GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
13711 FREEWAY DRIVE
SANTA FE SPRINGS, CALIFORNIA

Prepared For

REXFORD INDUSTRIAL REALTY & MANAGEMENT, INC.
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Rexford Industrial Realty & Management, Inc.
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Attention: Mr. Daniel Murphy, Senior Associate

Subject: Geotechnical Exploration Report
Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

Per our February 4, 2022 proposal, authorized on February 8, 2022, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. We understand the proposed development will include demolition of existing site improvements to accommodate construction of a new one-story, Type III-B industrial building with a total building area of 108,000 square feet. The proposed concrete tilt-up building will be constructed at grade with dock-high truck loading on the northern side of the building. Los Angeles Fire Department access and vehicular surface parking are planned on the west, north, and east sides of the building. Ancillary improvements likely consist of utility infrastructure, pavement, flatwork, and landscaping.

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site, identify potential geologic and seismic hazards that may impact the project, and provide geotechnical recommendations for design and construction of the proposed improvements as currently planned.

Based on the results of our study, the project is considered feasible from a geotechnical standpoint. The results of our exploration, conclusions and recommendations are presented in this report.

We appreciate the opportunity to be of service to you on this project. If you have any questions or if we can be of further service, please contact us at ***(866) LEIGHTON***; or specifically at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 13711 Freeway Drive in the city of Santa Fe Springs, Los Angeles County, California. The site location (latitude 33.891187°, longitude -118.039773°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

The project site is irregular in shape and covers approximately 5 acres. The site is bordered by Freeway Drive to the south, Spring Avenue to the west, an existing industrial property to the north, and an existing railroad easement followed by an industrial property to the east. The site is currently occupied by an existing and active industrial building with asphalt concrete (AC) and Portland cement concrete (PCC) paved parking and access along the north, west and southern sides of the existing building.

The project site is relatively flat with sheet flow generally directed to the southwest across the site over paved surfaces to curbs and gutters. Review of the United States Geological Survey (USGS) 7.5-Minute Whittier Quadrangle (USGS, 1981) indicates the site is at between approximately Elevation (El.) +65 and +70 feet mean sea level (msl).

Based on review of historical aerial photographs (NETR, 2022), the site was mostly vacant land used for agricultural purposes with a small structure located in the southern portion of the site until 1963. By 1972, the small structure had been removed and the existing building was constructed. The site appears to have remained in the same configuration since 1972.

Based on review of the *Conceptual Site Plan* (Sheet A1-0) dated January 25, 2022, we understand that the proposed development will consist of a new one-story industrial building with a total building area of 108,000 square feet. The proposed concrete tilt-up building will be constructed at grade with dock-high truck loading on the northern side of the building. Los Angeles Fire Department access and vehicular surface parking are planned on the west, north, and east sides of the building. Ancillary improvements likely consist of utility infrastructure, pavement, flatwork, and landscaping.

1.2 Purpose and Scope

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site relative to the proposed development concept and provide geotechnical recommendations to aid in the design and construction for the project as currently planned. The scope of this geotechnical exploration included the following tasks:

- *Background Review* – We reviewed readily available in-house geotechnical reports, literature, aerial photographs, and maps relevant to the site. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Section 5.0.

- *Pre-Field Exploration Activities* – A site visit was performed by a member of our technical staff to mark the proposed exploration locations. Dig Alert (811) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- *Field Exploration* – Our subsurface exploration was performed on February 10 and February 21, 2022; and included advancement of rotary-wash borings and cone penetrometer test (CPT) soundings. Two (2) small-diameter (4¾-inch) rotary-wash soil borings (designated RW-1 and RW-2) were drilled, logged, and sampled to a depth of approximately 51½ feet below the existing ground surface (bgs). Three (3) CPT soundings (designated CPT-1 through CPT-3) were advanced to a depth of approximately 50 feet bgs. The approximate locations of the explorations are shown on Figure 2, *Exploration Location Map.* Logs of the borings and CPTs are presented in Appendix A, *Exploration Logs.*

  During drilling of the borings, bulk and drive samples were obtained for geotechnical laboratory testing. Driven ring samples were collected from the borings using a Modified California ring-lined sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPTs) were also performed within the borings in accordance with ASTM Test Method D 1586. Samples were collected at 5-foot intervals throughout the depth of exploration. In both test methods, the sampler is driven below the bottom of the borehole by a 140-pound weight (hammer) free-falling 30 inches. The drilling rig was equipped with an automatic hammer to provide greater consistency in the drop height and striking frequency. The number of blows to drive the sampler the final 12 inches of the 18-inch drive interval is termed the “blowcount” or SPT N-value. The N-values provide a measure of relative density in granular (noncohesive) soils and comparative consistency in cohesive soils. The number of blows per 6 inches of penetration was recorded on the boring logs, see Appendix A.

The borings were logged in the field by a geologist from our firm. 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 for testing. After completion of drilling, the borings were backfilled to the ground surface with soil cuttings and patched with cold-mix asphalt concrete at the surface to match existing conditions.

The CPT soundings were performed in accordance with ASTM D 5778 using a 15 cm^2 cone. In addition, shear wave measurements were recorded at 5-foot intervals to the total depth explored in one (1) of the CPTs to evaluate the subsurface shear wave velocity profile at the site. A near-surface (upper 5 feet) bulk soil sample was collected for geotechnical laboratory testing from the hand-auger excavation performed at the location of CPT-2. Upon completion, each CPT was backfilled with cement-bentonite to the ground surface and patched with cold-mix asphalt concrete at the surface to match existing conditions.

- *Laboratory Testing* – Laboratory tests were performed on selected soil samples obtained from the borings during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the onsite soils. Tests performed during this investigation include:

  - In-situ Moisture Content and Dry Density (ASTM D 2216 and ASTM D 2937);
  - Atterberg Limits (ASTM D 4318);
  - Direct Shear (ASTM D 3080);
  - Consolidation (ASTM D 2435);
  - Maximum Dry Density (ASTM D 1557);
  - Expansion Index (ASTM D 4829);
  - R-value (California Test Method 301); and
  - Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

Results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A. Other laboratory test results are presented in Appendix B, *Laboratory Test Results.*

- *Engineering Analysis* – The data obtained from our background review and field exploration were evaluated and analyzed to develop recommendations for the proposed development.
- *Report Preparation* – This report presents our findings, conclusions, and recommendations for the proposed development.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The site is located in the Los Angeles Basin in the northwestern portion of the Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges province extends approximately 900 miles southward from the Santa Monica Mountains to the tip of Baja California (Yerkes, et al., 1965) and is characterized by elongated, northwest-trending mountain ridges and sediment-floored valleys. The province includes numerous northwest trending fault zones, most of which either gradually truncate, merge with, or are terminated by faults that form the southern margin of the Transverse Ranges province. These northwest trending fault zones include the San Jacinto, Whittier-Elsinore, Palos Verdes, and Newport-Inglewood fault zones.

Approximately 65 million years ago (at the end of the Cretaceous Period) a deep, structural trough existed off the current coast of southern California (Yerkes, 1972). Over time, sedimentation filled the trough with hundreds to thousands of feet of sediment. About 7 million years ago, as sedimentation continued, an eastward shift of the boundary between the Pacific and North American plates to its present position would begin shaping the Los Angeles basin from this deep trough. Today the Los Angeles basin refers to the area defined by the Santa Monica, Whittier and Palos Verdes faults, and San Joaquin Hills. Basin depth is limited to the sediments deposited over the basement rock in the last 7 million years (Wright, 1991). The deepest part of the Los Angeles basin contains Tertiary to Quaternary-aged (65 million years and younger) marine and non-marine sedimentary rocks that are about 30,000 feet thick (Yerkes, et al, 1965; Wright, 1991). During the Pleistocene epoch (the last two million years) the region was flooded as sea level rose in response to the worldwide melting of the Pleistocene glaciers.

2.2 Surficial Geology

The subject site is located approximately 1,600 feet west and southwest of the concrete-lined La Canada Verde Creek at its closest point. Regional geologic mapping of the project site and vicinity indicates that near-surface native soils beneath the site consist of Quaternary-aged (Holocene) unconsolidated to slightly consolidated young alluvial fan deposits comprised of boulders, cobbles, gravel, sand and silt deposits (Bedrossian and Roffers, 2010; Dibblee Jr., 2001). The surficial geologic units mapped in the vicinity of the project site are shown on Figure 3, *Regional Geology Map.*

2.3 Subsurface Conditions

Based on our subsurface explorations, the site is underlain by a layer of undocumented artificial fill materials (Afu) overlying Quaternary-aged (Holocene) young alluvial fan deposits (Qyf). The artificial fill encountered in our borings at the explored locations is generally about 5 feet in thickness across the site, likely associated with the existing and previous site improvements. The fill soils consist primarily of locally derived clayey silt. Localized thicker accumulations of the fill materials should be anticipated between explored locations during future earthwork construction, particularly below the existing buildings.

Below the artificial fill materials, young alluvial fan deposits (Qyf) were encountered in the borings to the maximum depth explored (51.5 feet bgs). The alluvial fan deposits encountered generally consist of light brown and gray to blue gray, moist to wet, medium dense to dense, silty sand and sand, and medium stiff to hard clay, sandy clay, silty clay, silt, and sandy silt.

Detailed descriptions of the subsurface materials encountered in the borings are presented on the logs included in Appendix A. Some of the engineering properties of these soils are described in the following sections. The locations of the borings are shown on Figure 2, *Exploration Location Map*.

2.3.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which 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.

One (1) near-surface bulk soil sample obtained during our subsurface exploration was tested for expansion potential. The test result indicates an Expansion Index (EI) value of 1 (“very low” potential for expansion). The Expansion Index laboratory test results are included in Appendix B of this report.

Variance in expansion potential of onsite soil is anticipated; therefore, additional testing is recommended upon completion of site grading and excavation to confirm the expansion potential presented in this report. For purposes of this report, and based upon visual characterization of alluvial materials at approximate foundation depth, very low expansion potential of site materials may be considered to support design and verified upon completion of earthwork grading.

2.3.2 Soil Corrosivity

One (1) near-surface bulk soil sample obtained during our subsurface exploration was tested for corrosivity to assess corrosion potential to buried concrete. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix B of this report.

The test results indicate soluble sulfate concentration of 99 parts per million (ppm), chloride content of 60 ppm, pH value of 7.82, and minimum resistivity value of 3,800 ohm-cm.

The results of the resistivity test indicate the underlying soil is moderately corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil sample, concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318 (ACI, 2014). The sample tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil.

2.3.3 Soil Compressibility

Three (3) samples of the onsite soils recovered from the borings were subjected to consolidation testing to evaluate the compressibility of these materials under assumed loads representative of anticipated structural bearing stresses. The results of testing indicate these soils generally exhibit low compressibility potential. The results of testing are presented in Appendix B.

2.3.4 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 as well as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.3.5 Excavation Characteristics

Based on our subsurface explorations performed at the site and our experience from grading jobs in the vicinity of the site, we anticipate the onsite artificial fill and native earth materials can generally be excavated using conventional excavation equipment in good operating condition.

2.4 Groundwater Conditions

Groundwater was encountered in borings RW-1 and RW-2 at an approximate depth of 30 feet bgs during our subsurface exploration. Based on review of groundwater level data available through the State Water Resources Control Board's (SWRCB) GeoTracker website, groundwater was measured at about 21.4 to 44.6 feet bgs during groundwater monitoring performed at the site in 2008 and 2009.

Based on review of information available from CGS, the historically shallowest groundwater depth at the site is approximately 8 feet bgs. However, the historic high groundwater level occurred nearly 100 years ago at a time with drastically different hydrologic conditions: the rivers and creeks in the Los Angeles Basin, including the San Gabriel River, were unlined. The lining of rivers and creeks for flood control, construction of buildings and paved surfaces, and the improvement of surface drainage has significantly reduced surface infiltration. The development of groundwater from underlying aquifers resulted in lowering of the groundwater level within the aquifers and reduction of upward leakage from underlying aquifers. These changes have permanently altered the hydrologic conditions of the area, making it extremely unlikely that groundwater levels will approach the historic high levels measured prior to the lining of the rivers and creeks.

For the foreseeable future, including the design life of the proposed building at the site, most channeled rivers are likely to remain lined, buildings and paved surfaces in the general area will not be replaced with farmland, and groundwater production from the underlying aquifers will likely be controlled to maintain stable water levels necessary to prevent damage to existing structures. Therefore, it is unlikely that the groundwater level beneath the site will ever reach the historic high.

We anticipate that the groundwater level will remain deeper than 10 feet bgs during the service life of the proposed building. This level is based on a minimum 20 foot rise in groundwater at the site based on our explorations and a minimum 10 foot rise from the shallowest groundwater level previously measured in 2008 and 2009. Accordingly, we recommend a design groundwater level of 10 feet bgs.

Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture, should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff, or from stormwater infiltration.

2.5 Surface Fault Rupture

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

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, 2008). The closest active fault to the site with the potential for surface fault rupture is the Elsinore fault, located approximately 6.1 miles from the site. The San Andreas fault, which is the largest active fault in California, is approximately 38.8 miles northeast of the site on the north side of the San Gabriel Mountains. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.6 Strong Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California (Figure 4). 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.

Accordingly, design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). The 2019 edition of the California Building Code (CBC) is the current edition of the code. Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following code-based seismic parameters should be considered for design under the 2019 CBC:

Table 1 – 2019 CBC Based Ground Motion Parameters (Mapped Values)

Categorization/Coefficient	Code-Based
Site Latitude	33.891187°
Site Longitude	-118.039773°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.59 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.567 g
Short Period (0.2 sec) Site Coefficient, F_a	1
Long Period (1 sec) Site Coefficient, F_v	null[1]
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.59 g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	null[1]
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.06 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	null[1]
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.748 g

[1]Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_S to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$

2.7 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 such as lateral spreading. Depending upon the relative thickness of the liquefied strata with respect to overlying non-liquefiable soils, other potentially adverse effects such as ground oscillation and ground fissuring may occur.

As shown on the *Seismic Hazard Zones* map for the Whittier Quadrangle (CGS, 1999), the project site **is** located within a liquefaction hazard zone as identified by the State of California (Figure 5, *Seismic Hazard Map*).

