	08/06/1997	110437.3	7.0	4.00	0.035	V	23.3(37.4)
GSB	36.9810	121.4720	12/13/1995	062554.2	4.0	4.10	0.037	V	23.3(37.5)
DMG	36.6700	121.2200	04/16/1932	184810.0	0.0	4.50	0.045	VI	23.5(37.9)
DMG	36.6000	121.2300	09/28/1957	21 439.0	0.0	4.50	0.045	VI	23.5(37.9)
DMG	37.0000	121.7700	06/22/1947	2330 0.0	0.0	4.70	0.050	VI	23.6(37.9)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	36.6500	121.2200	01/14/1942	94440.0	5.0	4.20	0.038	V	23.6(38.0)
GSB	36.9900	121.8080	10/19/1989	035500.7	19.0	4.00	0.034	V	23.6(38.0)
DMG	37.0000	121.7800	12/18/1967	172432.0	0.0	5.30	0.068	VI	23.7(38.2)
DMG	36.3300	121.6700	10/18/1931	1958 0.0	0.0	4.00	0.034	V	23.8(38.2)
DMG	36.6200	121.2200	08/06/1951	9 5 2.0	0.0	4.90	0.055	VI	23.8(38.3)
DMG	37.0000	121.5000	06/20/1897	2014 0.0	0.0	6.20	0.108	VII	23.9(38.5)
GSB	36.9800	121.8470	10/18/1989	051834.1	11.0	4.20	0.038	V	23.9(38.5)
MGI	37.0000	121.8000	01/09/1937	047 0.0	0.0	4.30	0.040	V	24.1(38.8)
BRK	37.0200	121.7300	08/18/1982	84349.0	0.0	4.50	0.044	VI	24.4(39.2)
BRK	36.5800	121.2200	12/15/1977	111528.0	0.0	4.20	0.037	V	24.4(39.3)
DMG	36.5800	121.2200	10/10/1966	65346.0	0.0	4.10	0.035	V	24.4(39.3)
GSB	36.5830	121.2180	11/07/1987	150601.2	8.0	4.00	0.033	V	24.5(39.4)
BRK	37.0000	121.4700	08/06/1979	223355.0	0.0	4.30	0.039	V	24.5(39.5)
DMG	37.0200	121.7500	03/21/1968	2155 0.3	0.0	4.30	0.039	V	24.6(39.6)
GSB	36.6070	121.2080	11/24/2004	020604.5	6.0	4.40	0.041	V	24.6(39.7)
DMG	36.7000	121.2000	02/17/1937	333 0.0	0.0	4.50	0.043	VI	24.7(39.8)
DMG	37.0000	121.8300	09/19/1923	2254 0.0	0.0	4.00	0.033	V	24.7(39.8)
DMG	36.7300	121.2000	01/07/1945	222533.0	0.0	4.70	0.048	VI	24.9(40.1)
DMG	36.5780	121.2090	02/24/1972	155651.0	7.5	5.10	0.059	VI	25.0(40.3)
GSB	36.6030	121.2010	04/23/1995	084136.6	7.0	5.00	0.056	VI	25.1(40.3)
GSB	37.0150	121.8030	10/18/1989	102204.8	7.0	4.40	0.040	V	25.1(40.4)
DMG	36.6000	121.2000	03/17/1936	155 0.0	0.0	4.00	0.033	V	25.2(40.5)
DMG	36.6000	121.2000	06/11/1948	82811.0	0.0	4.00	0.033	V	25.2(40.5)
DMG	36.6500	122.1000	10/22/1926	16 349.0	0.0	4.50	0.042	VI	25.2(40.6)
GSB	36.3730	121.9070	01/23/1984	054019.7	7.0	5.40	0.068	VI	25.3(40.8)
GSB	36.5720	121.2050	01/14/1986	030936.3	7.0	4.70	0.047	VI	25.4(40.8)
GSB	36.5730	121.2030	10/05/1984	161630.0	9.0	4.00	0.032	V	25.5(41.0)

GSB	36.5930	121.1960	07/12/1995	215800.5	7.0	4.00	0.032	V	25.5(41.0)
GSB	36.3850	121.9330	02/10/1984	072324.4	6.0	4.30	0.038	V	25.5(41.1)
DMG	36.5630	121.2050	06/22/1973	12912.2	9.6	4.16	0.035	V	25.5(41.1)
GSB	37.0280	121.7800	10/18/1989	022606.7	12.0	4.20	0.036	V	25.6(41.2)
DMG	37.0200	121.4800	03/14/1949	61015.0	0.0	4.40	0.040	V	25.6(41.2)
DMG	37.0200	121.4800	03/09/1949	122839.0	0.0	5.20	0.061	VI	25.6(41.2)
BRK	37.0100	121.4500	08/09/1979	7 320.0	0.0	4.20	0.036	V	25.6(41.2)
GSB	36.3620	121.9000	01/24/1984	010038.2	5.0	4.00	0.032	V	25.7(41.4)
GSB	36.3680	121.9130	01/23/1984	054857.2	5.0	4.10	0.034	V	25.8(41.5)
GSB	36.3630	121.9100	01/23/1984	065950.4	5.0	5.00	0.054	VI	26.0(41.8)
MGI	36.6200	122.1100	10/22/1926	144149.5	0.0	4.00	0.032	V	26.0(41.9)
GSB	36.8100	121.2030	08/28/1994	012236.7	7.0	4.00	0.032	V	26.2(42.2)
DMG	37.0000	121.4000	06/18/1935	415 0.0	0.0	4.00	0.032	V	26.3(42.3)
GSB	36.8150	121.2030	09/22/1995	160618.1	7.0	4.30	0.037	V	26.3(42.4)
GSB	37.0250	121.4580	01/16/1993	062934.9	5.0	5.30	0.063	VI	26.4(42.4)
DMG	37.0400	121.7800	09/07/1967	123917.2	0.0	4.70	0.046	VI	26.4(42.4)
GSB	36.5840	121.1800	08/27/2011	071821.2	7.0	4.70	0.045	VI	26.5(42.6)
DMG	36.5800	121.1800	07/29/1951	105345.0	0.0	5.00	0.053	VI	26.6(42.7)
GSB	36.5770	121.1770	08/30/2004	043057.0	9.0	4.00	0.031	V	26.8(43.1)
DMG	37.0600	121.6900	11/16/1964	24641.7	0.0	5.00	0.053	VI	26.8(43.1)
DMG	36.4500	121.2500	09/27/1938	1223 0.0	0.0	5.00	0.053	VI	26.8(43.1)
GSB	36.5840	121.1740	10/11/1993	071945.2	6.0	4.10	0.033	V	26.8(43.2)
GSB	37.0390	121.4790	06/15/1998	015922.4	8.0	4.00	0.031	V	26.8(43.2)
GSB	36.5770	121.1750	09/27/1995	164442.3	9.0	4.20	0.035	V	26.9(43.3)
GSB	37.0430	121.8070	10/18/1989	002504.9	5.0	4.80	0.047	VI	27.0(43.5)
DMG	36.5800	121.1700	10/22/1949	214520.0	0.0	4.70	0.045	VI	27.1(43.6)

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSB	36.5580	121.1750	07/26/1988	032655.9	2.0	4.70	0.045	VI	27.2(43.8)
DMG	36.7800	122.1200	11/15/1947	222936.0	0.0	4.10	0.032	V	27.3(43.9)
DMG	36.5000	121.2000	09/18/1937	1329 0.0	0.0	4.00	0.031	V	27.4(44.1)
GSB	37.0570	121.7970	11/02/1989	055011.0	12.0	4.90	0.049	VI	27.8(44.7)
DMG	36.5500	121.1600	05/13/1964	121837.2	0.0	4.00	0.030	V	28.2(45.4)
GSB	37.0600	121.8130	10/30/1989	111713.7	13.0	4.10	0.032	V	28.2(45.4)
GSB	37.0360	121.8830	10/18/1989	000415.2	18.5	7.00	0.145	VIII	28.3(45.5)
DMG	36.5200	121.1700	03/27/1954	154331.0	0.0	4.40	0.037	V	28.4(45.7)
DMG	36.7900	122.1400	03/08/1971	183146.5	0.0	4.10	0.031	V	28.5(45.9)
DMG	37.0800	121.7500	05/28/1941	62318.0	0.0	4.50	0.039	V	28.6(46.1)
GSB	37.0470	121.8770	10/21/1989	004943.7	14.0	4.60	0.040	V	28.8(46.3)
DMG	36.5000	121.1700	05/27/1936	1955 0.0	0.0	4.50	0.038	V	28.9(46.6)
DMG	36.9000	121.2000	03/06/1882	2145 0.0	0.0	5.70	0.072	VI	29.1(46.9)
DMG	36.8000	122.1500	03/09/1971	153516.2	0.0	4.60	0.040	V	29.3(47.1)
GSB	37.0500	121.8950	10/26/1989	090129.3	14.0	4.00	0.029	V	29.4(47.4)
DMG	37.0000	121.3000	02/04/1942	9 824.0	0.0	4.00	0.029	V	29.5(47.5)
DMG	37.1000	121.6000	03/26/1866	2012 0.0	0.0	5.40	0.061	VI	29.5(47.5)
DMG	37.1000	121.7000	02/26/1864	1347 0.0	0.0	5.90	0.079	VII	29.6(47.6)
GSB	37.0780	121.8320	10/25/1989	012726.6	14.0	5.00	0.049	VI	29.8(47.9)
DMG	37.0800	121.8300	11/25/1919	11 3 0.0	0.0	4.50	0.037	V	29.9(48.0)
T-A	37.0000	122.0000	06/30/1890	2030 0.0	0.0	5.00	0.049	VI	29.9(48.1)
T-A	37.0000	122.0000	08/31/1868	0 0 0.0	0.0	4.30	0.034	V	29.9(48.1)
T-A	37.0000	122.0000	09/03/1858	840 0.0	0.0	4.30	0.034	V	29.9(48.1)