As a part of this geotechnical exploration, we have evaluated the liquefaction potential at the site using the data obtained from the CPT soundings with the computer program *Cliq* (v.3.0.3.2). Our evaluation used a design groundwater level of 10 feet. Per guidelines in Los Angeles County Administrative Manual GS 045.0 (GS 045), our analysis used a peak ground acceleration (0.43g) and mean magnitude (6.7) corresponding to a hazard level of 10 percent probability of exceedance in 50 years (475-year average return period). The results indicate the potential for liquefaction to occur at the site is generally high with minor expression at the surface.

We also performed liquefaction analysis using PGA_M with its mean magnitude of 6.8. The results for PGA_M indicate the potential for liquefaction to occur at the site is high with moderate expression at the surface.

The results of our analyses are presented in Appendix C, *Liquefaction Analysis.*

2.8 Seismically-Induced Settlement

Seismically-induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event.

As a part of the liquefaction analysis, we estimated the corresponding seismically-induced ground deformations using the computer program *Cliq* (v.3.0.3.2). We considered all layers in our seismic settlement analysis. No layers were excluded in our evaluation (GS 045 allows exclusion of layers with factors of safety against liquefaction higher than 1.3).

Under existing conditions, the total seismically-induced settlement is not expected to exceed about 2 inches for peak ground acceleration corresponding to a hazard level of 10 percent probability of exceedance in 50 years. The differential seismically-induced is estimated at less than 1 inch over a horizontal distance of 40 feet.

For PGA_M, the total seismically-induced settlement is not expected to exceed about 2¼ inches with differential seismically-induced estimated at less than 1 inch over a horizontal distance of 40 feet.

The results of our analysis are presented in Appendix C.

2.9 Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since the site is relatively flat and constrained laterally, earthquake-induced lateral spreading is not considered a hazard at the site.

2.10 Earthquake-Induced Landsliding

As shown on Figure 5, the site is **not** mapped within a seismically-induced landslide hazard zone identified by the State of California (CGS, 1999). In addition, due to project site being relatively flat, it is our opinion that the potential for seismically-induced landslide hazard at the site is negligible.

2.11 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site is located within a flood hazard area identified as “Zone X”, which is defined as an area of minimal flood hazard. As shown on Figure 6, *Flood Hazard Zone Map*, the site is **not** located within a 100-

or 500-year flood hazard zone. Regionally, storm runoff flow is generally directed to the southwest.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site is **not** located within a flood impact zone from dam failure as indicated on Figure 7, *Dam Inundation Map*. Therefore, the risk of seismically-induced flooding due to dam failure is considered low.

2.12 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland location of the site, seiche and tsunami risks at the site are considered negligible.

2.13 Methane

Based on review of State of California Geologic Energy Management Division (CalGEM) records, the project site is **not** located within a documented oil field (CalGEM, 2022). The nearest oil field is the La Mirada oil field located approximately 550 feet to the east of the project site. The nearest documented oil well is located approximately 1,110 feet northwest of the site (API# 0403705641; Carmenita Lease, Well No. 1) and is reported as plugged (CalGEM, 2022). Based on these findings, methane hazard at the site is considered low.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

Based on this study, we conclude that the proposed development for the subject site is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are properly incorporated in design and construction.

The proposed structure may be supported on a mat-type foundation system established on engineered fill soils. There may be existing underground utilities that may be impacted by the planned development. Information on these utilities should be provided to Leighton for evaluation. Alternatively, the building may be supported on pile foundations or spread footings over improved ground. Ground improvement may consist of drilled displacement columns or displacement rammed aggregate piers down to about 35 to 40 feet bgs.

All existing undocumented fill is recommended to be removed from below the proposed building pad and other structural improvements prior to placement of engineered fill. We estimate removal and recompaction of existing undocumented fill materials will be on the order of approximately 5 feet bgs. Localized areas in the unexplored portions of the site should be anticipated to require deeper removals. Removals should be performed such that all undocumented fill is removed and replaced as engineered fill beneath the proposed building footprint. In addition, overexcavation should be performed so that a minimum of 3 feet of engineered fill is established below the proposed foundation elements. Based on our explorations performed at the site, there is a potential that overexcavations may extend into soils with moisture content significantly over optimum. As such, the excavation bottoms may require appropriate grading techniques to stabilize the areas for fill placement

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to determine the effect upon the recommendations subsequently presented. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the City of Santa Fe Springs, the County of Los Angeles and other governing agencies.

Leighton should review the grading plans, foundation plans and project specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans for this project.

3.1 Site Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. Earthwork for the project is expected to include overexcavation and recompaction of existing fill soils below new improvement footprints. We recommend that earthwork on the site be performed in accordance with the recommendations presented in this report and the project specifications as prepared by others. The *Earthwork and Grading Guide Specifications* included in Appendix D may be used for guidance in developing the project specifications. If conflict arises, the recommendations in Appendix D shall be superseded by the project specifications, recommendations contained in this report and/or the City of Santa Fe Springs requirements, whichever is more stringent. Leighton should review the final grading and foundation plans when it becomes available to verify the recommendations in this report have been incorporated.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, former foundation remnants and/or debris within the area of proposed grading. These materials should be removed from the site. Any underground obstructions onsite should be removed. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits.

3.1.2 Removals and Overexcavations

To provide uniform foundation support and reduce the potential for excessive static settlement, all existing undocumented fill and any unsuitable alluvial soil, as deemed by the geotechnical engineer, should be removed to expose suitable native soils and replaced as engineered fill below the proposed building pad and other structural improvements. Based on our field explorations, we estimate removals of existing undocumented fill will be on the order of approximately 5 feet bgs across most of the site. Unexplored portions of the site, including areas beneath existing buildings, in areas of existing utilities, and areas disturbed during demolition of existing buildings and improvements may also require deeper removals. Deeper removals in localized areas may be recommended during grading by a representative of the geotechnical engineer depending on observed subsurface conditions.

In addition, overexcavations should be performed such that a minimum of 3 feet of engineered fill is established below the proposed building foundation elements. The lateral extent of overexcavation beyond foundations should be equal to the depth of overexcavation below the proposed foundations.

Care must be used and precautions implemented in performing earthwork and grading operations along the property lines. It is essential that excavation not undermine existing adjacent improvements. Overexcavation performed along property lines that may extend to depths greater than 4 feet below grade are recommended to be properly shored or performed using slot-cutting techniques to reduce the potential for adversely affecting the adjacent improvements.

The depth of overexcavation in non-structural areas planned for new pavement construction is recommended to be 2 feet below the current grade or planned subgrade elevation to develop a suitable bearing subgrade for pavement support. Deeper overexcavations in localized areas may be recommended during grading by a representative of the geotechnical engineer depending on observed subsurface conditions. Preparation limited to 2 feet of overexcavation below subgrade may result in the need for increased pavement maintenance and periodic repairs where existing undocumented fill is left in place below the recommended overexcavation depth of 2 feet.

3.1.3 Excavation Bottom Preparation

All excavation bottoms or removal bottoms should be observed by a representative of the geotechnical engineer prior to placement of fill or other improvements to determine that geotechnically suitable soil is exposed. Excavation bottoms observed to be suitable for fill placement or other improvements should be scarified to a depth of at least 8 inches, moisture-conditioned as necessary to achieve a moisture content within 2 percentage points of the optimum moisture content, and then compacted to a minimum of 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor).

Removals and overexcavations performed at the site may extend into soils with moisture content significantly over optimum. Therefore, if necessary, the excavation bottoms can then be stabilized with geogrid or crushed rock to reduce the potential for pumping and provide a firm working surface for heavy equipment. If a rock or gravel layer is placed, a layer of nonwoven filter fabric such as Mirafi 140N or equivalent, should be placed over the gravel/rock layer to reduce the potential for migration of sediments into the void space between the coarse aggregate. Once stabilized, the excavation can then be backfilled with excavated materials and placed as engineered fill.

3.1.4 Fill Materials

On-site soil that is free of construction debris, organics, cobbles, boulders, rubble, or rock larger than 4-inches in largest dimension is suitable to be used as fill for support of structures. Natural soils encountered onsite below existing fill consist predominantly of very moist to wet soils that will require substantial drying and processing for use as engineered fill. Any imported fill soil should be approved by the geotechnical engineer prior to import or use onsite.

3.1.5 Fill Placement and Compaction

Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to within 2 percent of optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

When grading is interrupted by heavy rains, fill operations should not be resumed until the moisture content and the dry density of the placed fill are satisfactory.

3.1.6 Shrinkage

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.

Based upon the results of the in-place density and the moisture-density relationship exhibited by representative bulk samples of the near surface soils, recompaction of the soils is anticipated to result in volume shrinkage in the range of 10 to 15 percent. The estimated shrinkage does not include material losses due to removal of organic material or other unsuitable bearing materials (debris, rubble, oversize material greater than 6-inches) and the actual shrinkage that occurs during grading may vary throughout the site.

3.1.7 Reuse of Concrete and Asphalt Rubble

If encountered during site clearing and/or during preparation activities, construction rubble (i.e., Portland cement concrete and asphalt concrete) may be incorporated in the proposed development. For use as structural fill, the processed material should be crushed to develop a relatively well-graded mixture with a maximum particle size of 3-inch nominal diameter. Concrete rubble should be free of rebar and processed asphalt pavement rubble may be used if mixed with the existing base course (where present). Processed material may be used as structural fill if uniformly mixed with onsite soils in proportion of 1 part processed material to 3 parts soil. For use as pavement base course, rubble should be crushed to satisfy gradation requirements of Section 200-2.4 of the Standard Specifications for Public Works Construction (SSPWC). Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

3.2 Foundation Design

We anticipate a mat-type foundation system established on engineered fill will be required for the proposed building to accommodate the estimated seismic settlement at the site and the potential settlement of the soft clay soils that exist at depth. It may be feasible to use spread footings with foundation ties if the anticipated settlement due to gravity loads and seismic loading can be accommodated.

Alternatively, the building may be supported on conventional spread footings over improved ground. The ground improvement system should be designed by a specialty contractor specializing in design and construction of ground improvement techniques. Feasible alternatives for ground improvement at this site that may be considered are Geopiers® or rammed aggregate piers, drilled displacement columns, and stone columns. The performance target for ground improvement is an allowable bearing capacity of at least 5,000 pounds per square foot (psf) and a reduction in total static plus seismically-induced settlement of less than 1½ inches.

3.2.1 Shallow Foundations

Mat-type foundations or spread footings with foundation ties may be designed using an allowable bearing capacity 1,500 pounds per square foot (psf) and a modulus of subgrade reaction of 10 pounds per cubic inch (pci).

The total settlement due to the static and seismic loads is expected to be on the order of 3 inches. Differential settlement of the mat foundation due to the static and seismic loads is expected to be on the order of 1½ inches over a distance of 40 feet. The bearing capacity may be increased by one-third for wind or seismic loading. The spread footings or perimeter of the mat foundation should have a minimum embedment of 24 inches below the lowest adjacent grade.

The ultimate bearing capacity can be taken as 4,500 psf, which does not incorporate a factor of safety. A resistance factor of 0.45 should be used for initial bearing capacity evaluation with factored loads. The recommended bearing values are net values, and the weight of concrete in the mat foundation can be taken as 50 pcf; the weight of soil backfill can be neglected when determining the downward loads.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings. For calculating lateral resistance above the design groundwater at 10 feet bgs, a passive pressure of 250 pcf and a frictional coefficient of 0.30 may be used. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

3.2.2 Conventional Spread Footings Over Improved Ground

Footings should be embedded a minimum 18 inches below the lowest adjacent grade. An allowable soil bearing pressure of 5,000 psf may be used for footings with a minimum width of 12 inches for continuous footings and 18 inches for isolated footings. A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used. The ultimate bearing capacity can be taken as 15,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.45 should be used for initial bearing capacity evaluation with factored loads.

The recommended bearing values are net values, 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

The allowable bearing capacity for shallow footings is based on a total static and seismic settlement of 1 inch. Differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. Leighton should review the settlement estimates when final foundation plans and loads for the proposed structures become available.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance above the design groundwater at 10 feet bgs, a passive pressure of 300 pcf and a frictional coefficient of 0.30 may be used. Below groundwater, the passive resistance should be reduced to 200 pcf to a maximum of 3,000 psf. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

3.3 Slabs-on-Grade

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 3.1. 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.

3.4 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with 2019 CBC requirements. However, concrete exposed to recycled water should be designed using Type V cement.

Based on our laboratory testing, the onsite soil is considered moderately corrosive to ferrous metals. Ferrous pipe should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Ferrous pipe, if used, should be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from onsite soils.

3.5 Retaining Walls

Recommended lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft. or pcf. These values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Onsite soils are likely suitable to be used as retaining wall backfill due to its very low expansion potential; however, field and laboratory verification are recommended before use. Should site soil be considered for reuse behind retaining walls, it should be tested to ensure Expansion potential is less than 20 (EI<20). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 9, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 2 – Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	40
At-Rest (braced)	60
Passive Resistance (compacted fill)	250
Seismic Increment (add to active pressure)	20

Walls that are free to rotate or deflect may be designed using active earth pressure. For basement walls or walls that are fixed against rotation, the at-rest pressure should be used. For seismic condition, the pressure should be distributed as an inverted triangular distribution and the dynamic thrust should be applied at a height of 0.6H above the base of the wall.

3.5.1 Sliding and Overturning

Total depth of retained earth for design of walls and for uplift resistance, should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, if drained, or 60 pcf if submerged, for properly compacted backfill.

3.5.2 Drainage

Adequate drainage may be provided by a subdrain system positioned behind the walls. Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill, particularly where horizontal space is limited adjacent to shoring (where walls are cast against shoring). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

3.6 Paving

To provide support for paving, the subgrade soils should be prepared as recommended in the Section 3.1. Compaction of the subgrade, including trench backfills, to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. Landscape areas must be separated from pavements with concrete curbs and/or edge drains. Excessive over-irrigation will have an adverse impact on adjacent pavements. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from paving, will result in premature pavement failure.

3.6.1 Asphalt Concrete

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 engineered fill with an R-value greater than 50, compacted to at least 90 percent as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on one (1) near surface sample of existing onsite soils indicate a value of 63.