T-A	37.0000	122.0000	12/28/1885	1130 0.0	0.0	4.30	0.034	V	29.9(48.1)
MGI	37.0000	122.0000	12/11/1919	2327 0.0	0.0	4.00	0.029	V	29.9(48.1)
MGI	37.0000	122.0000	11/03/1916	9 3 0.0	0.0	4.00	0.029	V	29.9(48.1)
T-A	37.0000	122.0000	11/17/1868	2130 0.0	0.0	4.30	0.034	V	29.9(48.1)
DMG	37.0000	122.0000	02/12/1938	20 014.0	0.0	4.50	0.037	V	29.9(48.1)
MGI	37.0000	122.0000	07/17/1933	032 0.0	0.0	4.30	0.034	V	29.9(48.1)
T-A	37.0000	122.0000	09/02/1893	716 0.0	0.0	4.30	0.034	V	29.9(48.1)
DMG	37.0000	122.0000	11/12/1937	250 0.0	0.0	4.00	0.029	V	29.9(48.1)
DMG	37.0000	122.0000	04/23/1934	16 8 0.0	0.0	4.00	0.029	V	29.9(48.1)
T-A	37.0000	122.0000	04/03/1860	4 0 0.0	0.0	4.30	0.034	V	29.9(48.1)
DMG	36.5700	122.1700	10/22/1926	133522.0	0.0	6.10	0.087	VII	30.0(48.2)
GSG	37.0970	121.5140	09/13/1995	203646.6	8.0	4.20	0.032	V	30.1(48.5)
GSB	37.0570	121.9050	10/21/1989	221457.0	13.0	4.90	0.046	VI	30.1(48.5)
DMG	36.5300	121.1300	11/03/1960	65024.0	0.0	4.10	0.030	V	30.2(48.6)
DMG	36.6500	122.1900	08/04/1970	41421.4	0.0	4.70	0.041	V	30.2(48.6)
BRK	36.8200	121.1300	03/17/1976	4 152.0	0.0	4.30	0.033	V	30.2(48.6)
PAS	36.7330	121.1020	11/28/1985	151354.7	6.0	4.90	0.046	VI	30.3(48.8)
GSB	37.0580	121.9150	11/05/1989	133734.3	15.0	4.50	0.037	V	30.4(49.0)
DMG	37.0300	121.9700	04/14/1941	161654.0	0.0	4.00	0.028	V	30.5(49.0)
GSB	37.0620	121.9080	10/18/1989	041633.0	16.0	4.10	0.030	V	30.5(49.1)
BRK	37.1000	121.5000	08/06/1979	17 522.0	0.0	5.80	0.073	VII	30.5(49.1)
DMG	37.1000	121.8000	05/24/1865	1121 0.0	0.0	5.50	0.062	VI	30.7(49.3)
DMG	37.1000	121.8000	09/17/1888	1151 0.0	0.0	4.50	0.037	V	30.7(49.3)
GSB	37.1020	121.4920	06/15/2006	122451.1	3.0	4.30	0.033	V	30.8(49.5)
DMG	36.8000	122.1800	04/16/1971	125831.7	0.0	4.50	0.036	V	30.9(49.7)
DMG	36.4000	121.2000	07/22/1961	18 155.0	0.0	4.00	0.028	V	31.1(50.0)
DMG	36.4000	121.2000	09/16/1938	611 0.0	0.0	4.00	0.028	V	31.1(50.0)
BRK	36.5700	121.0900	10/13/1980	24654.0	0.0	4.00	0.027	V	31.6(50.8)
DMG	36.5290	121.1030	02/27/1972	2213 8.4	11.4	4.62	0.038	V	31.7(50.9)
DMG	37.1300	121.5700	01/29/1959	164123.0	0.0	4.30	0.032	V	31.8(51.1)

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSB	37.1220	121.5120	06/20/1988	152638.2	7.0	4.20	0.030	V	31.8(51.2)
GSB	37.0770	121.9250	11/05/1989	013042.4	15.0	4.20	0.030	V	31.9(51.3)
BRK	37.1000	121.8700	04/25/1981	194137.0	0.0	4.10	0.029	V	32.0(51.4)
DMG	36.3000	121.3000	07/23/1956	8 348.0	0.0	4.70	0.039	V	32.1(51.7)
GSB	37.1380	121.5700	04/26/1984	062951.9	5.0	4.00	0.027	V	32.3(52.0)
GSG	36.5190	121.0930	04/01/2006	122559.8	1.0	4.30	0.032	V	32.4(52.1)
DMG	37.1000	121.9000	08/08/1916	1650 0.0	0.0	4.00	0.027	V	32.6(52.5)
DMG	37.1000	121.9000	04/15/1889	328 0.0	0.0	4.80	0.041	V	32.6(52.5)
DMG	36.4500	121.1200	10/11/1959	2 3 9.0	0.0	4.10	0.028	V	33.0(53.0)
BRK	36.5800	121.0600	08/08/1983	1313 9.0	0.0	4.00	0.027	V	33.0(53.2)
GSB	37.1500	121.5820	01/19/1987	080904.4	7.0	4.30	0.031	V	33.1(53.2)
GSB	37.0930	121.9420	10/20/1989	001820.8	12.0	4.30	0.031	V	33.3(53.5)
BRK	36.9000	122.1800	07/02/1978	115757.0	0.0	4.20	0.029	V	33.5(53.8)
MGI	37.0000	122.1000	04/29/1908	841 0.0	0.0	4.00	0.026	V	33.8(54.3)
MGI	37.0000	122.1000	01/08/1907	2345 0.0	0.0	4.60	0.036	V	33.8(54.3)
GSB	37.1300	121.8780	06/27/1988	184322.3	13.0	5.70	0.064	VI	34.0(54.8)
BRK	36.5100	121.0600	06/19/1982	101733.0	0.0	4.10	0.027	V	34.3(55.3)
DMG	37.1700	121.5200	11/08/1945	20 917.0	0.0	4.20	0.028	V	35.0(56.3)
DMG	36.1700	121.5300	02/22/1937	1810 0.0	0.0	4.00	0.025	V	35.3(56.9)

GSB	37.1230	121.9630	10/18/1989	003041.4	5.0	4.20	0.028	V	35.6(57.4)
GSB	37.1300	121.9520	08/08/1989	081327.5	15.0	5.30	0.050	VI	35.8(57.6)
GSB	37.1350	121.9450	08/08/1989	084410.0	13.0	4.50	0.032	V	35.9(57.8)
DMG	37.1800	121.8000	04/10/1954	221653.0	0.0	4.10	0.026	V	36.0(57.9)
GSB	37.0920	122.0420	12/29/1998	123812.4	10.0	4.00	0.025	V	36.3(58.4)
DMG	36.3500	121.1200	09/21/1958	72455.0	0.0	4.60	0.034	V	36.7(59.1)
DMG	36.4300	121.0500	11/17/1969	204919.5	0.0	4.40	0.030	V	37.0(59.6)
DMG	37.2000	121.5000	07/06/1899	2010 0.0	0.0	5.80	0.063	VI	37.2(59.9)
GSB	37.1500	121.9730	08/08/1989	155328.4	15.0	4.80	0.037	V	37.5(60.4)
DMG	36.5000	121.0000	06/16/1934	23 3 0.0	0.0	4.00	0.024	V	37.7(60.7)
DMG	36.5000	121.0000	02/10/1946	11 119.0	0.0	4.20	0.027	V	37.7(60.7)
DMG	36.2000	121.3000	05/13/1938	1934 0.0	0.0	4.00	0.024	IV	37.9(61.0)
DMG	36.2000	121.3000	05/10/1938	1041 0.0	0.0	4.00	0.024	IV	37.9(61.0)
DMG	36.2000	121.3000	05/10/1938	1032 0.0	0.0	4.50	0.031	V	37.9(61.0)
DMG	37.2300	121.6100	09/28/1967	153836.1	0.0	4.90	0.038	V	38.5(61.9)
DMG	37.0200	122.2000	10/24/1926	225149.5	0.0	5.50	0.052	VI	38.9(62.6)
GSB	37.1630	121.9950	10/18/1989	032357.0	22.0	4.00	0.023	IV	38.9(62.6)
DMG	36.9500	122.2600	02/15/1927	2354 3.5	0.0	5.00	0.040	V	39.0(62.7)
DMG	36.6100	122.3500	10/22/1926	1235 7.0	0.0	6.10	0.070	VI	39.3(63.2)
DMG	37.1700	122.0000	11/09/1914	231 0.0	0.0	5.50	0.051	VI	39.5(63.5)
DMG	37.2500	121.6700	07/09/1958	52340.0	0.0	4.10	0.024	V	39.8(64.1)
DMG	36.4200	121.0000	11/26/1929	8 5 0.0	0.0	4.50	0.030	V	39.8(64.1)
GSB	37.1500	122.0530	10/18/1989	003828.8	20.0	4.30	0.027	V	39.9(64.1)
DMG	37.2500	121.7500	07/01/1911	22 0 0.0	0.0	6.60	0.090	VII	40.2(64.7)
GSB	37.2580	121.6620	11/11/1991	095643.4	4.0	4.10	0.024	V	40.4(64.9)
DMG	36.4000	121.0000	04/12/1885	4 5 0.0	0.0	6.20	0.072	VII	40.5(65.1)
GSG	36.4335	120.9710	01/20/2015	132137.0	10.0	4.43	0.028	V	40.9(65.8)
GSB	37.2680	121.6100	06/05/1984	165620.8	6.0	4.20	0.025	V	41.1(66.1)
MGI	37.2000	122.0000	01/03/1935	1736 0.0	0.0	4.30	0.026	V	41.3(66.4)
MGI	37.2000	122.0000	12/30/1934	1355 0.0	0.0	4.30	0.026	V	41.3(66.4)
GSB	37.1930	122.0170	10/18/1989	045027.7	14.0	4.30	0.026	V	41.3(66.5)
DMG	37.2700	121.7200	03/24/1959	22113.0	0.0	4.10	0.024	IV	41.4(66.6)
GSB	37.2770	121.6700	04/27/1984	164834.1	10.0	4.00	0.022	IV	41.7(67.1)
DMG	37.2700	121.8000	08/27/1945	913 4.0	0.0	4.50	0.029	V	42.1(67.7)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSB	37.2850	121.6150	03/30/2009	174029.3	7.0	4.30	0.026	V	42.2(68.0)
DMG	37.2260	121.9750	11/12/1973	181713.3	12.9	4.01	0.022	IV	42.3(68.0)
GSB	37.2870	121.6580	01/08/2011	001016.7	10.0	4.10	0.023	IV	42.4(68.2)
GSB	37.2870	121.6680	05/03/1984	130711.8	10.0	4.40	0.027	V	42.4(68.2)
GSB	37.2870	121.6900	12/30/1988	235424.4	7.0	4.30	0.026	V	42.4(68.3)
DMG	37.1000	122.2000	03/26/1884	040 0.0	0.0	5.90	0.060	VI	42.5(68.4)
GSB	37.2880	121.6970	08/17/1990	022617.2	5.0	4.00	0.022	IV	42.5(68.4)
GSB	37.1830	122.0800	10/20/1989	081254.2	16.0	4.00	0.022	IV	42.6(68.5)
GSB	36.0800	121.8620	07/08/1986	004022.9	14.0	4.40	0.027	V	42.7(68.7)
GSB	37.2950	121.6670	09/03/1986	043114.8	10.0	4.20	0.024	V	42.9(69.1)
GSB	37.2080	122.0450	12/02/1989	200200.8	11.0	4.00	0.022	IV	43.0(69.2)
DMG	37.3000	121.6700	06/10/1949	3 640.0	0.0	4.60	0.030	V	43.3(69.6)
BRK	37.3000	121.6800	05/08/1979	511 7.0	0.0	4.80	0.033	V	43.3(69.7)
MGI	37.3000	121.6000	12/11/1901	2158 0.0	0.0	4.00	0.022	IV	43.3(69.7)
MGI	37.3000	121.6000	08/27/1904	205847.0	0.0	4.60	0.030	V	43.3(69.7)

MGI	37.3000	121.7500	03/11/1936	1356 0.0	0.0	4.30	0.025	V	43.6(70.2)
GSB	36.7770	120.8670	01/06/1988	224948.3	6.0	4.20	0.024	IV	43.6(70.2)
GSB	37.2270	122.0370	11/07/1989	234237.7	11.0	4.30	0.025	V	43.9(70.7)
BRK	36.6800	120.8500	02/05/1983	12 127.0	0.0	4.20	0.024	IV	44.0(70.8)
DMG	37.3000	121.8000	08/03/1903	649 0.0	0.0	5.50	0.047	VI	44.1(70.9)
DMG	37.3000	121.8000	01/02/1891	20 0 0.0	0.0	5.50	0.047	VI	44.1(70.9)
DMG	37.2000	122.1000	02/17/1870	2012 0.0	0.0	5.80	0.055	VI	44.2(71.1)
GSG	37.3130	121.6750	08/11/1993	223304.0	9.0	4.80	0.032	V	44.2(71.1)
GSB	37.1980	122.1050	10/18/1989	004124.7	19.0	5.10	0.038	V	44.2(71.2)
GSB	36.0350	121.5600	02/04/1991	163836.2	13.0	4.70	0.031	V	44.3(71.4)
DMG	37.2500	122.0000	07/19/1925	1924 0.0	24.0	4.00	0.021	IV	44.4(71.4)
GSB	37.3200	121.6980	04/24/1984	211519.0	8.0	6.20	0.067	VI	44.7(72.0)
DMG	37.1700	122.1700	12/28/1914	1042 0.0	0.0	4.50	0.027	V	44.9(72.2)
GSB	37.3230	121.7250	02/08/1988	140915.3	6.0	4.00	0.021	IV	45.1(72.5)
GSG	36.6255	120.8337	09/28/2014	204513.3	7.6	4.43	0.026	V	45.1(72.5)
T-A	37.3300	121.6700	01/15/1890	13 5 0.0	0.0	4.30	0.024	V	45.3(73.0)
T-A	37.3300	121.6700	03/08/1912	948 0.0	0.0	4.30	0.024	V	45.3(73.0)
T-A	37.3300	121.6700	06/28/1891	11 3 0.0	0.0	4.30	0.024	V	45.3(73.0)
DMG	37.3300	121.6700	04/03/1924	2354 0.0	0.0	4.50	0.027	V	45.3(73.0)
T-A	37.3300	121.6700	11/04/1888	1136 0.0	0.0	4.30	0.024	V	45.3(73.0)
T-A	37.3300	121.6700	10/12/1887	855 0.0	0.0	4.30	0.024	V	45.3(73.0)
T-A	37.3300	121.6700	02/05/1892	1428 0.0	0.0	4.30	0.024	V	45.3(73.0)
DMG	37.3300	121.6800	06/22/1949	18 846.0	7.0	4.10	0.022	IV	45.4(73.0)
DMG	37.3000	121.9000	04/21/1904	1150 0.0	0.0	4.00	0.021	IV	45.5(73.2)
DMG	37.3000	121.9000	10/08/1865	2046 0.0	0.0	6.30	0.070	VI	45.5(73.2)
MGI	37.3000	121.9000	05/28/1927	1739 0.0	0.0	5.00	0.035	V	45.5(73.2)
MGI	37.3000	121.9000	04/04/1905	1023 0.0	0.0	4.00	0.021	IV	45.5(73.2)
GSB	37.3330	121.6990	02/25/2001	231822.5	7.0	4.50	0.027	V	45.6(73.4)
DMG	37.3300	121.7500	10/26/1917	920 0.0	0.0	4.00	0.021	IV	45.7(73.5)
USG	37.3380	121.6970	09/26/1984	2046 6.0	9.5	4.50	0.027	V	46.0(74.0)
GSB	37.3400	121.7320	09/26/1984	204606.2	6.0	4.40	0.025	V	46.3(74.4)
MGI	37.3300	121.8200	09/02/1927	0 0 0.0	0.0	4.60	0.028	V	46.3(74.6)
DMG	37.2500	122.0800	06/14/1932	94417.0	0.0	4.50	0.027	V	46.5(74.8)
DMG	36.6000	120.8000	07/25/1926	175749.0	15.0	5.00	0.034	V	47.1(75.8)
GSG	37.3590	121.7240	05/21/1996	205020.2	7.0	4.70	0.029	V	47.5(76.5)
BRK	37.3600	121.7200	08/29/1978	01446.0	0.0	4.10	0.021	IV	47.6(76.6)
T-A	37.3300	121.9200	07/23/1885	2025 0.0	0.0	4.30	0.023	IV	47.8(76.9)
T-A	37.3300	121.9200	04/04/1859	21 0 0.0	0.0	4.30	0.023	IV	47.8(76.9)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
T-A	37.3300	121.9200	08/30/1873	0 0 0.0	0.0	4.30	0.023	IV	47.8(76.9)
DMG	37.3300	121.9200	06/10/1931	1220 0.0	0.0	4.00	0.020	IV	47.8(76.9)
MGI	37.3300	121.9200	09/09/1920	1647 0.0	0.0	4.00	0.020	IV	47.8(76.9)
T-A	37.3300	121.9200	08/11/1859	530 0.0	0.0	4.30	0.023	IV	47.8(76.9)
DMG	37.3700	121.7800	09/05/1955	2 118.0	0.0	5.50	0.043	VI	48.7(78.3)
GSB	37.3730	121.7570	11/10/1988	050803.0	7.0	4.80	0.030	V	48.7(78.3)
DMG	37.3800	121.6800	09/18/1941	733 0.0	0.0	4.00	0.020	IV	48.8(78.5)
BRK	37.3800	121.7200	01/15/1981	124751.0	0.0	4.80	0.030	V	48.9(78.8)
GSB	37.3850	121.7720	06/13/1988	014536.8	7.0	5.40	0.041	V	49.6(79.8)
DMG	36.7800	120.7500	04/22/1932	0 816.0	0.0	4.00	0.019	IV	50.1(80.6)
GSB	36.3110	120.8560	10/21/2012	065509.5	9.0	5.30	0.038	V	50.4(81.2)

DMG	36.0000	122.0000	06/27/1932	51725.0	0.0	4.00	0.019	IV	50.5(81.3)
GSB	37.3950	121.4870	02/05/2005	184330.4	7.0	4.10	0.020	IV	50.6(81.4)
DMG	35.9500	121.5000	12/03/1934	154 0.0	0.0	4.50	0.025	V	50.6(81.4)
MGI	36.1500	121.0000	11/03/1937	10 0 0.0	0.0	4.30	0.022	IV	50.9(81.9)
GSB	37.4030	121.7880	07/09/1989	133844.5	5.0	4.00	0.019	IV	51.0(82.0)
GSB	36.4050	120.7880	01/12/1990	091023.1	14.0	4.20	0.021	IV	51.0(82.1)
DMG	37.4000	121.4000	04/10/1881	10 0 0.0	0.0	5.90	0.051	VI	51.9(83.6)
GSB	37.4270	121.7000	04/20/1991	194153.4	5.0	4.20	0.021	IV	52.1(83.8)
GSB	37.4230	121.7630	07/01/1990	003641.6	7.0	4.20	0.021	IV	52.1(83.9)
DMG	37.4300	121.6800	10/26/1943	45033.0	12.0	4.90	0.030	V	52.3(84.1)
GSB	37.4220	121.7950	04/03/1989	174634.4	9.0	4.70	0.027	V	52.3(84.2)
BRK	37.1900	120.9500	06/18/1975	175019.0	0.0	4.10	0.020	IV	52.4(84.3)
GSB	37.4340	121.7740	10/31/2007	030454.8	10.0	5.50	0.041	V	53.0(85.3)
DMG	36.8000	120.7000	12/05/1937	137 0.0	0.0	4.00	0.018	IV	53.0(85.3)
GSG	37.4350	121.7790	10/31/2007	225424.5	8.0	4.00	0.018	IV	53.1(85.4)
DMG	36.1700	120.9200	05/23/1936	441 0.0	0.0	4.00	0.018	IV	53.2(85.6)
DMG	36.0000	121.1700	10/24/1916	1330 0.0	0.0	4.00	0.018	IV	53.5(86.1)
DMG	37.3300	122.1700	09/12/1912	1727 0.0	0.0	4.50	0.024	IV	53.8(86.5)
GSB	37.4550	121.7000	02/28/1987	162428.3	6.0	4.00	0.018	IV	54.0(87.0)
DMG	37.4200	121.3300	07/29/1954	85136.0	0.0	4.20	0.020	IV	54.4(87.5)
DMG	36.5300	120.6800	08/19/1974	124719.1	0.0	4.40	0.022	IV	54.4(87.5)
GSB	37.4580	121.8000	12/29/1986	152804.9	6.0	4.50	0.023	IV	54.8(88.2)
DMG	37.4200	122.0000	02/11/1930	2121 0.0	0.0	4.00	0.018	IV	55.1(88.7)
DMG	37.2700	122.3100	05/22/1963	2241 4.8	0.0	4.60	0.025	V	55.1(88.7)
GSB	37.4830	121.6900	03/31/1986	115540.0	8.0	5.70	0.043	VI	55.9(90.0)
DMG	37.4800	121.7800	10/31/1958	02614.0	0.0	4.20	0.020	IV	56.2(90.4)
GSB	37.4810	121.7980	01/07/2010	180935.0	11.0	4.10	0.019	IV	56.4(90.7)
GSG	35.8652	121.4993	02/27/2014	185230.0	9.8	4.04	0.018	IV	56.4(90.8)
DMG	37.3500	122.2200	10/31/1957	1947 6.0	0.0	4.10	0.019	IV	56.4(90.8)
DMG	37.5000	121.8000	12/01/1938	1617 0.0	0.0	4.50	0.022	IV	57.7(92.8)
DMG	36.0000	121.0000	02/26/1932	1658 0.0	0.0	5.00	0.029	V	58.7(94.5)
DMG	36.0000	121.0000	06/30/1935	2328 0.0	0.0	4.00	0.017	IV	58.7(94.5)
MGI	36.0000	121.0000	10/20/1942	1025 0.0	0.0	4.30	0.020	IV	58.7(94.5)
MGI	36.0000	121.0000	10/18/1942	12 142.0	0.0	4.30	0.020	IV	58.7(94.5)
DMG	37.5000	121.9000	11/26/1858	835 0.0	0.0	6.10	0.051	VI	58.8(94.6)
MGI	36.3300	120.6700	07/31/1919	2131 0.0	0.0	4.00	0.017	IV	59.1(95.1)
T-A	37.4200	122.1700	02/00/1888	12 0 0.0	0.0	4.30	0.020	IV	59.1(95.1)
DMG	36.4200	120.6200	04/15/1962	841 2.3	21.0	4.70	0.024	V	59.5(95.7)
DMG	36.4170	120.6170	04/15/1963	841 2.3	0.0	4.70	0.024	V	59.7(96.1)
DMG	37.5000	121.3000	07/15/1866	630 0.0	0.0	5.80	0.043	VI	60.1(96.8)
BRK	35.8400	121.3300	08/29/1983	101031.0	0.0	5.20	0.031	V	60.2(96.8)
DMG	35.8000	121.5000	12/20/1948	44246.0	0.0	4.50	0.022	IV	60.9(97.9)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSB	35.8280	121.3230	09/17/1991	211029.3	8.0	5.10	0.029	V	61.1(98.3)
PAS	36.0360	120.8810	11/24/1985	192139.8	6.0	4.40	0.020	IV	61.2(98.4)
DMG	36.0000	120.9300	03/02/1955	1559 1.0	0.0	4.80	0.025	V	61.2(98.5)
BRK	37.5600	121.6600	08/24/1980	124117.0	0.0	4.00	0.016	IV	61.2(98.5)
DMG	35.8700	122.1200	01/29/1957	211953.0	0.0	4.90	0.026	V	61.5(98.9)
DMG	36.0000	120.9200	11/02/1955	1940 6.0	0.0	5.20	0.031	V	61.6(99.1)
GSB	37.5140	121.2620	09/07/1994	190959.9	3.0	4.10	0.017	IV	61.7(99.4)

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GSB |37.5680|121.6650|12/11/1986|141805.3|  4.0| 4.10| 0.017 | IV | 61.8( 99.4)
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-END OF SEARCH- 485 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2016

LENGTH OF SEARCH TIME: 217 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 1.4 MILES (2.3 km) AWAY.