Table 3 – Asphalt Concrete Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5	3	4
6	3	6
7	4	6
8	4	8
9	5	8

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.

3.6.2 Portland Cement Concrete Paving

We have assumed that the subgrade below paving will have an R-value of at least 50. Portland cement concrete (PCC) paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete 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 4 – PCC Pavement Sections

Traffic Index	PCC (inches)	Base Course (inches)
5	5	4
6	5½	4
7	6	4
8	7	4
9	8	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 and a 4-inch-thick aggregate base course layer under paving may be added to reduce cracking and to prolong the life of the paving.

3.6.3 Base Course

The base course for both asphalt concrete and Portland cement concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D 1557.

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at

45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements, Article 6, Section 1541, effective October 1, 1995. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Near-surface onsite soils are to be considered Type B soils.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.8 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the Standard Specifications for Public Works Construction, ("Greenbook"), 2021 Edition. Utility trenches can be backfilled with onsite sandy 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) **Sand:** A uniform, sand material that has 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"), 2021 Edition. CLSM should not be jetted.

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 D 1557 laboratory maximum density.

3.9 Drainage and Landscaping

Building walls below grade should be waterproofed or at least damp proofed, depending upon the degree of moisture protection desired. Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.10 Additional Geotechnical Services

Leighton should review the grading plans, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated. In addition, should stormwater infiltration be considered for the project, we recommend additional testing be performed at the specific location and depth of the planned infiltration device to confirm that infiltration will be feasible due to the high variability in test results.

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

- Grading and excavation of the site;
- Installation of ground improvement (if implemented);
- 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.

4.0 LIMITATIONS

This geotechnical exploration does not address the potential for encountering hazardous soil at this site. In addition, this report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is, by necessity, incomplete. Please also refer GBA's *Important Information About Your Geotechnical Report* (included at the rear of the text), presenting additional information and limitations regarding geotechnical engineering studies and reports. 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 Consulting, Inc. has the opportunity to observe subsurface conditions during grading and construction, to confirm that our data are representative for the site. Leighton Consulting, Inc. 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 at this time in Los Angeles County. We do not make any warranty, either expressed or implied.

5.0 REFERENCES

American Concrete Institute (ACI), 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, an ACI Standard, reported by ACI Committee 318.

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, with Supplement 1, Effective December 12, 2018.

Bedrossian, T.L., and Roffers, P.D., 2010, Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, California Geological Survey (CGS) Special Report 217, Plate 12, map scale 1:100,000.

Bryant, W.A. and Hart, E.W., Interim Revision 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Geological Survey, Special Publications 42, 42p.

California Building Standards Commission, 2019, 2019 California Building Code (CBC), California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on 2018 International Building Code, Effective January 1, 2020.

California Department of Water Resources (DWR), 2022, Interactive Website, Water Data Library, http://wdl.water.ca.gov/waterdatalibrary/index.cfm

California Department of Conservation, Geologic Energy Management Division (CalGEM), 2021, Interactive Wellfinder Website, https://maps.conservation.ca.gov/doggr/wellfinder/

California Geological Survey (CGS; formerly California Division of Mines and Geology, CDMG), 1998, Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report No. 037.

_____, 1999, Seismic Hazard Zones, Whittier Quadrangle, map scale 1:24,000, Official Map, released March 25, 1999.

______, 2000, CD-ROM containing digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones that affect the Southern Region, DMG CD 2000-003 2000.

______, 2002, Note 49, Guidelines for Evaluating the Hazard of Surface Fault Rupture, dated May 2002.

______, 2008, Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California; originally adopted March 13, 1997 by the State Mining and Geology Board in Accordance with the Seismic Hazards Mapping Act of 1990, Revised and Re-Adopted September 11, 2008.

______, 2010, Fault Activity Map of California, 2010.

______, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42.

California State Water Resources Control Board (CSWRCB), GeoTracker, http://geotracker.waterboards.ca.gov/.

Dibblee, Jr., T.W., 2001, Geologic Map of the Whittier and La Habra Quadrangles (Western Puente Hills), Los Angeles and Orange Counties, California, Dibblee Geologic Foundation Map #DF-74, map scale 1:24,000.

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map, Los Angeles County and Incorporated Areas, Map Number 06037C1843F, dated September 26, 2008.

Jennings, C.W., 2010, Fault Activity Map of California, California Geological Survey, Geologic Data Map No. 6, map scale 1:750,000.

Nationwide Environmental Title Research, LLC (NETR), 2022, Historic Aerials by NETR Online, website: http://www.historicaerials.com/.

Roffers, P.D., and Bedrossian, T.L., 2010, Geologic Compilation of Quaternary Surficial Deposits in Southern California, Onshore Portion of the Long Beach 30’x60’ Quadrangle, California Geological Survey (CGS) Special Report 217, Plate 8, Map Scale 1:100,000.

Saucedo, G.J., Greene, H.G., Kennedy, M.P., Bezore, S.P, 2003, Geologic Map of the Long Beach 30’x60’ Quadrangle, California, Version 1.0, California Geological Survey, Regional Geologic Map Series, Map No. 5, Scale 1:100,000.

United States Geological Survey (USGS), 1965 (Photorevised 1981), Whittier 7.5 Minute Series Quadrangle, California, map scale 1:24,000.

______, 2008, National Seismic Hazard Maps – Fault Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

______, 2022a, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/

______, 2022b, Interactive Geologic Map, http://ngmdb.usgs.gov/maps/MapView/

Wright, T.L., 1991, Structural Geology and Tectonic Evolution if the Los Angeles Basin, California, in Biddle, K.T. (editor), Active Margin Basins: American Association of Petroleum Geologists Memoir 52, pp. 35-134.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E. and Vedder, J.G., 1965, Geology of the Los Angeles Basin, California -- An Introduction: U. S. Geological Survey Professional Paper 420-A, 57 p.

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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

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

- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.*

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain 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 judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

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, the engineer's services were not designed, conducted, or intended to prevent 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

FIGURES

Project: 13429.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: February 2022

Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

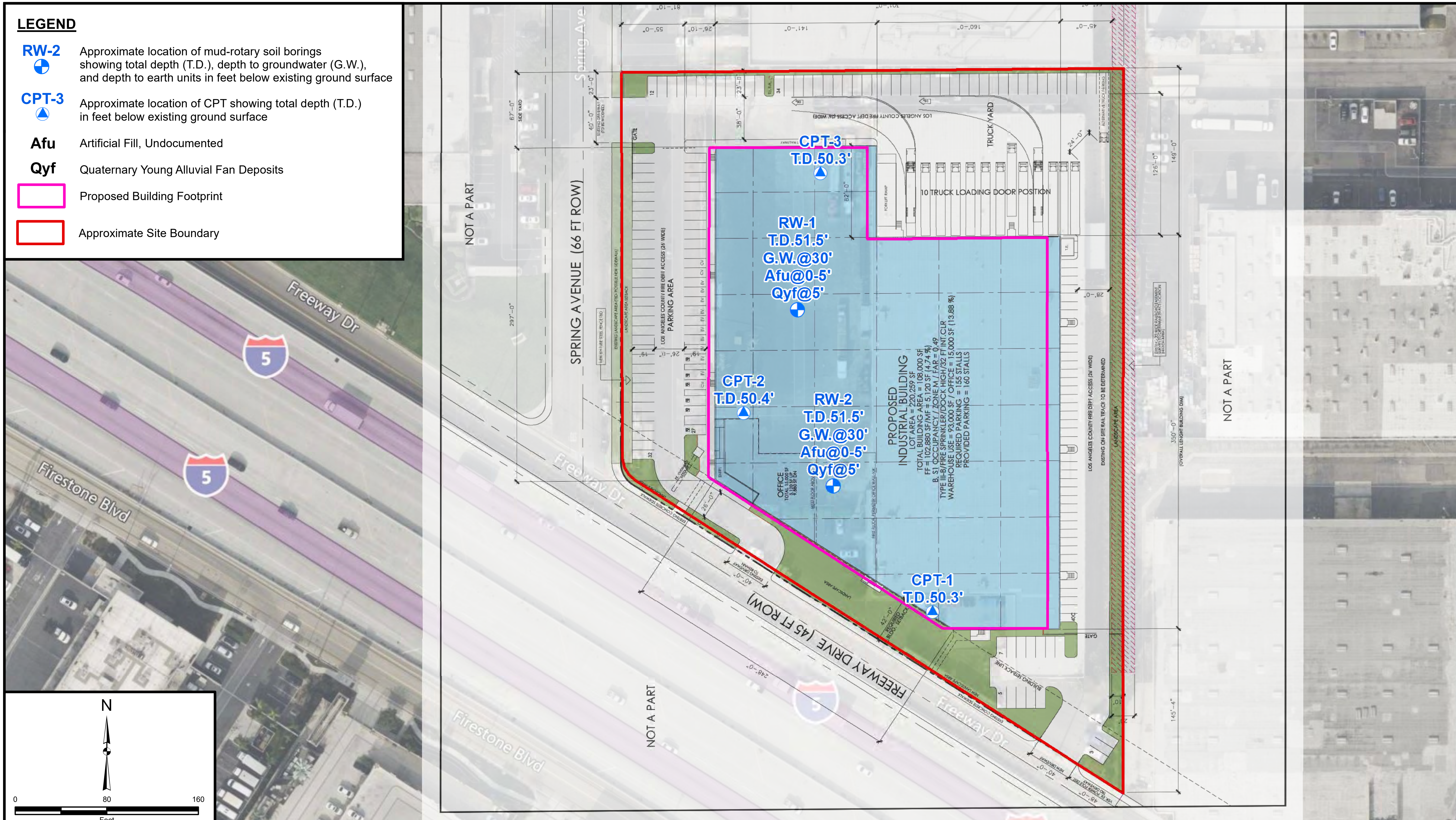
SITE LOCATION MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F01_SLM_2022-02-24.mxd on 2/23/2022 10:42:11 AM Author: KVM (btran)



Project: 13429.001	Eng/Geol: CCK/JMP
Scale: 1 " = 80 '	Date: February 2022

Base Map: Conceptual Site Plan, Sheet A1-0
by Reford Industrial Realty, Dated: October 20, 2021

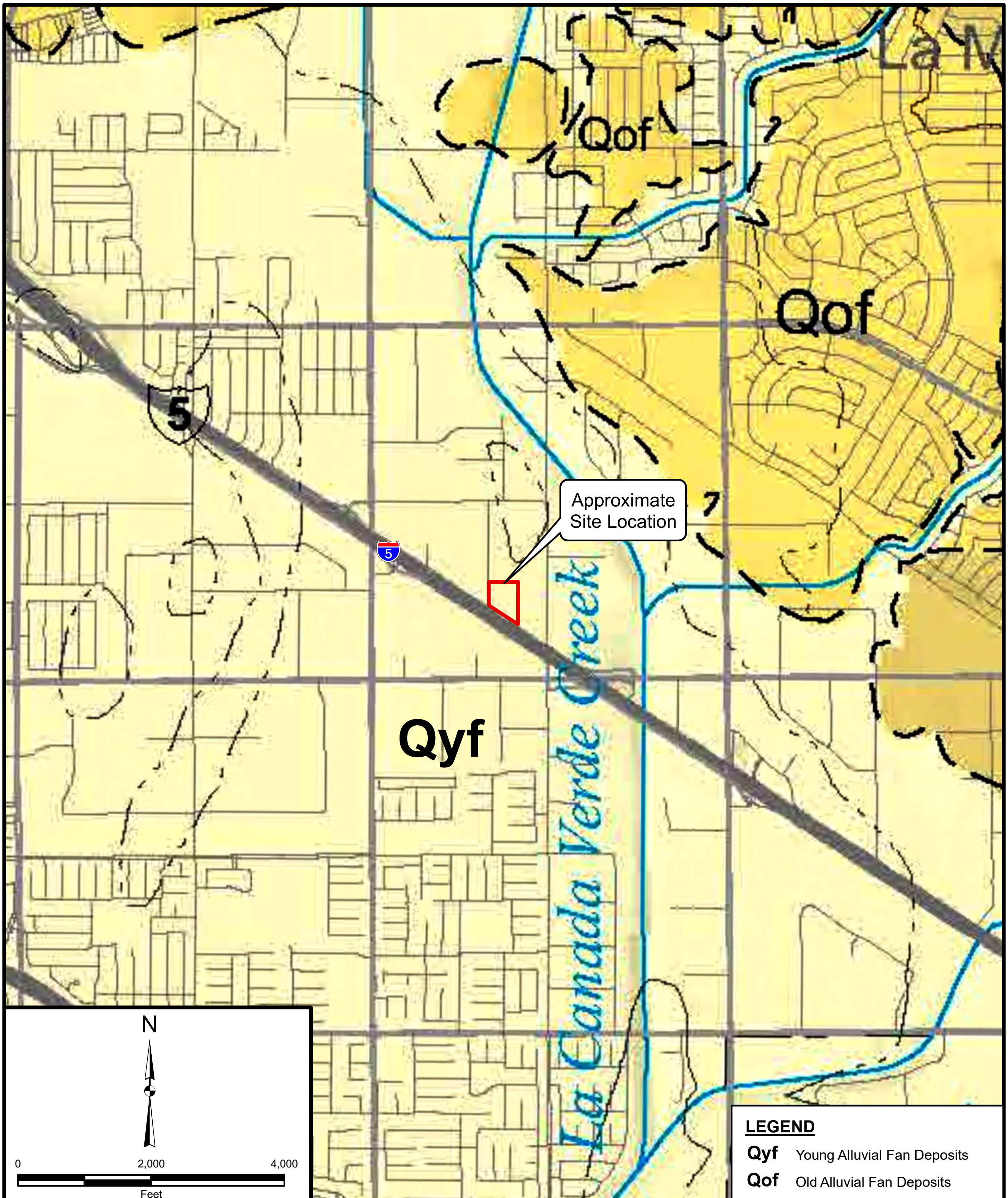
EXPLORATION LOCATION MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F02_ELM_2022-02-24.mxd on 2/23/2022 10:41:14 AM Author: KVM (btran)



Project: 13429.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: February 2022

Basemap: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, CGS SR217 Plate 12, by Bedrossian, T.L. and Roffers, P.D., July 2010

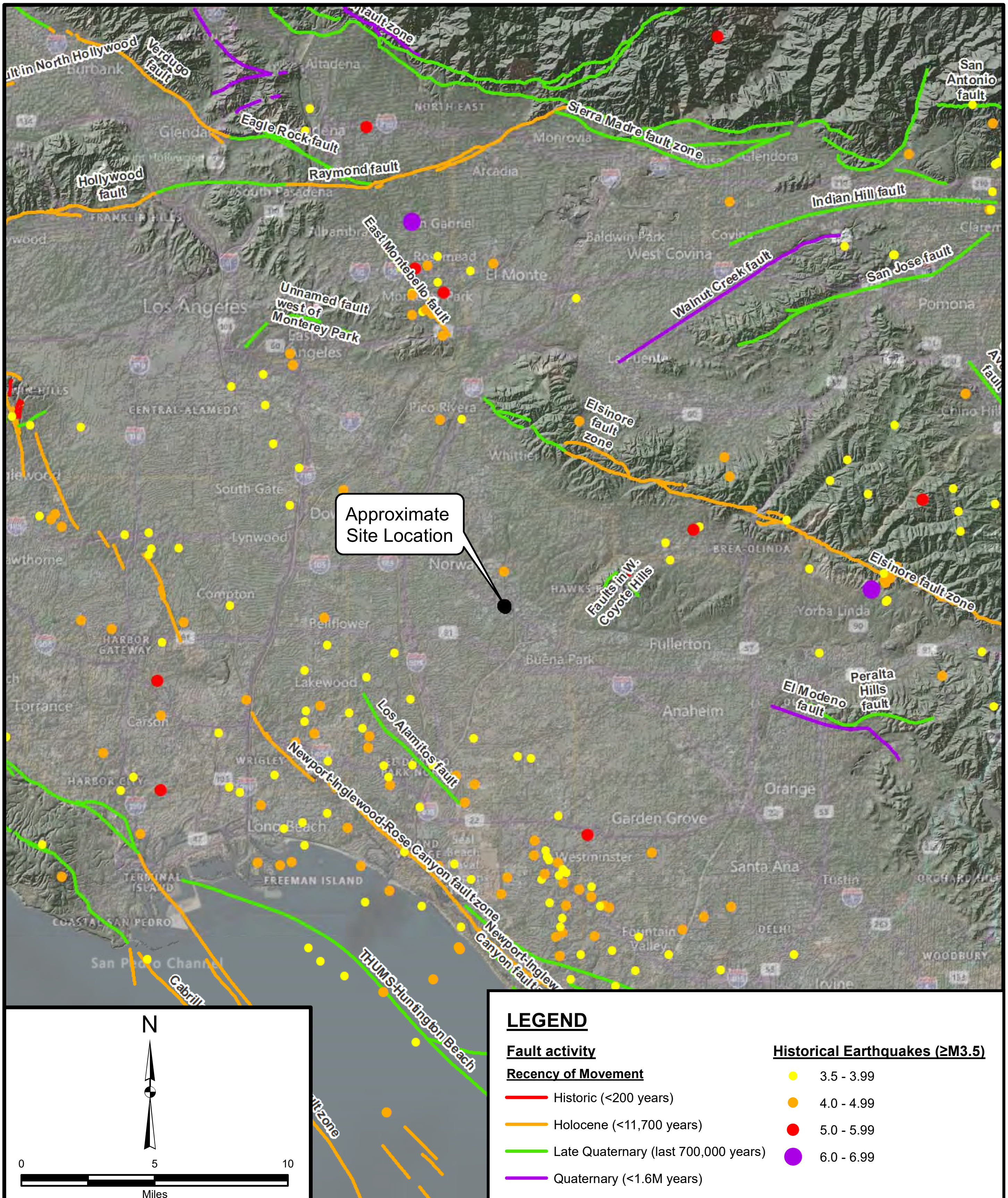
REGIONAL GEOLOGY MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 3

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F03_RGM_2022-02-24.mxd on 2/23/2022 10:54:44 AM Author: KVM (btran)



Project: 13429.001	Eng/Geol: CCK/JMP
Scale: 1 " = 5 miles	Date: February 2022

Base Map: ESRI ArcGIS Online 2022
Reference: maps.conservation.ca.gov

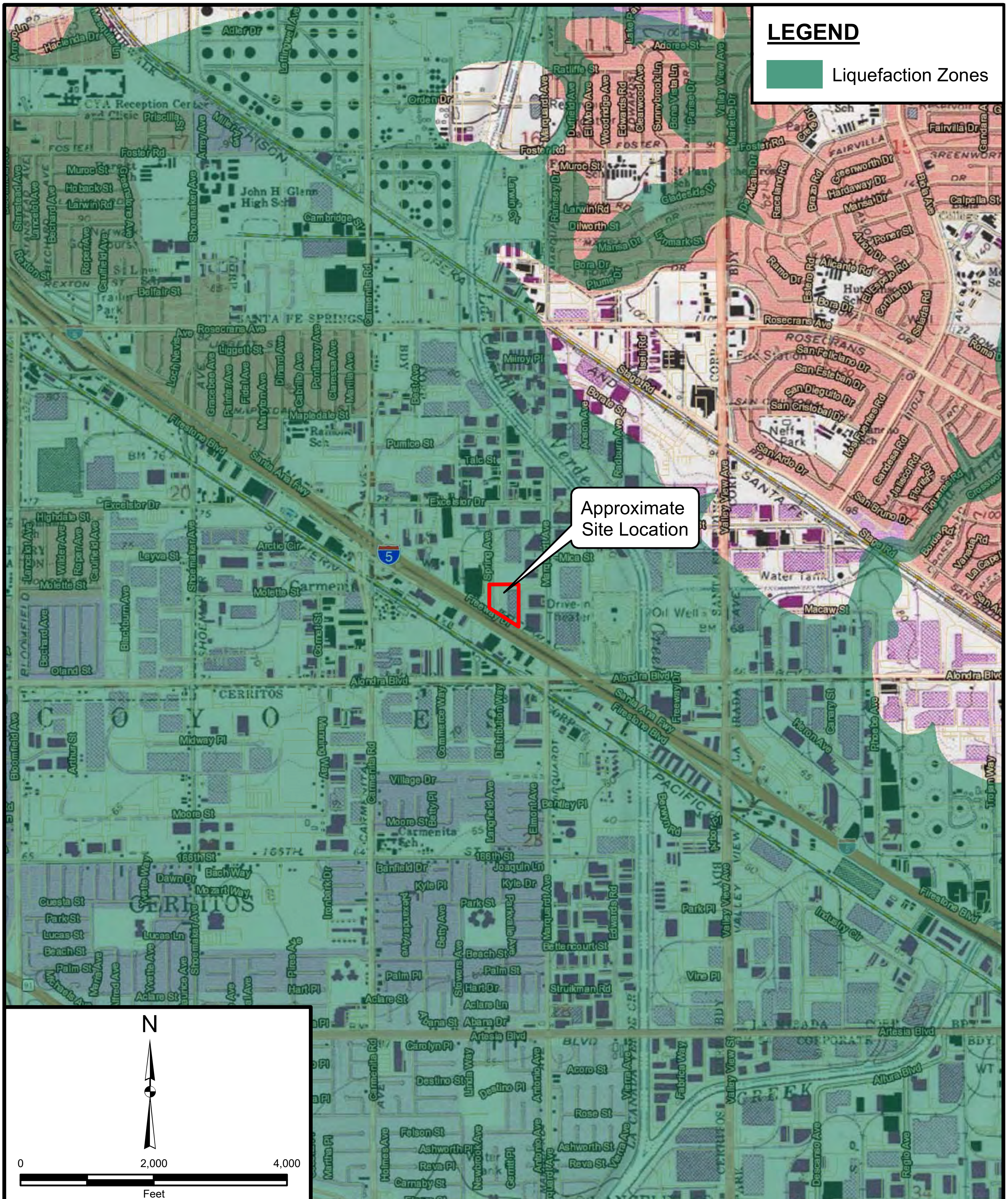
REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 4

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F04_RFHSM_2022-02-24.mxd on 2/23/2022 10:43:26 AM Author: KVM (btran)



Project: 13429.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: February 2022

Base Map: Esri, HERE, Garmin, (c) OpenStreetMap contributors
Seismic Hazards Program, California Geological Survey,

SEISMIC HAZARD MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 5

Leighton

Project: 13429.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: February 2022

Base Map: ESRI ArcGIS Online 2022
Reference: FEMA, DWR

FLOOD HAZARD ZONE MAP

Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 6

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F06_FHM_2022-02-24.mxd on 2/23/2022 10:46:35 AM Author: KVM (btran)

Project: 13429.001 | Eng/Geol: CCK/JMP

Scale:1 " = 2,000 ' | Date: February 2022

Base Map: ESRI ArcGIS Online 2022
Reference: Office of Emergency Services (2007),
Dept of Safety of Dams (2021)
National Inventory of Dams, Army Corps of Engrs (2021)

DAM INUNDATION MAP

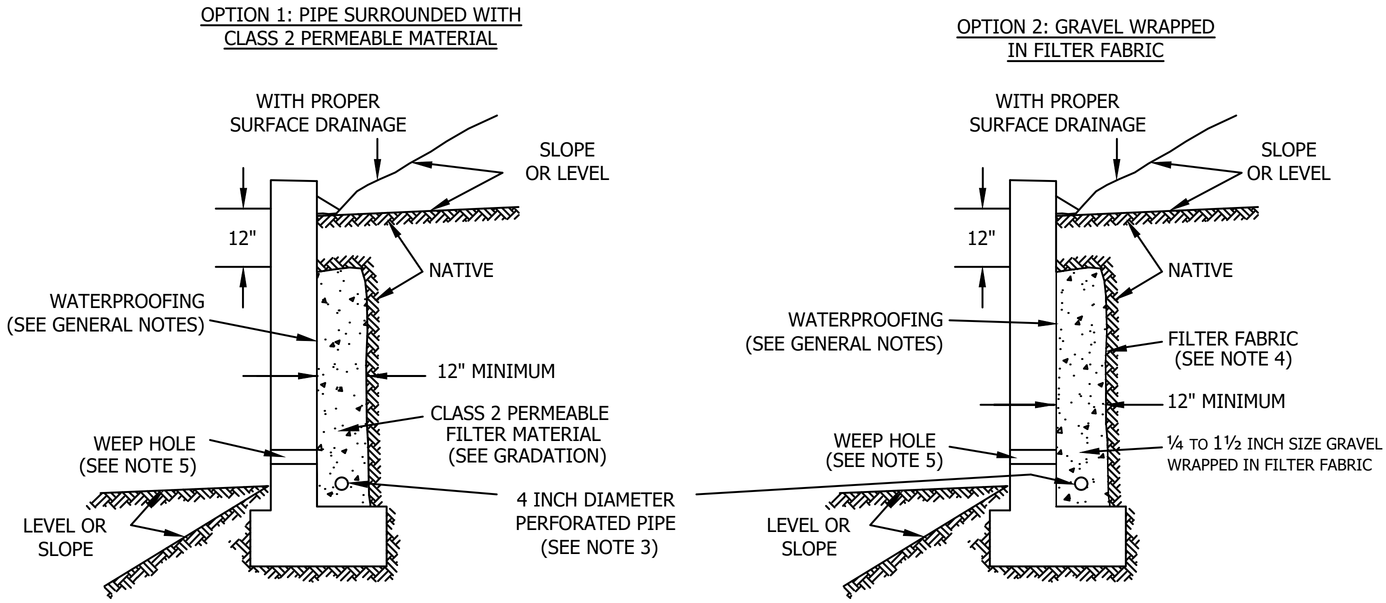
Proposed Industrial Building
13711 Freeway Drive
Santa Fe Springs, California

FIGURE 7

Leighton

Map Saved as V:\Drafting\13429\001\Maps\13429-001_F07_DIM_2022-02-24.mxd on 2/23/2022 10:47:56 AM Author: KVM (btran)

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



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

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 FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

FIGURE 8

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG RW-1

Project No.	13429.001	**Date Drilled**	2-10-22
Project	Rexford Freeway Drive	**Logged By**	MM
Drilling Co.	SoCal Drilling Co.	**Hole Diameter**	8"
Drilling Method	Rotary Wash - 140lb - Autohammer - 30" Drop	**Ground Elevation**	66'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

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			B-1					@Surface: 6 inches of asphalt concrete over 6 inches of aggregate base	
65								ML	**Artificial Fill, Undocumented (Afu):** @1': Clayey SILT, brown, wet	
	5			R-1	5 8 8	93	22	ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf):** @5': SILT, very stiff, light brown, very moist, little clay, FeO stains	
60										
				S-2	2 5 6		32		@7.5': SILT, very stiff, light brown, very moist, FeO stains	
	10			R-3	7 14 10	103	7		@10': Sandy SILT, very stiff, light gray, slightly moist, mostly fine sand	CN
55										
	15			S-4	7 9 10		19	SP	@15': Poorly graded SAND, medium dense, light brown, very moist, mostly fine sand, micaceous	
50										
	20			R-5	9 10 7	86	23		@20': Poorly graded SAND, medium dense, gray, wet, fine sand	
45										
	25			S-6	1 3 5		29	CL	@25': CLAY, stiff, light gray, very moist, little silt, trace pinhole pores, carbonate precipitation throughout, few organic fragments	
40										
	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 RW-1

Project No.	13429.001	**Date Drilled**	2-10-22
Project	Rexford Freeway Drive	**Logged By**	MM
Drilling Co.	SoCal Drilling Co.	**Hole Diameter**	8"
Drilling Method	Rotary Wash - 140lb - Autohammer - 30" Drop	**Ground Elevation**	66'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