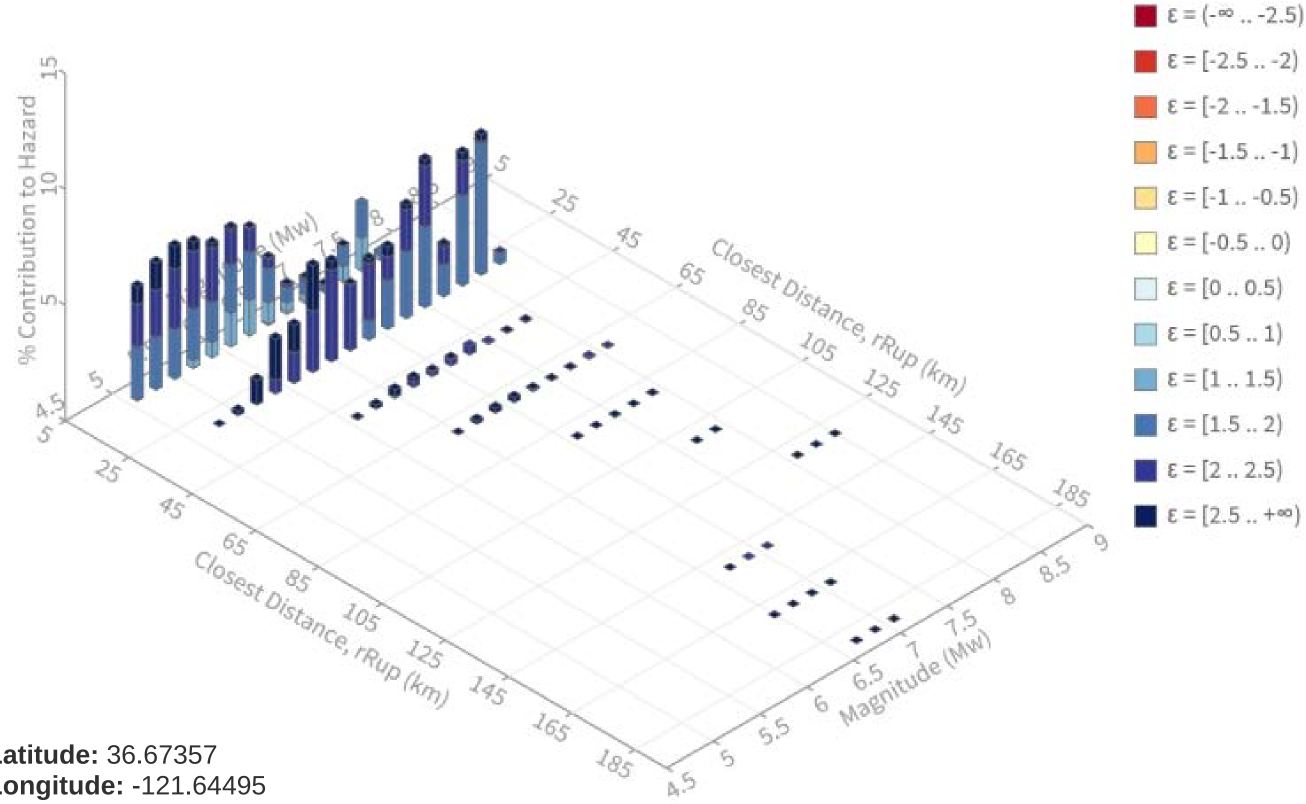
LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.284 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 3.525
b-value= 0.793
beta-value= 1.826

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	485	2.24537
4.5	185	0.85648
5.0	79	0.36574
5.5	39	0.18056
6.0	11	0.05093
6.5	3	0.01389
7.0	2	0.00926



Latitude: 36.67357
Longitude: -121.64495
Edition: Dynamic, Conterminous U.S. 2008 (v3.3.1)
Site Class: D/E Boundary, 180m/s

Summary Statistics for, Deaggregation: Total

Deaggregation targets
Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA Ground Motion: 0.6093494g

Recovered targets
Return period: 3427.4809 yrs
Exceedance rate: 0.00029175946 yr^{-1}

Totals
Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (for all sources)
r: 18.66 km
m: 6.59
ε_0: 2.04 σ

Mode (largest r-m bin)
r: 24.07 km
m: 7.5
ε_0: 2.03 σ
Contribution: 6.37 %

Mode (largest ε_0 bin)
r: 20.7 km
m: 8.08
ε_0: 1.81 σ
Contribution: 5.66 %

Discretization
r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys
ε0: [-∞ ‥ -2.5)
ε1: [-2.5 ‥ -2.0)
ε2: [-2.0 ‥ -1.5)
ε3: [-1.5 ‥ -1.0)
ε4: [-1.0 ‥ -0.5)
ε5: [-0.5 ‥ 0.0)
ε6: [0.0 ‥ 0.5)
ε7: [0.5 ‥ 1.0)
ε8: [1.0 ‥ 1.5)
ε9: [1.5 ‥ 2.0)
ε10: [2.0 ‥ 2.5)
ε11: [2.5 ‥ +∞]

USGS Design Maps Detailed Report

ASCE 7-10 Standard (36.67357°N, 121.64495°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.596$ g

From Figure 22-2 [2] $S_1 = 0.590$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.596 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.590 g, F_v = 1.500

Equation (11.4-1):	$S_{MS} = F_aS_S = 1.000 \times 1.596 = 1.596$ g
Equation (11.4-2):	$S_{M1} = F_vS_1 = 1.500 \times 0.590 = 0.885$ g

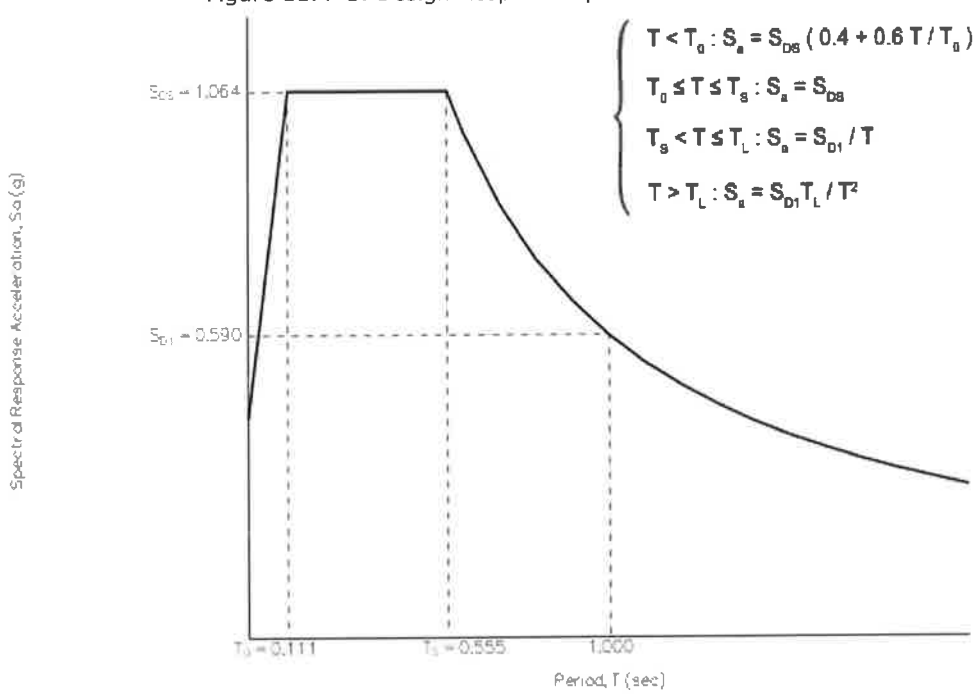
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.596 = 1.064$ g
Equation (11.4-4):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.885 = 0.590$ g

Section 11.4.5 — Design Response Spectrum

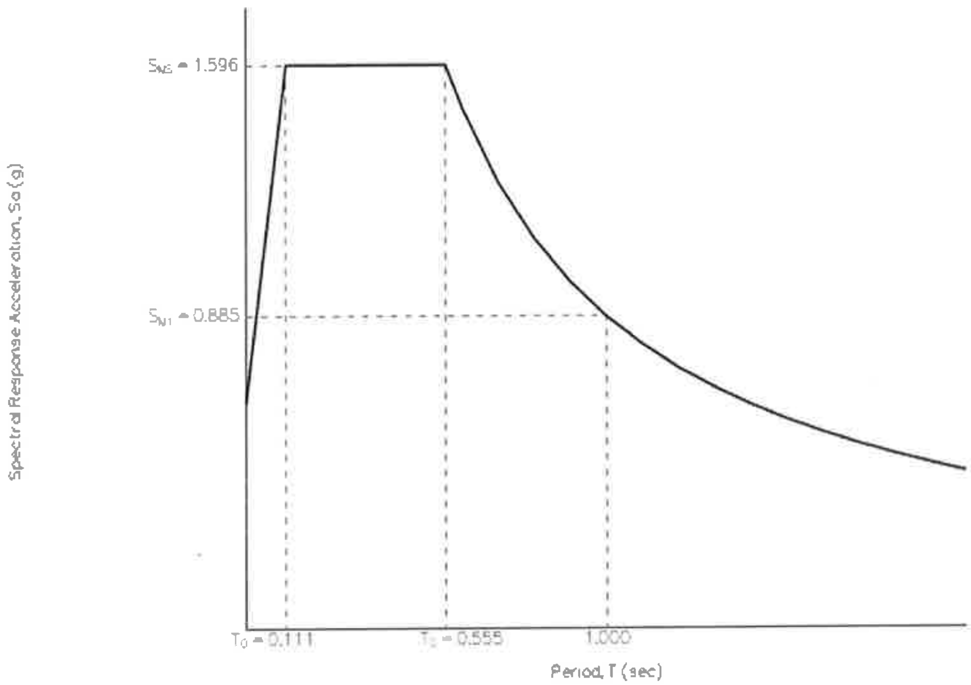
From Figure 22-12 [3]	$T_L = 12$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.603

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.603 = 0.603$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.603 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.059

From Figure 22-18 [6] C_{R1} = 1.001

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.064 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.590 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX D2