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
35	30			R-7	7 18 23	92	31	ML	@30': SILT, hard, blue gray, wet, thinly bedded to laminated, few micas, trace MnO spots	
30	35			S-8	1 3 3		83	CL	@35': CLAY, medium stiff to stiff, blue gray, wet, laminated, trace MnO spots and trace fine organic material @36': Becomes dark gray CLAY, few fine shells, few organic material up to approximately 1 inch long	
25	40			R-9	7 12 18	91	32	ML-CL	@40': SILT to CLAY, very stiff, blue gray, wet, micaceous, calcium carbonates throughout, trace fine organic material, trace MnO spots, low plasticity	
20	45			S-10	3 5 7		19		@45': SILT to CLAY, very stiff, blue gray, wet, few fine sand, micaceous, trace MnO spots, trace fine organics, yellow orange FeO staining	
15	50			R-11	7 16 21				@50': SILT to CLAY, hard, blue gray to gray, wet	
10	55								**Total Depth: 51.5 feet** **Groundwater encountered at 30 feet** **Drummed cuttings and backfilled with cement bentonite grout and patched with black-dyed concrete.**	
	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
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- 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 RW-2

Project No.	13429.001	**Date Drilled**	2-10-22
Project	Rexford Freeway Drive	**Logged By**	MM
Drilling Co.	SoCal Drilling Co.	**Hole Diameter**	8"
Drilling Method	Rotary Wash - 140lb - Autohammer - 30" Drop	**Ground Elevation**	68'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0						ML	@Surface: 5 inches of asphalt concrete over 6 inches of aggregate base. Encountered rebar in concrete. **Artificial Fill, Undocumented (Afu):** @0.91': Clayey SILT, brown, wet	
65									
	5		S-1	3 4 5		18	ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf):** @5': Sandy SILT, stiff, light brown, very moist, micaceous, fine sand	
60			R-2	2 5 6	94	16	CL-ML	@7.5': Silty CLAY, stiff, light brown, moist, micaceous, FeO veins	CN, DS
	10		S-3	2 3 4		29	SC	@10': Clayey SAND, medium dense, light brown, very moist, micaceous, FeO veins, trace organic material	AL
55									
	15		R-4	3 7 9	102	26	CL-ML	@15': Silty CLAY, stiff, gray brown, very moist, fine sand, FeO spots	, DS
50									
	20		S-5	3 9 11		34	CL	@20': Sandy Lean CLAY, very stiff to hard, gray brown, wet, micaceous, laminated, few FeO stains, clay lenses	AL
45									
	25		R-6	4 7 9	90	31	ML-CL	@25': SILT to CLAY, stiff, gray, very moist, micaceous, calcium carbonate nodules, trace organic debris, trace FeO staining, blocky, low plasticity	
40									
	∇ 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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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

GEOTECHNICAL BORING LOG RW-2

Project No.	13429.001	**Date Drilled**	2-10-22
Project	Rexford Freeway Drive	**Logged By**	MM
Drilling Co.	SoCal Drilling Co.	**Hole Diameter**	8"
Drilling Method	Rotary Wash - 140lb - Autohammer - 30" Drop	**Ground Elevation**	68'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

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
	30			S-7	7 12 14		23	SP-SM	@30': Poorly graded SAND to Silty SAND, dense, gray, very moist, fine sand, micaceous	
35										
	35			R-8	9 14 16	89	32	ML	@35': SILT, very stiff, gray, wet, micaceous, laminated	
30										
	40			S-9	6 6 12		30		@40': SILT, very stiff, gray, wet, few clay, micaceous, increased clay with depth	
25										
	45			R-10	6 12 21	85	37	CL	@45': CLAY, very stiff, gray, very moist, calcium carbonate throughout, trace fine organic material, high plasticity	
20										
	50			S-11	2 5 8		32		@50': CLAY, very stiff	
15									**Total Depth: 51.5 feet** **Groundwater encountered at 30 feet** **Drummed cuttings and backfilled with cement bentonite grout and patched with black-dyed concrete.**	
	55									
10										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
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- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Rexford
Santa Fe Springs, CA
February 21, 2022

Prepared for:

Mr. Jeff Pflueger
Leighton & Associates
17781 Cowan
Irvine, CA 92614-6009
Office (800) 253-4567 / Fax (949) 250-1114

Prepared by:

KTE
KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Rexford project located in Santa Fe Springs, California. The work was performed by Kehoe Testing & Engineering (KTE) on February 21, 2022. The scope of work was performed as directed by Leighton & Associates personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at three locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	50	
CPT-2	50	
CPT-3	50	

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At location CPT-2, shear wave measurements were obtained at approximately 5-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, “Interpretation of Cone Penetration Test…”, 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

02/23/22-aa-3857

APPENDIX

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

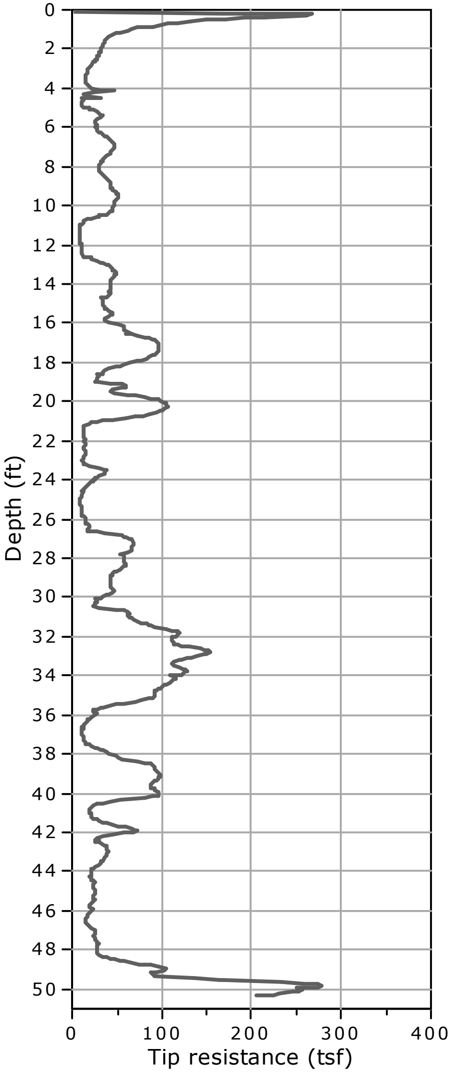
Project: Leighton & Associates / Rexford

Location: Santa Fe Springs, CA

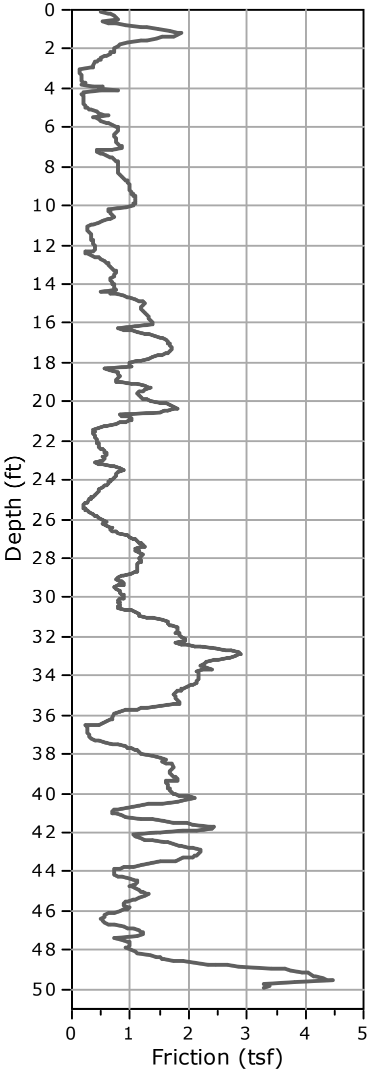
CPT-1

Total depth: 50.33 ft, Date: 2/21/2022

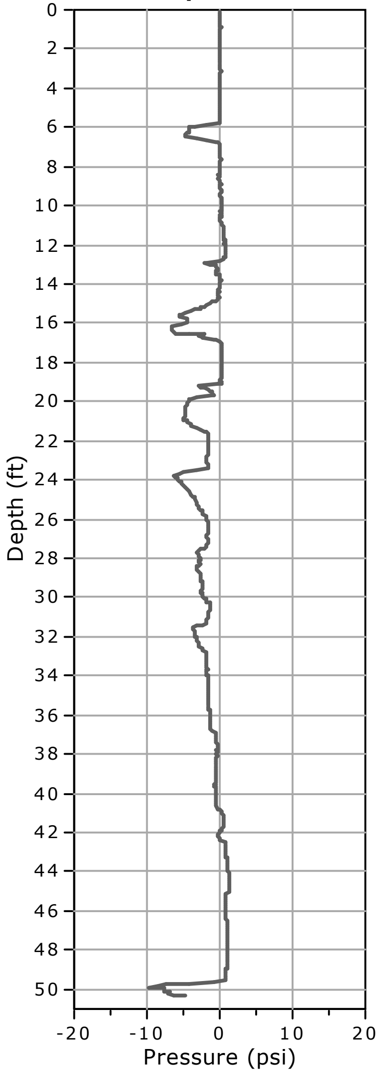
Cone resistance



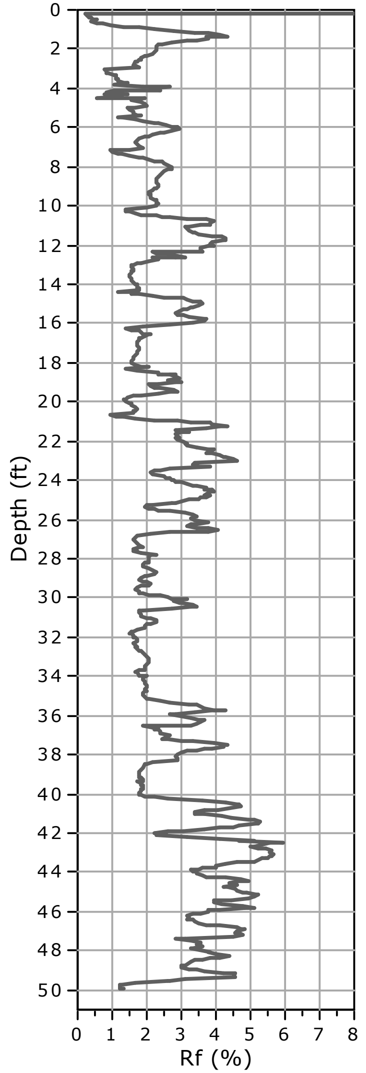
Sleeve friction



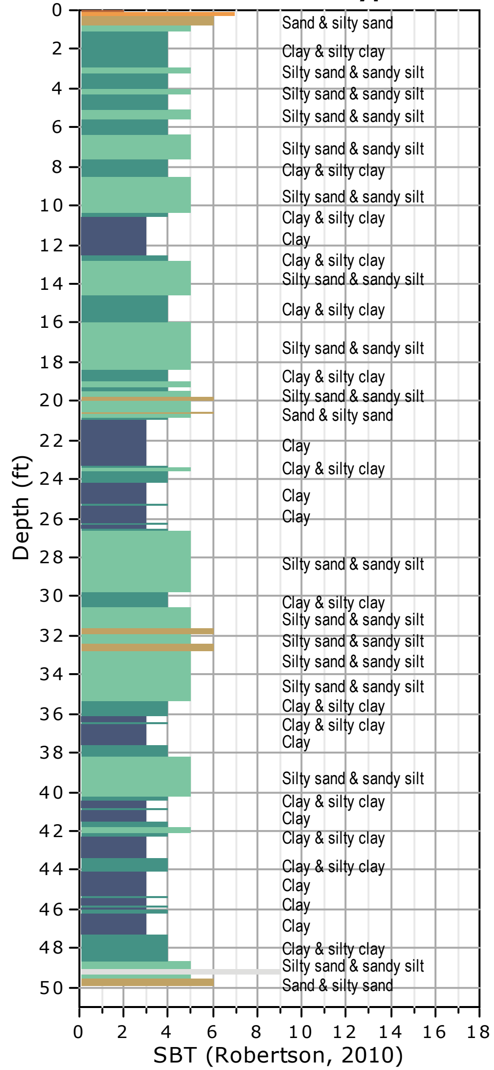
Pore pressure u



Friction ratio



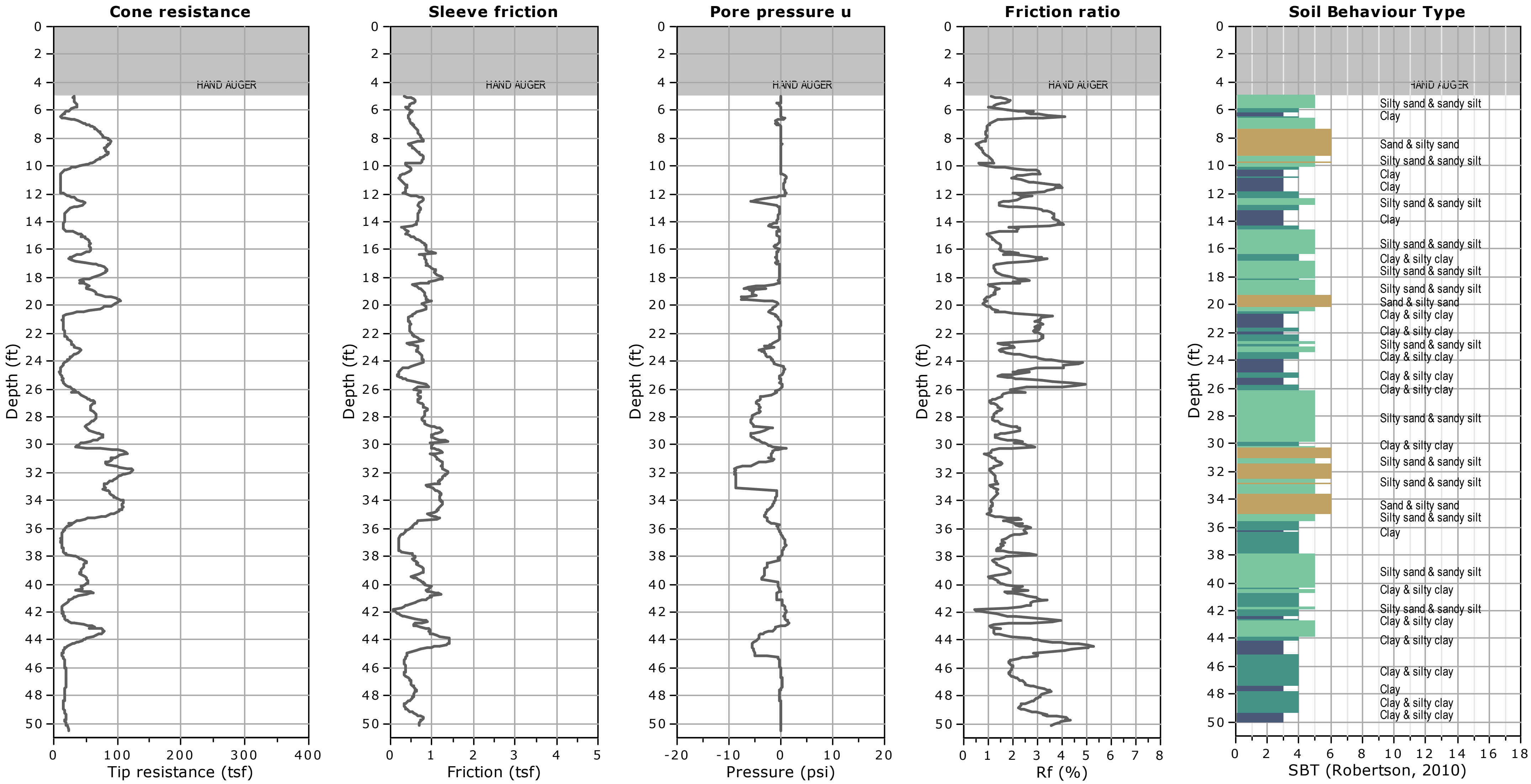
Soil Behaviour Type



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Leighton & Associates / Rexford
Location: Santa Fe Springs, CA