Liquefaction Analysis

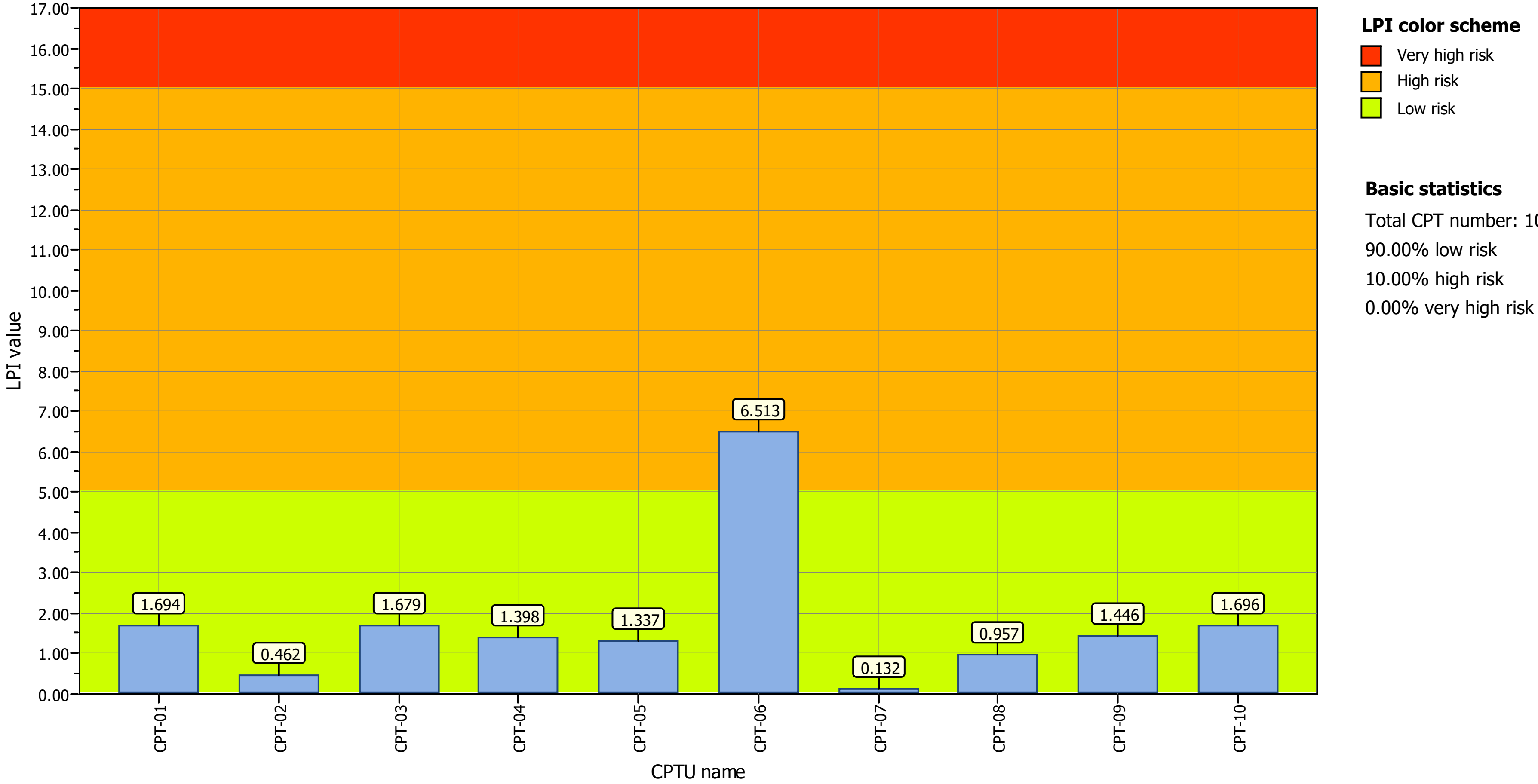
Leighton

Leighton and Associates, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongroup.com

Project title : Salinas Police HQ

Location : 321 E. Alisal Street, Salinas, CA

Overall Liquefaction Potential Index report



Basic statistics

Total CPT number: 10

90.00% low risk

10.00% high risk

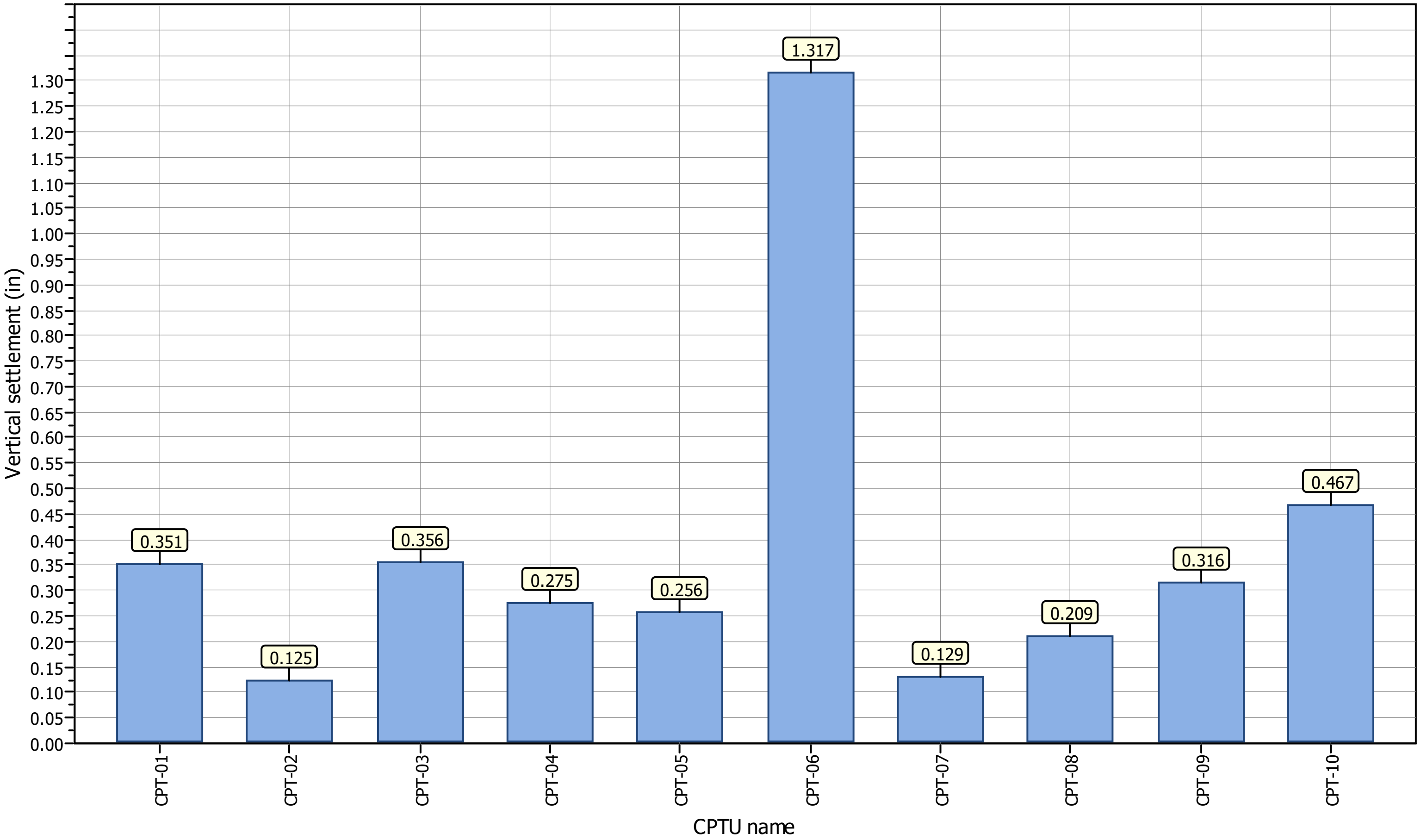
0.00% very high risk

Leighton and Associates, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongroup.com

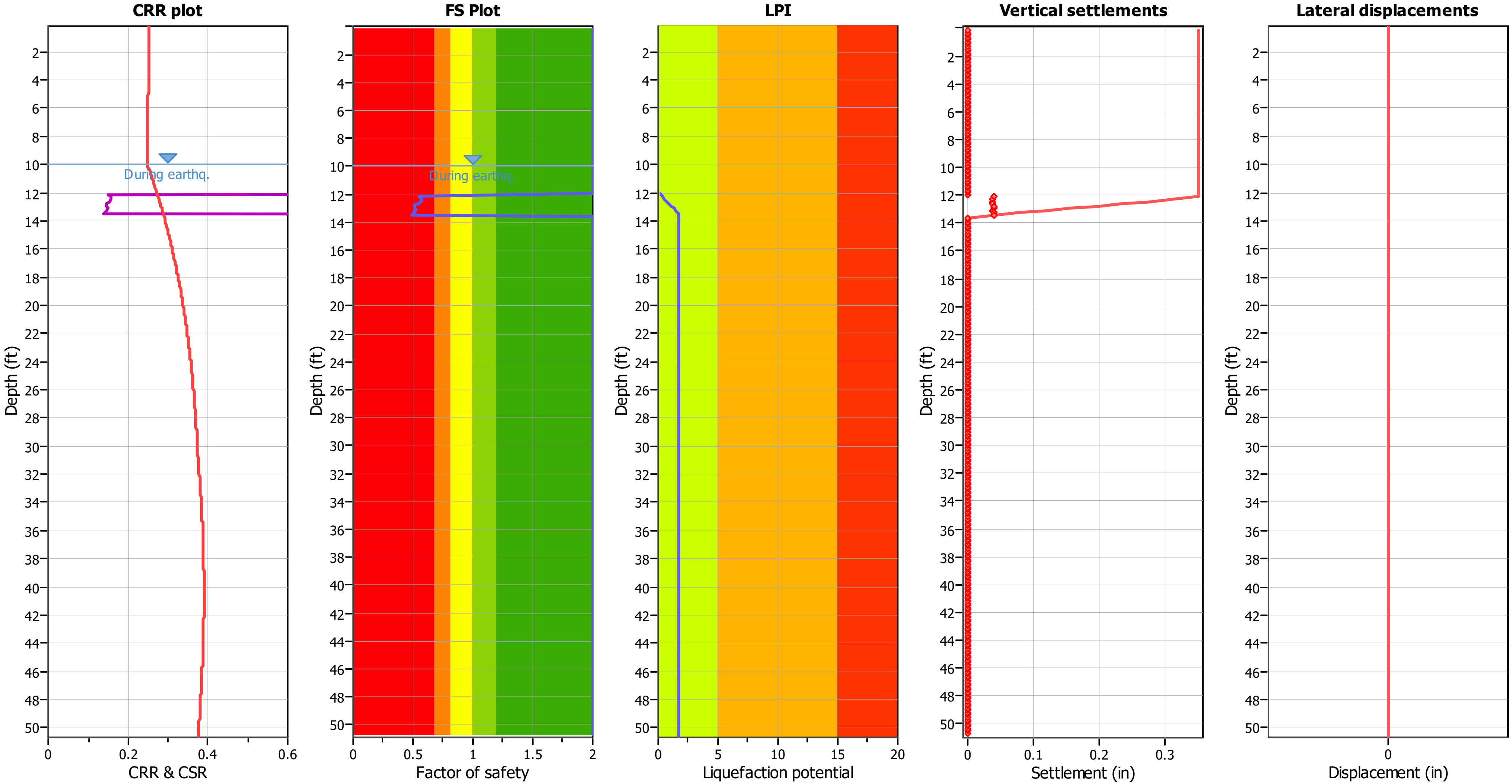
Project title : Salinas Police HQ

Location : 321 E. Alisal Street, Salinas, CA

Overall vertical settlements report



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

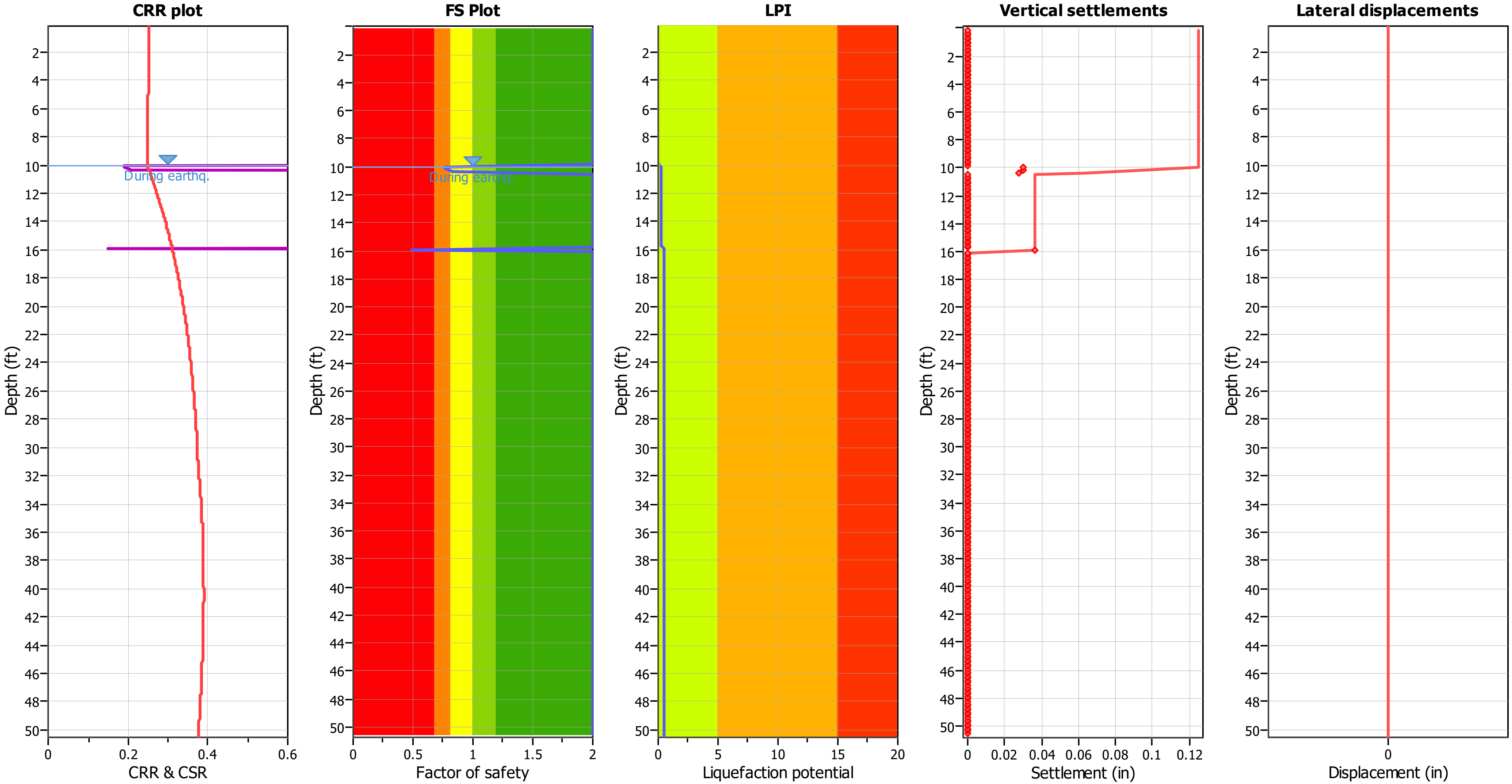
F.S. color scheme

- 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

Liquefaction analysis overall plots



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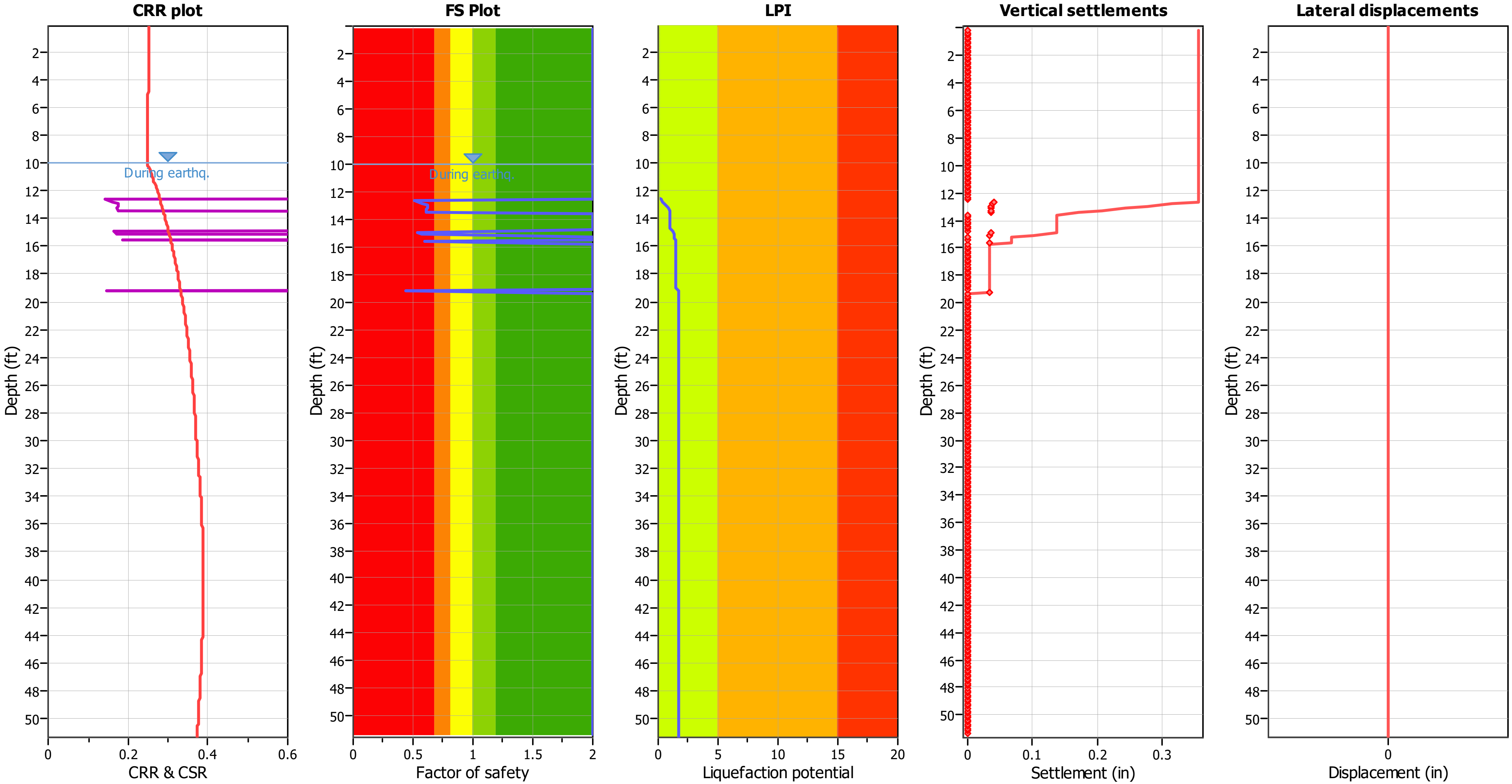

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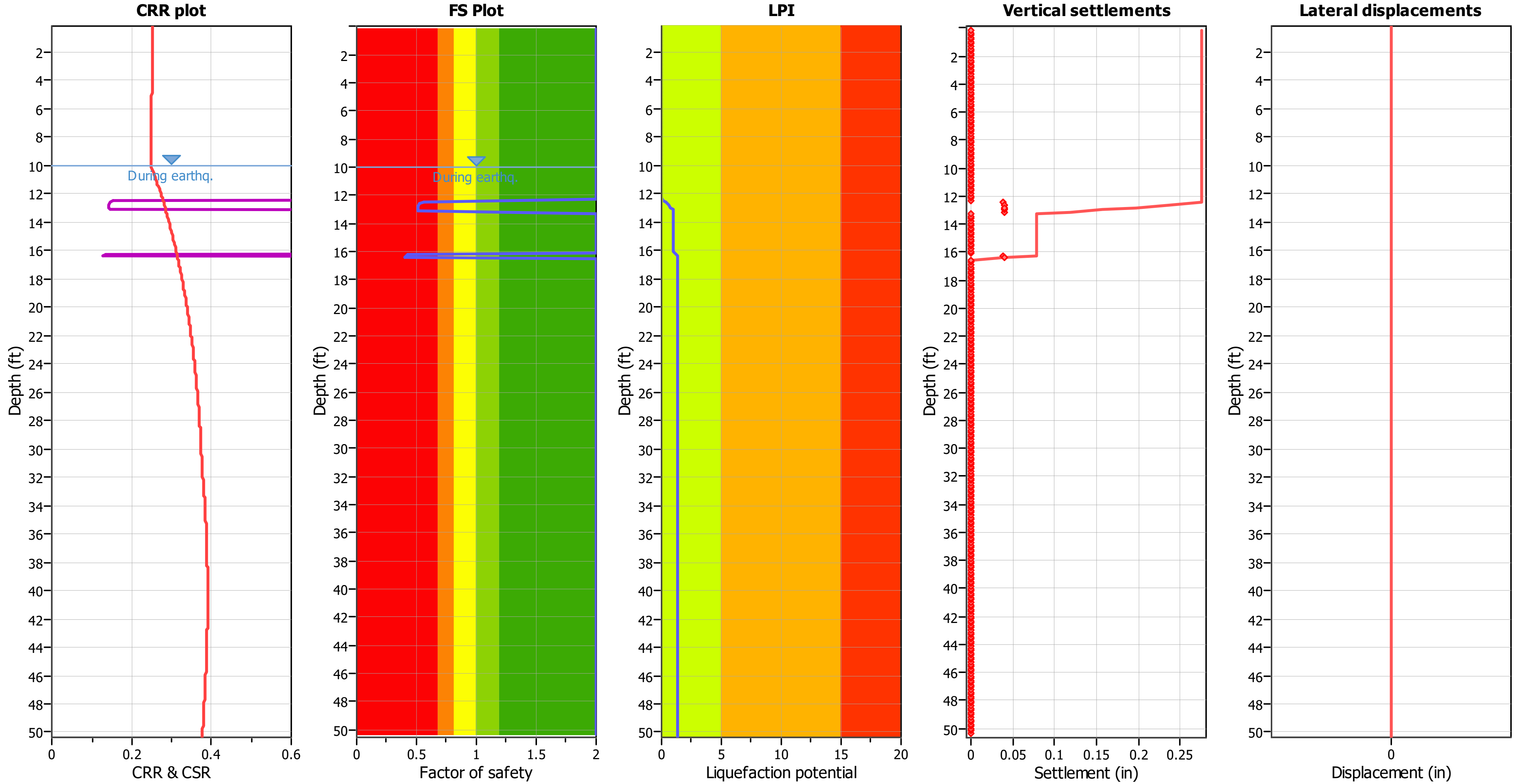