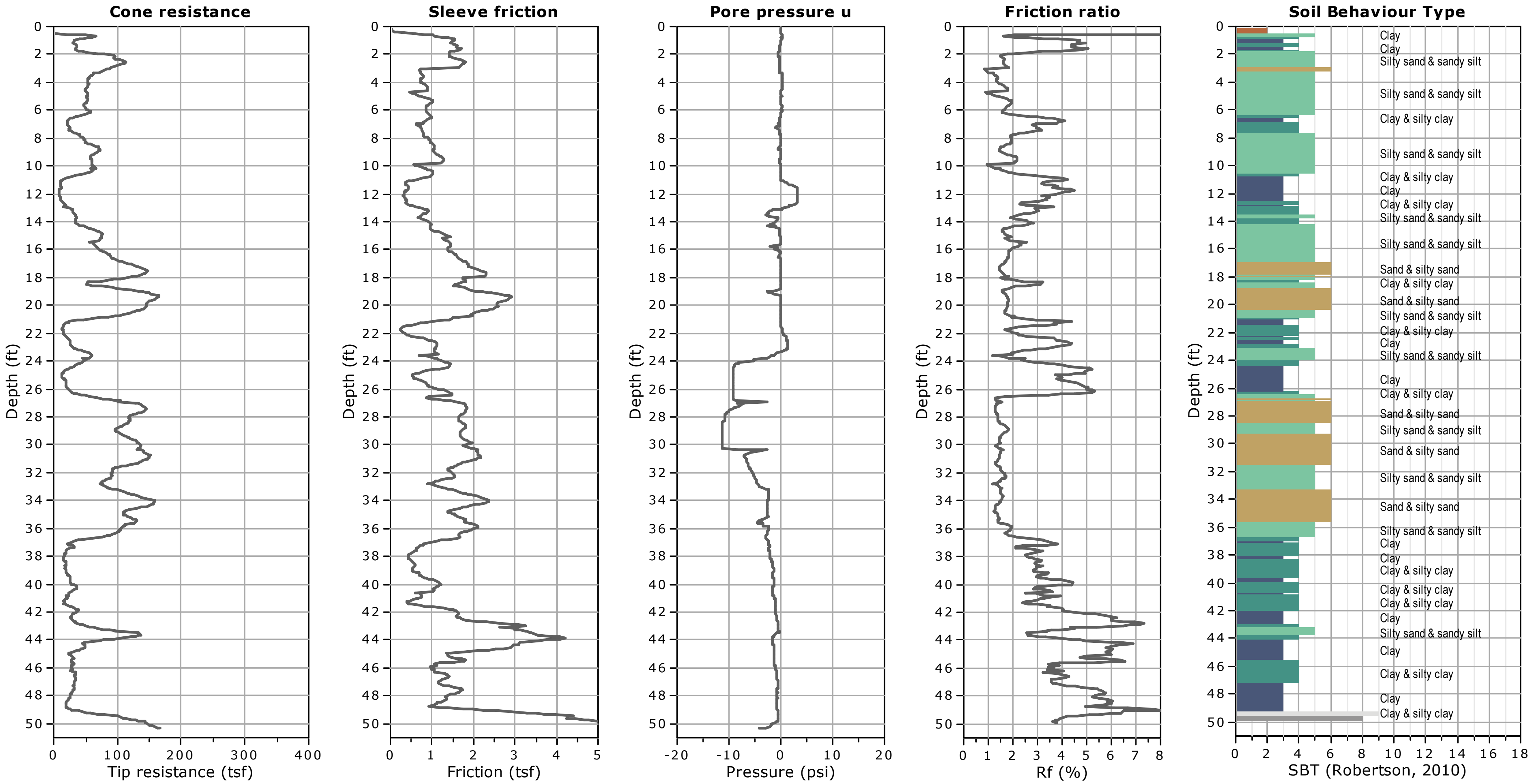
CPT-2
Total depth: 50.47 ft, Date: 2/21/2022



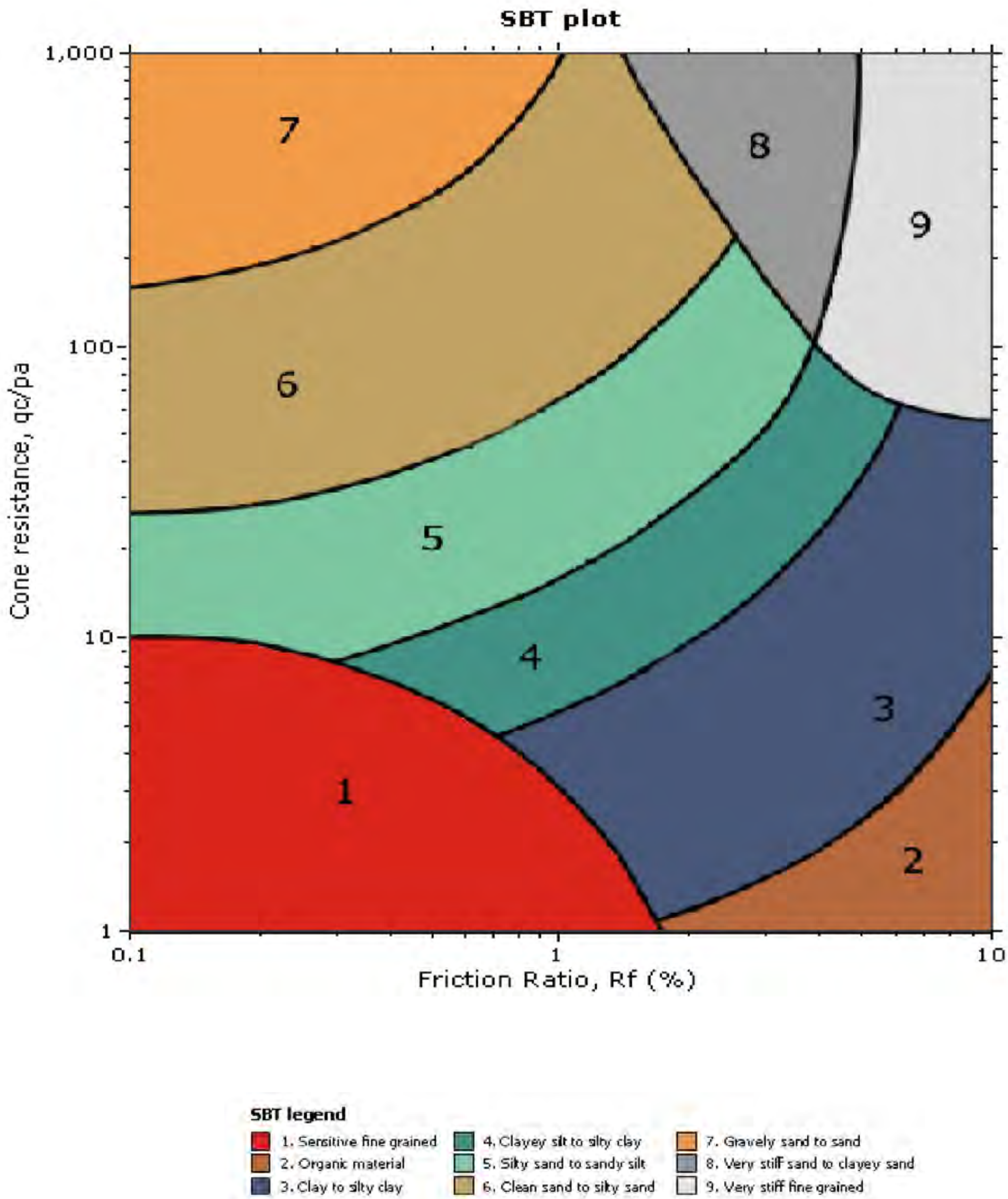
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Leighton & Associates / Rexford
Location: Santa Fe Springs, CA

CPT-3
Total depth: 50.35 ft, Date: 2/21/2022



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com



Leighton & Associates
Rexford
Santa Fe Springs, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-2	5.05	4.05	4.52	6.64	680	
	10.07	9.07	9.29	13.04	712	745
	15.03	14.03	14.17	21.52	659	576
	20.05	19.05	19.15	29.42	651	631
	25.20	24.20	24.28	37.38	650	644
	30.09	29.09	29.16	44.52	655	683
	35.17	34.17	34.23	51.72	662	704
	40.55	39.55	39.60	59.72	663	672
	45.11	44.11	44.16	65.84	671	744
	50.46	49.46	49.50	73.12	677	734

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

APPENDIX B

LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	J. Gonzalez	Date:	02/23/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	02/24/22
Boring No.:	CPT-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown sandy silt s(ML)				

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)	3621	3702	3767	3745		
Weight of Mold (g)	1826	1826	1826	1826		
Net Weight of Soil (g)	1795	1876	1941	1919		
Wet Weight of Soil + Cont. (g)	487.7	463.0	461.2	514.6		
Dry Weight of Soil + Cont. (g)	453.3	421.0	411.1	447.1		
Weight of Container (g)	38.5	39.8	40.2	39.3		
Moisture Content (%)	8.29	11.02	13.51	16.55		
Wet Density (pcf)	118.8	124.2	128.5	127.0		
Dry Density (pcf)	109.7	111.9	113.2	109.0		

Maximum Dry Density (pcf) **113.2** **Optimum Moisture Content (%)** **13.5**

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

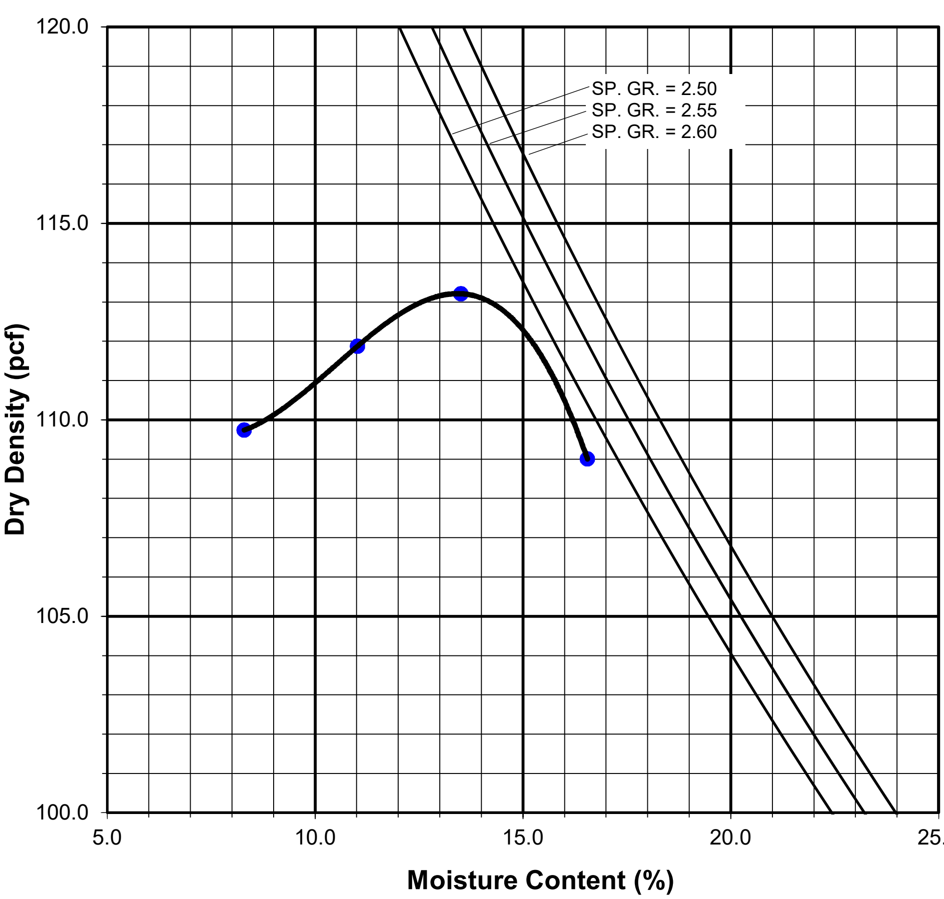
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Berdy	Date:	03/01/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	03/22/22
Boring No.:	CPT-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown sandy silt s(ML)				

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	1.0010
Wt. Comp. Soil + Mold	(g)	603.00	423.70
Wt. of Mold	(g)	208.60	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	798.10	632.30
Dry Wt. of Soil + Cont.	(g)	725.50	567.15
Wt. of Container	(g)	0.00	208.60
Moisture Content	(%)	10.01	18.17
Wet Density	(pcf)	119.0	127.7
Dry Density	(pcf)	108.1	108.0
Void Ratio		0.559	0.560
Total Porosity		0.359	0.359
Pore Volume	(cc)	74.2	74.4
Degree of Saturation (%) [S_{meas}]		**48.3**	87.6