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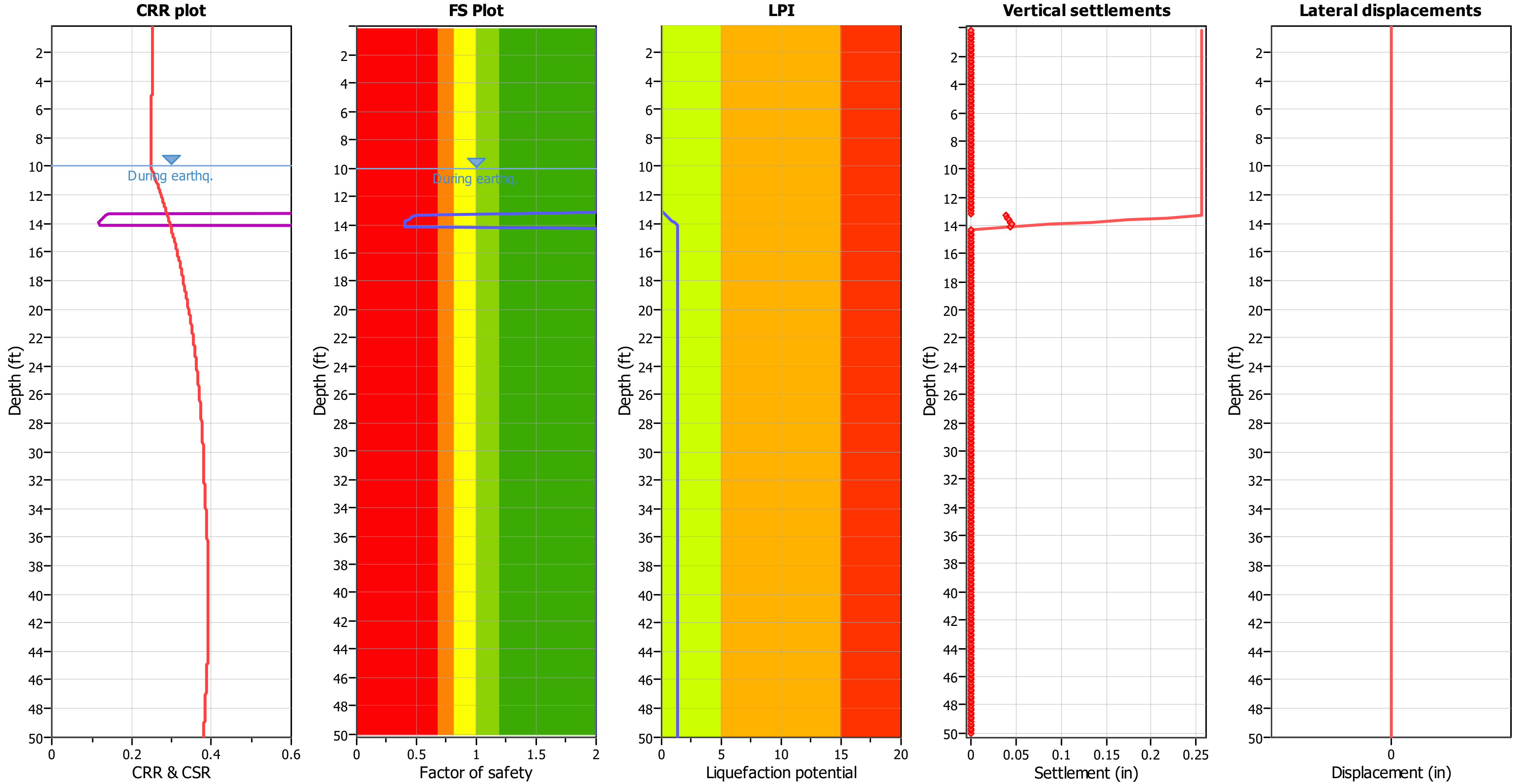

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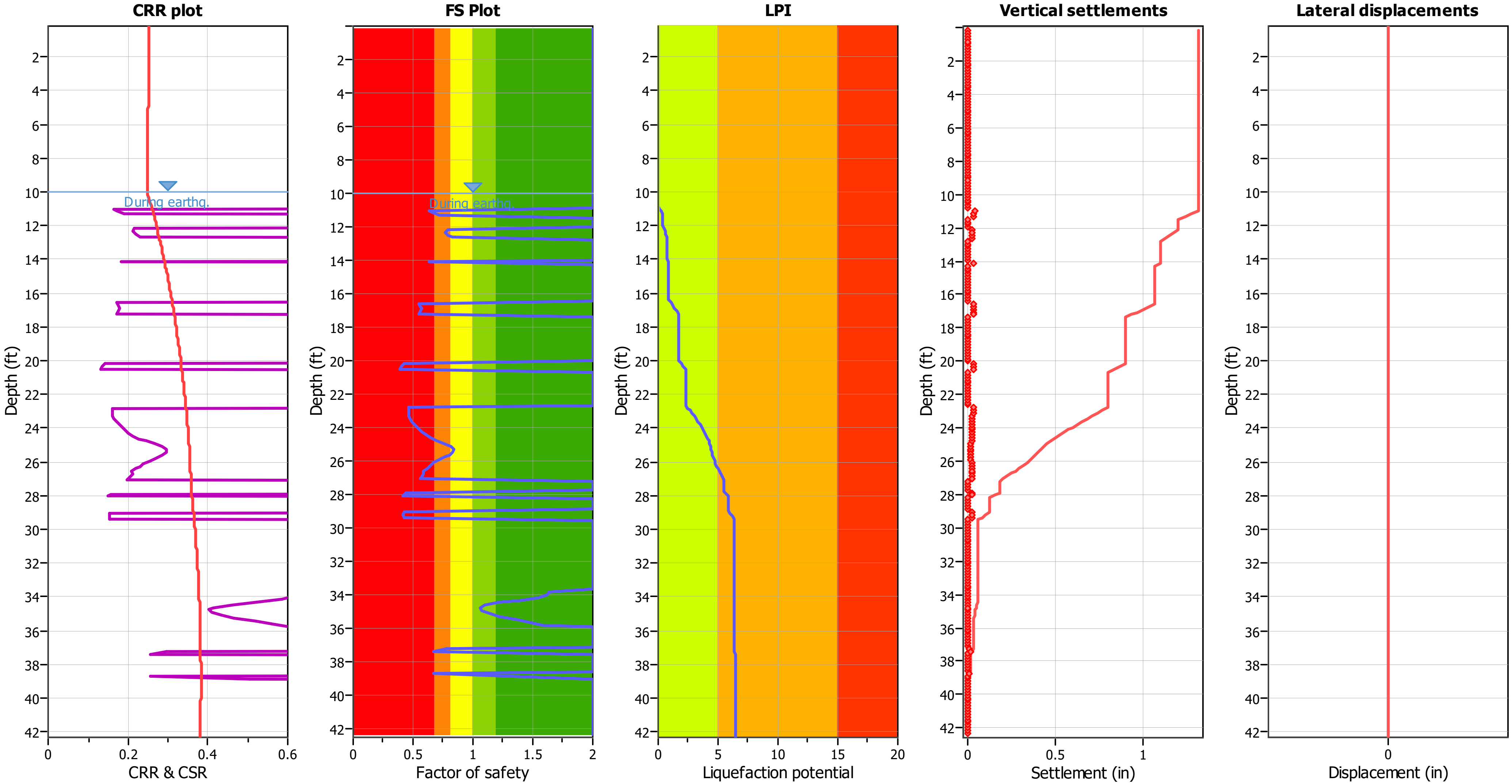

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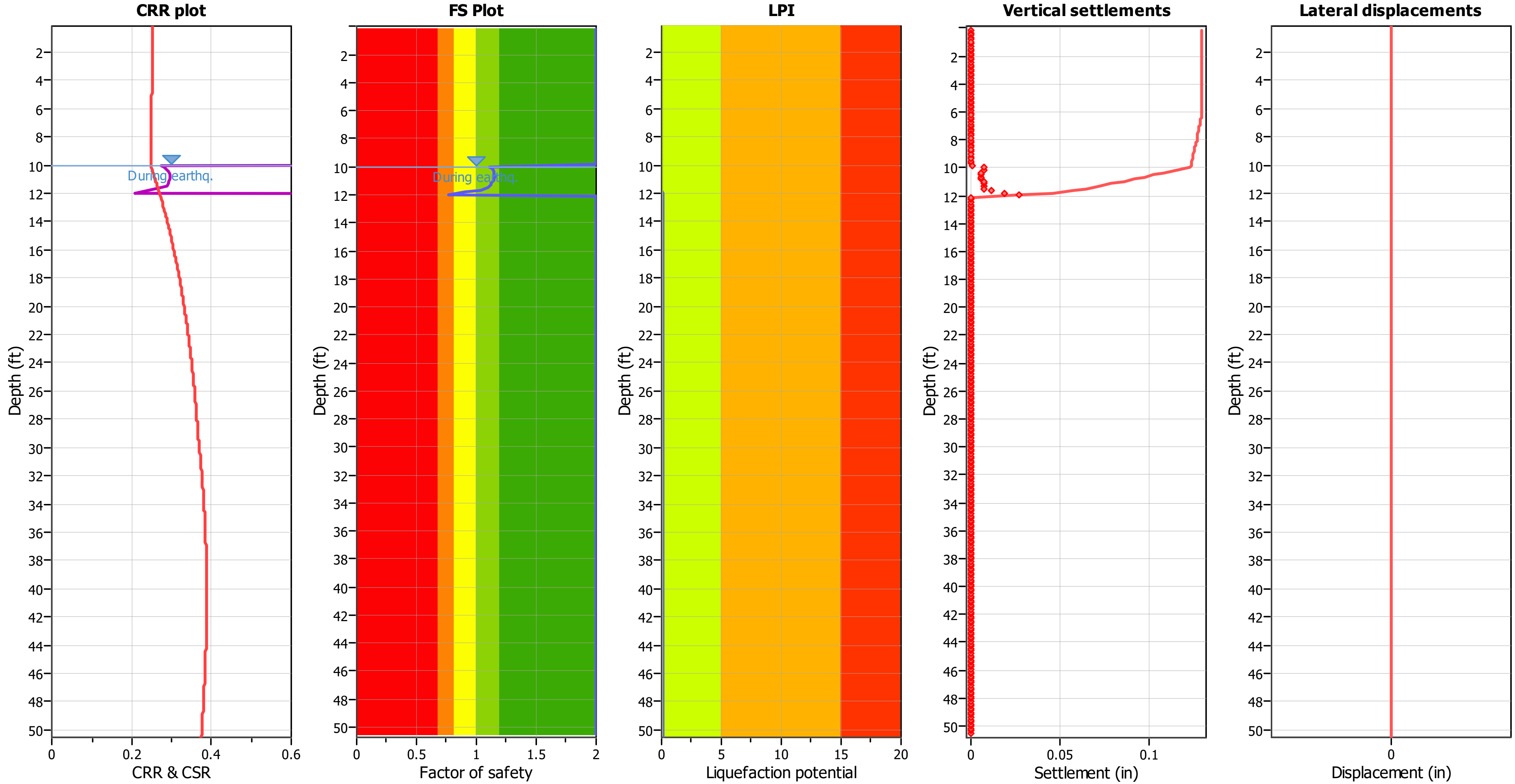