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.)
03/01/22	7:29	1.0	0	0.6045
03/01/22	7:39	1.0	10	0.6045
Add Distilled Water to the Specimen				
03/01/22	8:02	1.0	23	0.6055
03/02/22	6:30	1.0	1371	0.6055
03/02/22	7:38	1.0	1439	0.6055

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**1**

ATTERBERG LIMITS

ASTM D 4318

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	J. Domingo	Date:	02/24/22
Project No. :	13429.001	Input By:	G. Bathala	Date:	03/09/22
Boring No.:	CPT-2	Checked By:	A. Santos		
Sample No.:	B-1	Depth (ft.)	0-5		
Soil Identification:	Olive brown sandy silt s(ML)				

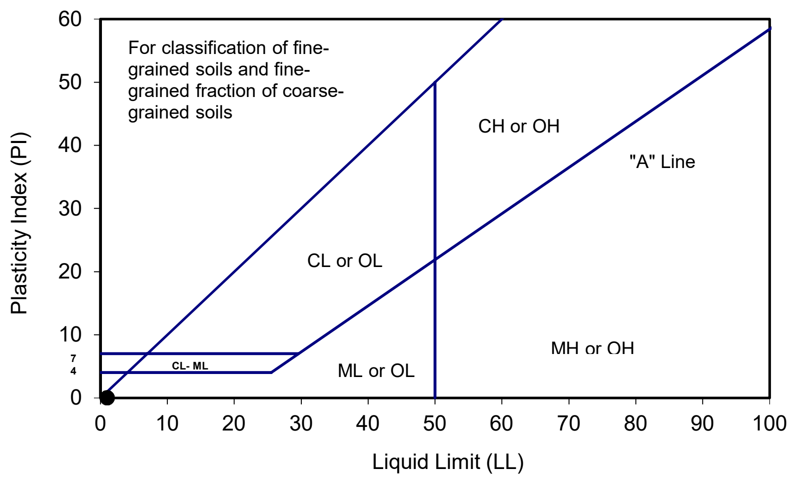
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			5			
Wet Wt. of Soil + Cont. (g)	**Cannot be rolled:**		22.93	**Cannot get more than 5 blows:**		
Dry Wt. of Soil + Cont. (g)	**NonPlastic**		18.57	**NonPlastic**		
Wt. of Container (g)			1.07			
Moisture Content (%) [Wn]			24.91			

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**NP**

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

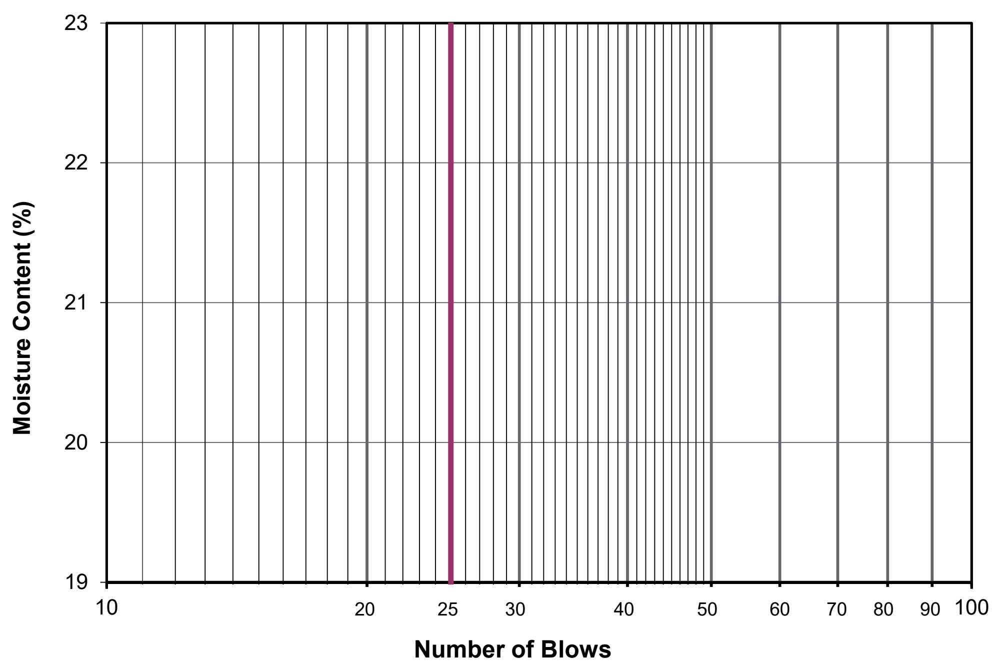
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



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:	Rexford Freeway Santa Fe Springs	Tested By:	S. Felter	Date:	02/24/22
Project No. :	13429.001	Input By:	G. Bathala	Date:	02/28/22
Boring No.:	RW-2	Checked By:	A. Santos		
Sample No.:	S-3	Depth (ft.)	10.0		
Soil Identification:	Dark gray clayey sand (SC)				

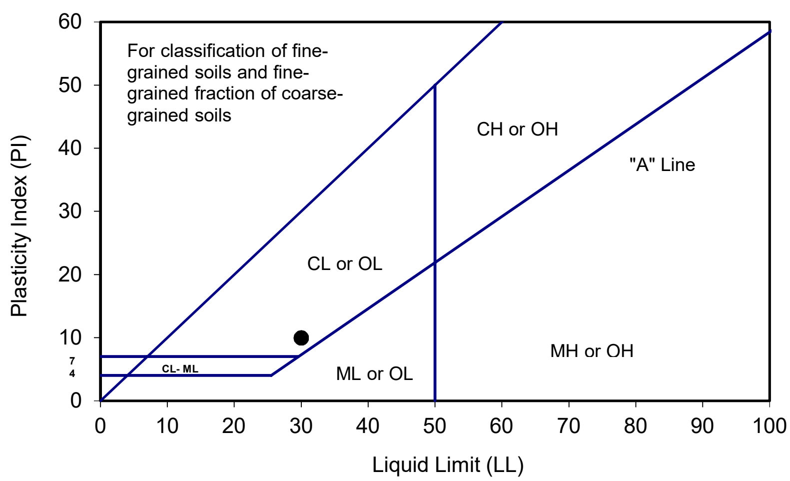
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			26	20	15	
Wet Wt. of Soil + Cont. (g)	10.32	10.18	22.16	21.27	20.47	
Dry Wt. of Soil + Cont. (g)	8.77	8.67	17.32	16.48	15.81	
Wt. of Container (g)	1.08	1.11	1.11	1.04	1.12	
Moisture Content (%) [Wn]	20.16	19.97	29.86	31.02	31.72	

Liquid Limit	**30**
Plastic Limit	**20**
Plasticity Index	**10**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 7.3

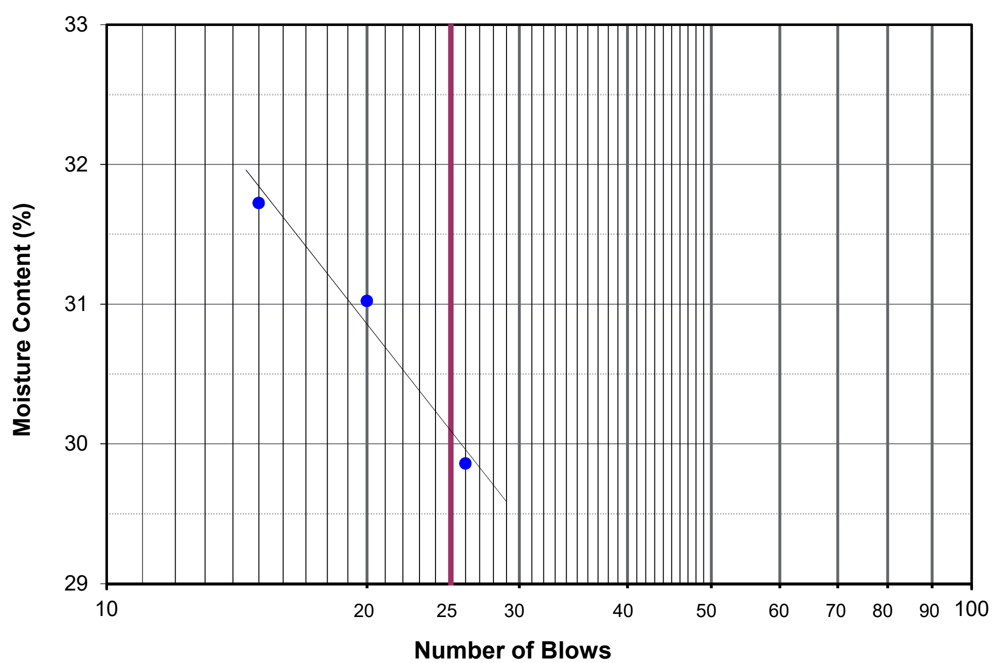
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



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:	Rexford Freeway Santa Fe Springs	Tested By:	S. Felter	Date:	02/24/22
Project No. :	13429.001	Input By:	G. Bathala	Date:	02/28/22
Boring No.:	RW-2	Checked By:	A. Santos		
Sample No.:	S-5	Depth (ft.)	20.0		
Soil Identification:	Dark gray sandy lean clay s(CL)				

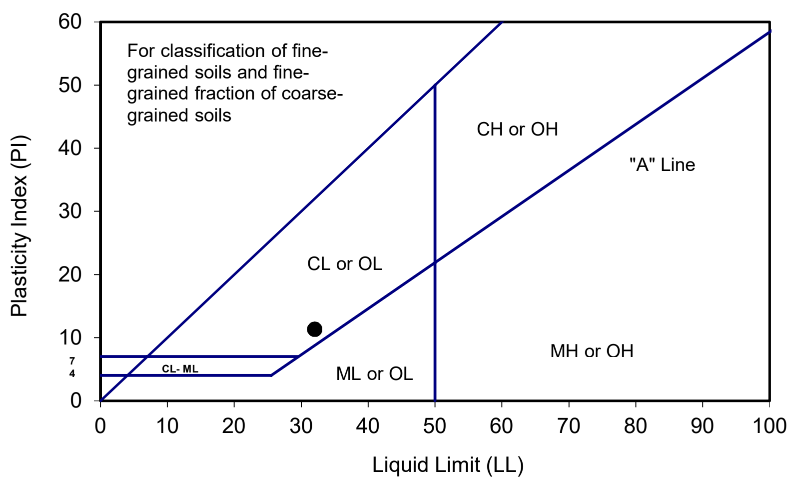
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	23	16	
Wet Wt. of Soil + Cont. (g)	9.99	10.05	20.65	20.34	20.55	
Dry Wt. of Soil + Cont. (g)	8.45	8.52	15.96	15.59	15.60	
Wt. of Container (g)	1.06	1.06	1.01	1.09	1.05	
Moisture Content (%) [Wn]	20.84	20.51	31.37	32.76	34.02	

Liquid Limit	**32**
Plastic Limit	**21**
Plasticity Index	**11**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 8.76

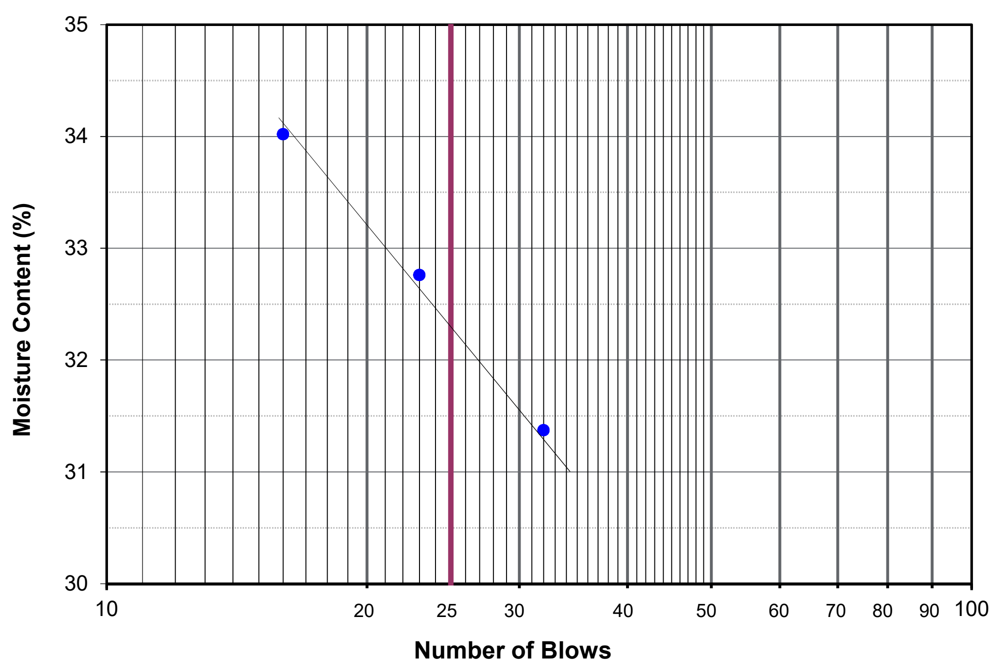
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