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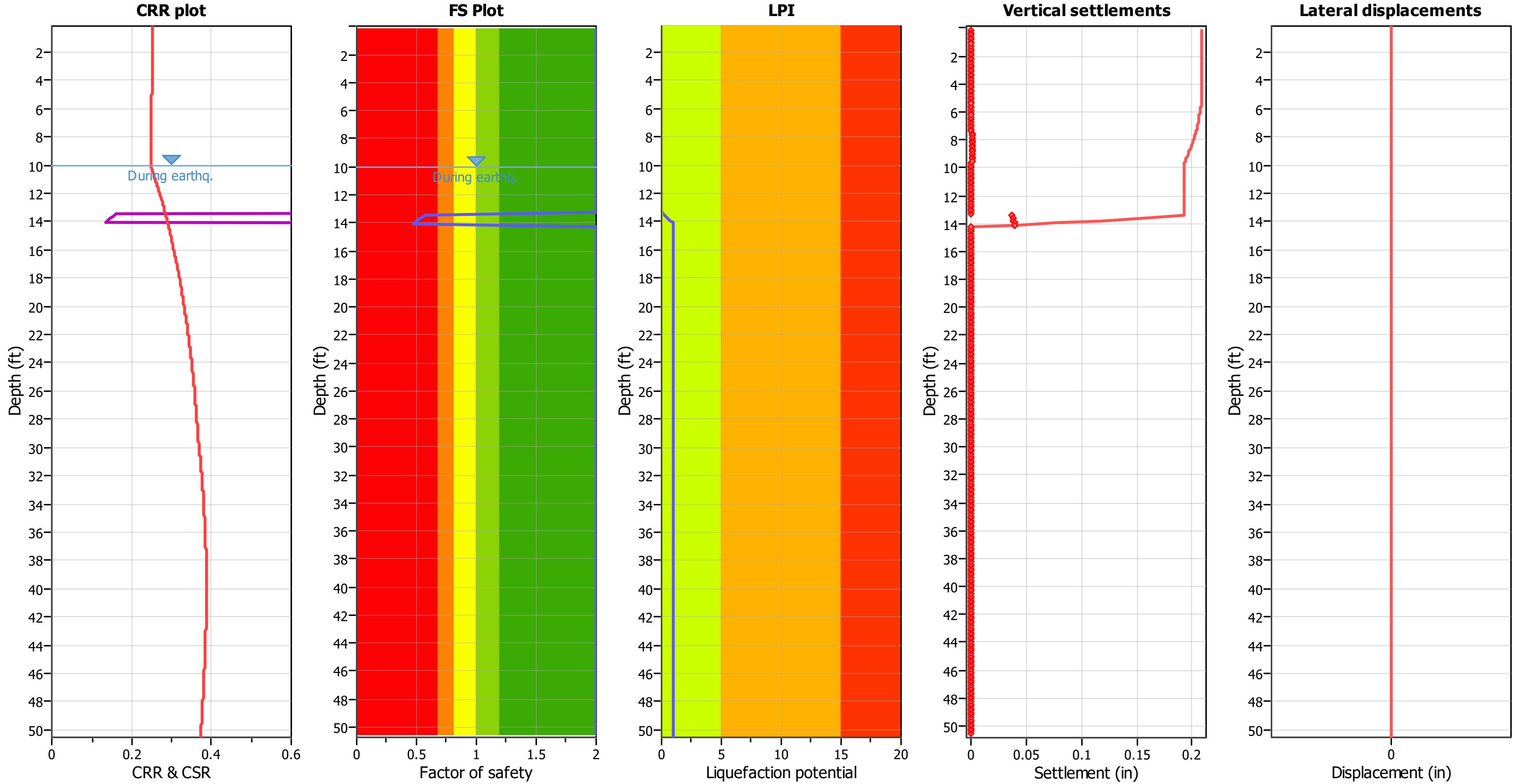

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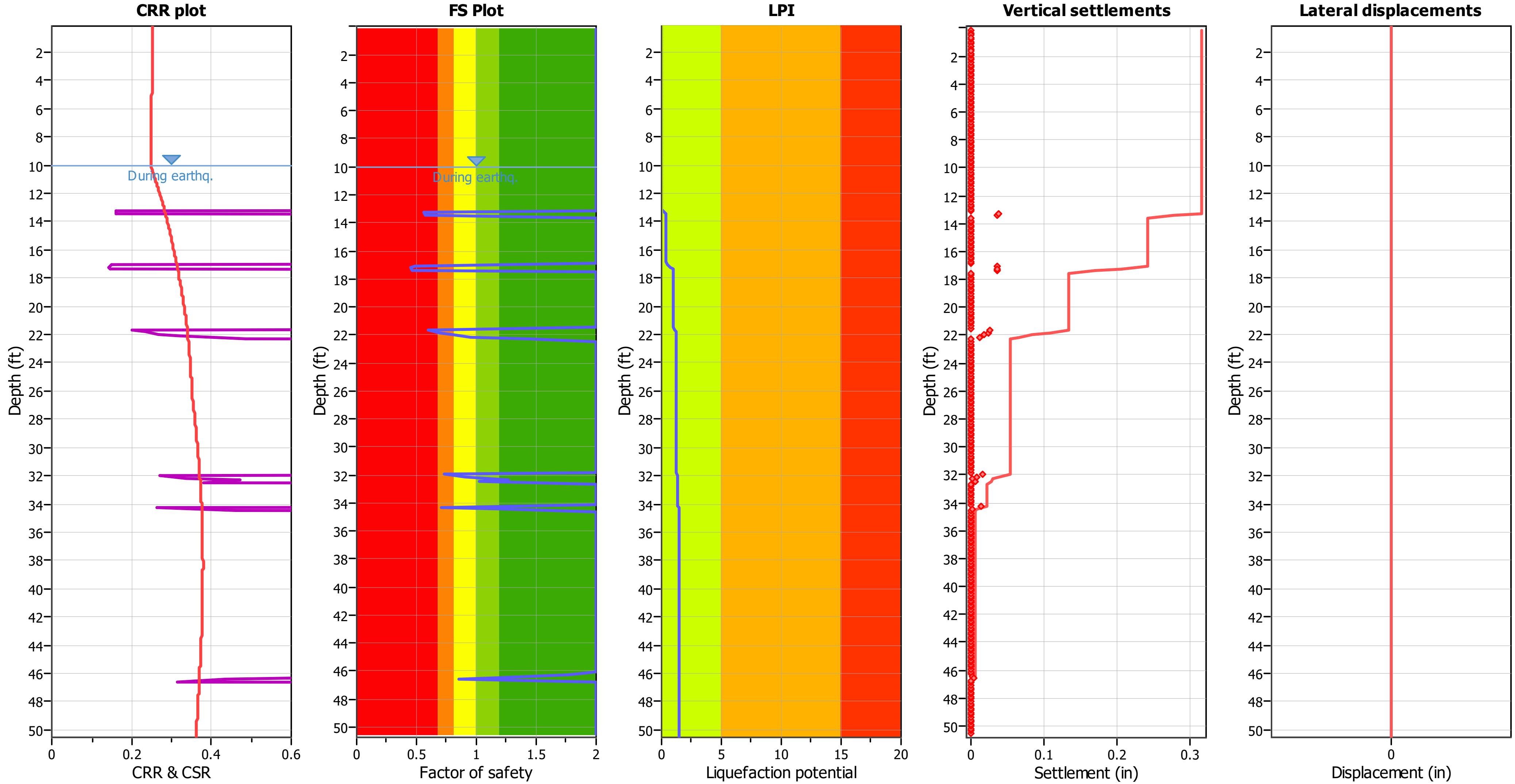

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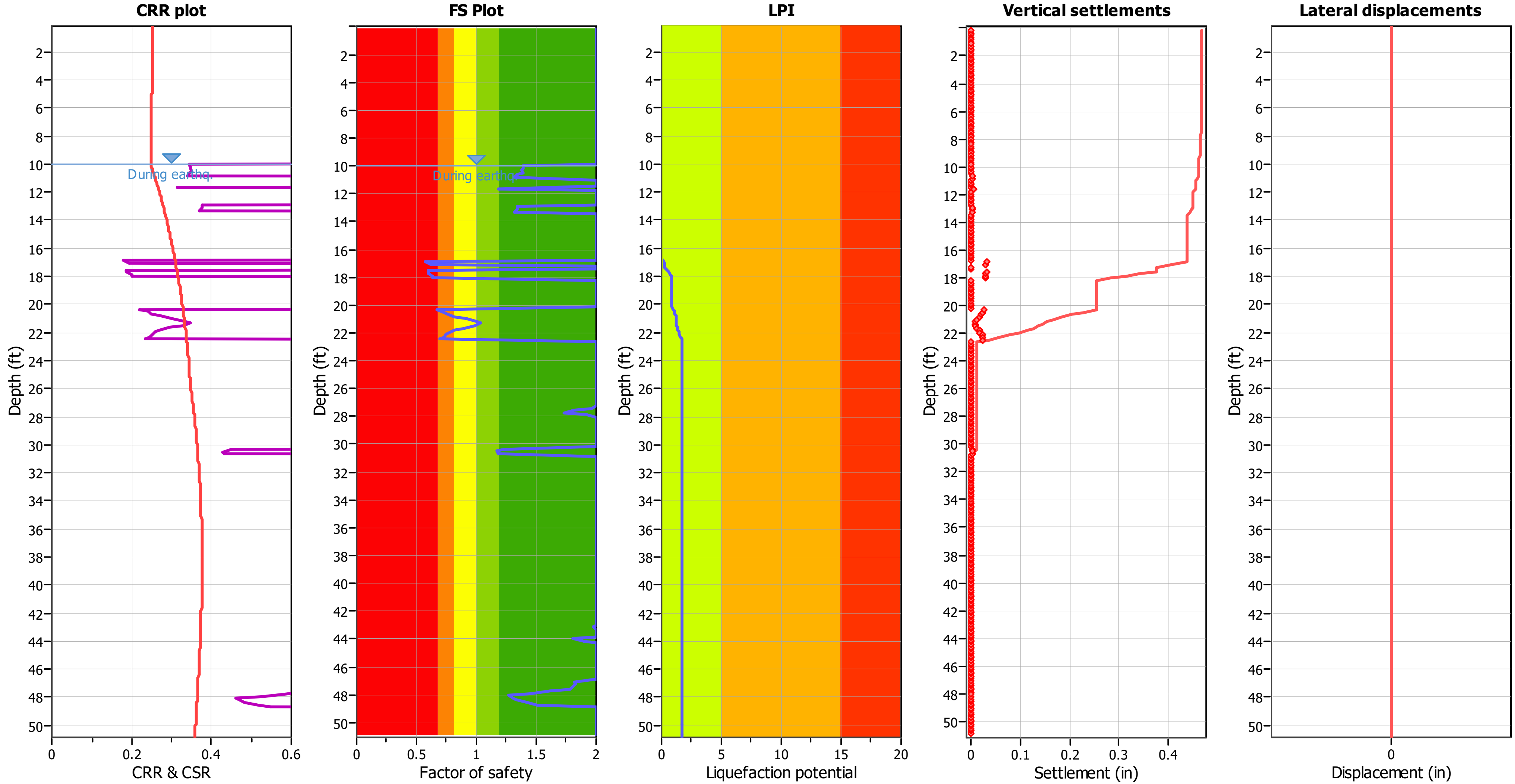

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APPENDIX E

Earthwork Grading Guide Specifications

Leighton

APPENDIX E

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

Leighton

E-1.0 GENERAL

E-1.1 Intent

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton Consulting, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton Consulting, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton Consulting, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

E-1.2 Role of Leighton Consulting, Inc.

Prior to commencement of earthwork and grading, Leighton Consulting, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton Consulting, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton Consulting, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton Consulting, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton Consulting, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

E-1.3 The Earthwork Contractor

The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Guide

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton Consulting, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton Consulting, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton Consulting, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton Consulting, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

E-2.0 PREPARATION OF AREAS TO BE FILLED

E-2.1 <u>Clearing and Grubbing</u>

Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton Consulting, Inc.. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

E-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton Consulting, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section D-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

E-2.3 Overexcavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton Consulting, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

E-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton Consulting, Inc.. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton Consulting, Inc.. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

E-2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton Consulting, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton Consulting, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

E-3.0 FILL MATERIAL

E-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton Consulting, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton Consulting, Inc. or mixed with other soils to achieve satisfactory fill material.

E-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton Consulting, Inc.. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

E-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section D-3.1, and be free of hazardous materials ("contaminants") and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton Consulting, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

E-4.0 FILL PLACEMENT AND COMPACTION

E-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section D-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton Consulting, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

E-4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D 1557.

E-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D 1557 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

E-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557 laboratory maximum density.

E-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton Consulting, Inc.. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

E-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton

Consulting, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

E-5.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton Consulting, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton Consulting, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton Consulting, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton Consulting, Inc..

E-6.0 TRENCH BACKFILLS

E-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2009 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

E-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2015 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2015 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

E-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton Consulting, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.