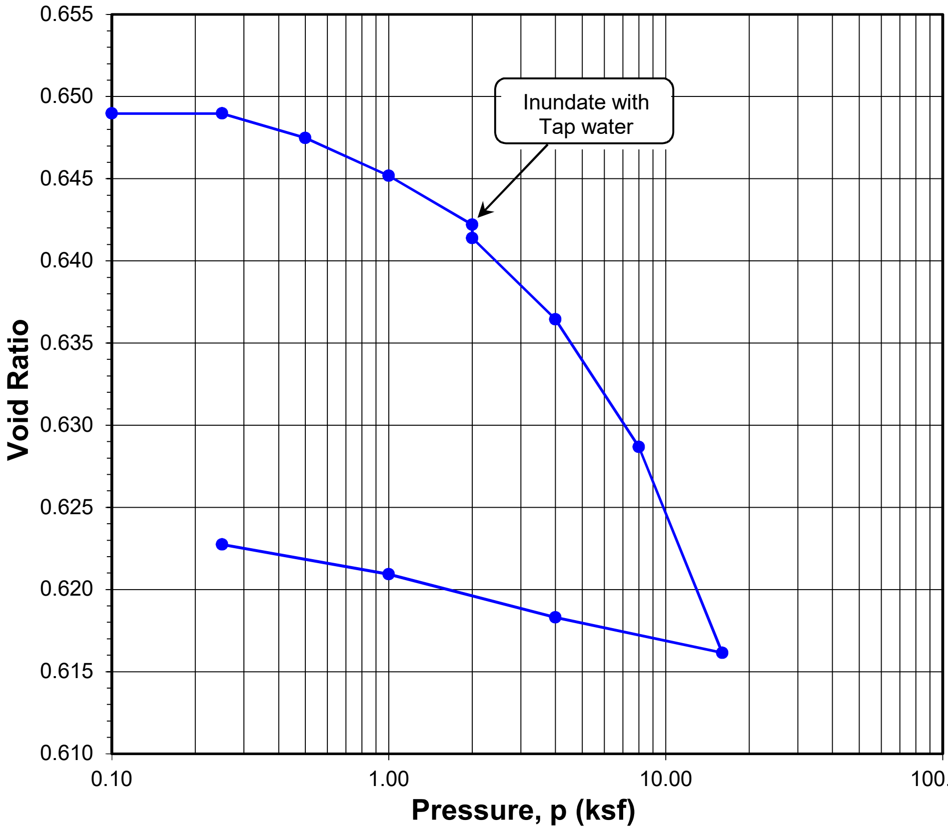
- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Bathala	Date:	02/24/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	03/22/22
Boring No.:	CPT-2	Depth (ft.):	0-5		
Sample No.:	B-1	Sample Type:	90% Remold		
Soil Identification:	Olive brown sandy silt s(ML)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	184.74
Weight of Ring (g)	45.45
Height after consol. (in.)	0.9840
Before Test	
Wt.Wet Sample+Cont. (g)	155.00
Wt.of Dry Sample+Cont. (g)	143.60
Weight of Container (g)	58.11
Initial Moisture Content (%)	13.3
Initial Dry Density (pcf)	102.2
Initial Saturation (%)	55
Initial Vertical Reading (in.)	0.3349
After Test	
Wt.of Wet Sample+Cont. (g)	250.05
Wt. of Dry Sample+Cont. (g)	226.27
Weight of Container (g)	58.11
Final Moisture Content (%)	19.38
Final Dry Density (pcf)	103.7
Final Saturation (%)	84
Final Vertical Reading (in.)	0.3162
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

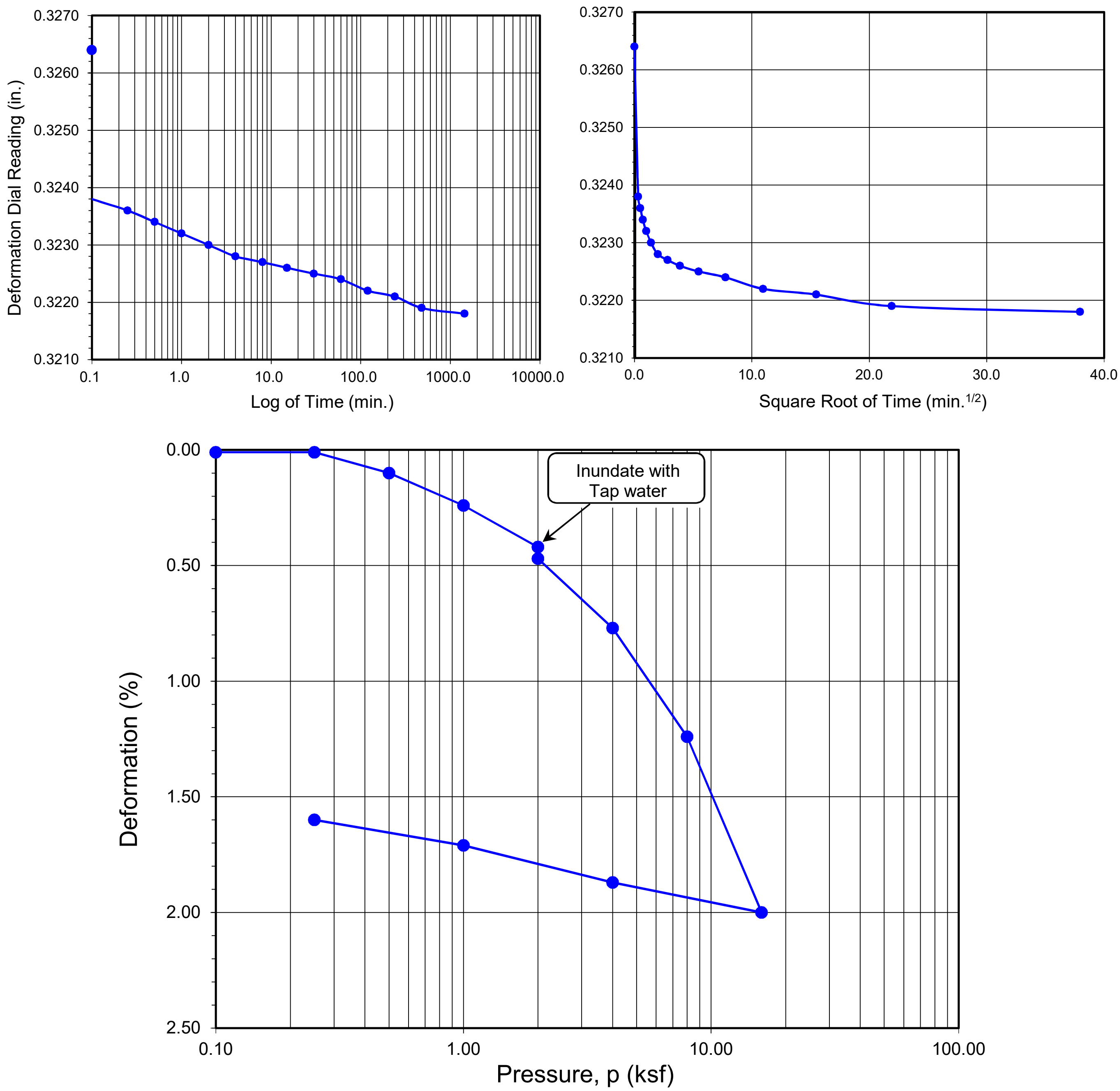


Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3348	0.9999	0.00	0.01	0.649	0.01
0.25	0.3343	0.9994	0.05	0.06	0.649	0.01
0.50	0.3326	0.9977	0.13	0.23	0.647	0.10
1.00	0.3302	0.9953	0.23	0.47	0.645	0.24
2.00	0.3269	0.9920	0.38	0.80	0.642	0.42
2.00	0.3264	0.9915	0.38	0.85	0.641	0.47
4.00	0.3218	0.9869	0.54	1.31	0.636	0.77
8.00	0.3153	0.9804	0.72	1.96	0.629	1.24
16.00	0.3055	0.9706	0.94	2.94	0.616	2.00
4.00	0.3090	0.9741	0.72	2.59	0.618	1.87
1.00	0.3130	0.9781	0.48	2.19	0.621	1.71
0.25	0.3162	0.9813	0.27	1.87	0.623	1.60

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/1/22	10:45:00	0.0	0.0	0.3264
3/1/22	10:45:06	0.1	0.3	0.3238
3/1/22	10:45:15	0.2	0.5	0.3236
3/1/22	10:45:30	0.5	0.7	0.3234
3/1/22	10:46:00	1.0	1.0	0.3232
3/1/22	10:47:00	2.0	1.4	0.3230
3/1/22	10:49:00	4.0	2.0	0.3228
3/1/22	10:53:00	8.0	2.8	0.3227
3/1/22	11:00:00	15.0	3.9	0.3226
3/1/22	11:15:00	30.0	5.5	0.3225
3/1/22	11:45:00	60.0	7.7	0.3224
3/1/22	12:45:00	120.0	11.0	0.3222
3/1/22	14:45:00	240.0	15.5	0.3221
3/1/22	18:45:00	480.0	21.9	0.3219
3/2/22	10:45:00	1440.0	37.9	0.3218

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
CPT-2	**B-1**	**0-5**	**13.3**	**19.4**	**102.2**	**103.7**	**0.649**	**0.623**	**55**	**84**

Soil Identification: Olive brown sandy silt s(ML)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

03-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: Rexford Freeway Santa Fe Springs

Project No.: 13429.001

Boring No.: RW-1

Sample No.: R-3

Soil Identification: Olive gray sandy silt s(ML)

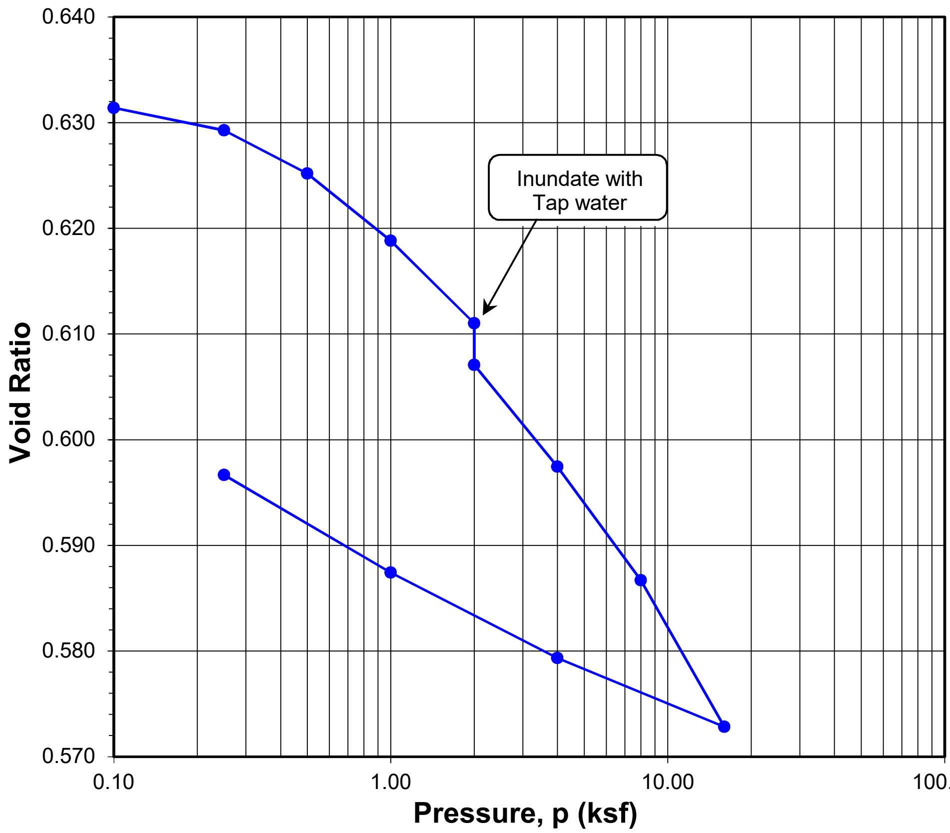
Tested By: G. Bathala Date: 02/24/22

Checked By: A. Santos Date: 03/09/22

Depth (ft.): 10.0

Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	177.54
Weight of Ring (g)	45.15
Height after consol. (in.)	0.9787
Before Test	
Wt.Wet Sample+Cont. (g)	183.66
Wt.of Dry Sample+Cont. (g)	176.06
Weight of Container (g)	60.24
Initial Moisture Content (%)	6.6
Initial Dry Density (pcf)	103.3
Initial Saturation (%)	28
Initial Vertical Reading (in.)	0.2692
After Test	
Wt.of Wet Sample+Cont. (g)	259.79
Wt. of Dry Sample+Cont. (g)	233.75
Weight of Container (g)	66.97
Final Moisture Content (%)	21.41
Final Dry Density (pcf)	103.4
Final Saturation (%)	92
Final Vertical Reading (in.)	0.2444
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

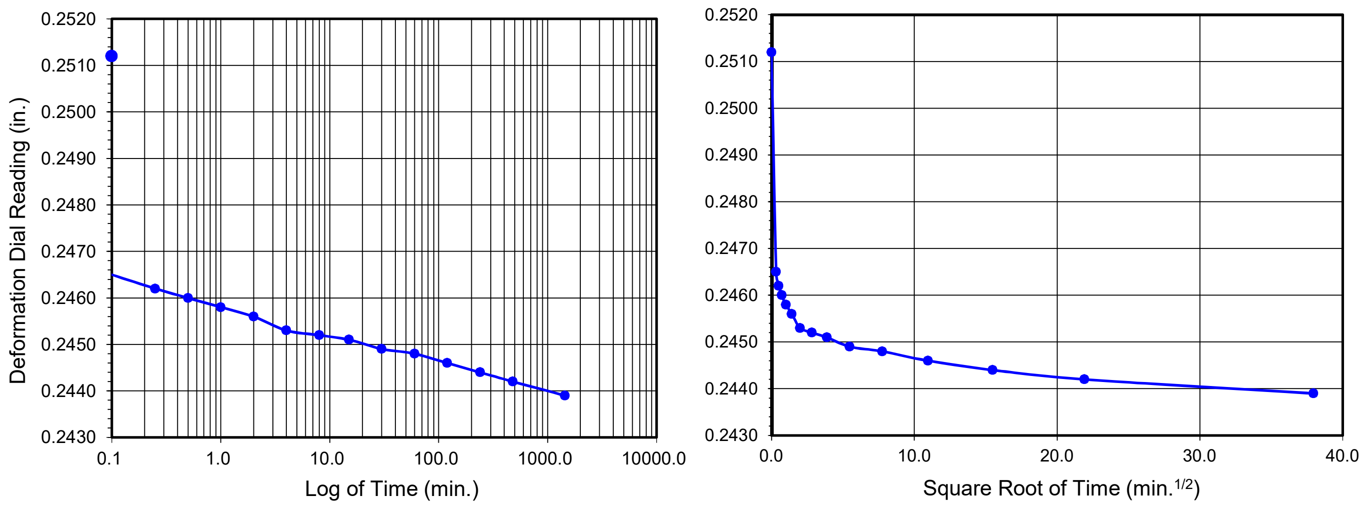


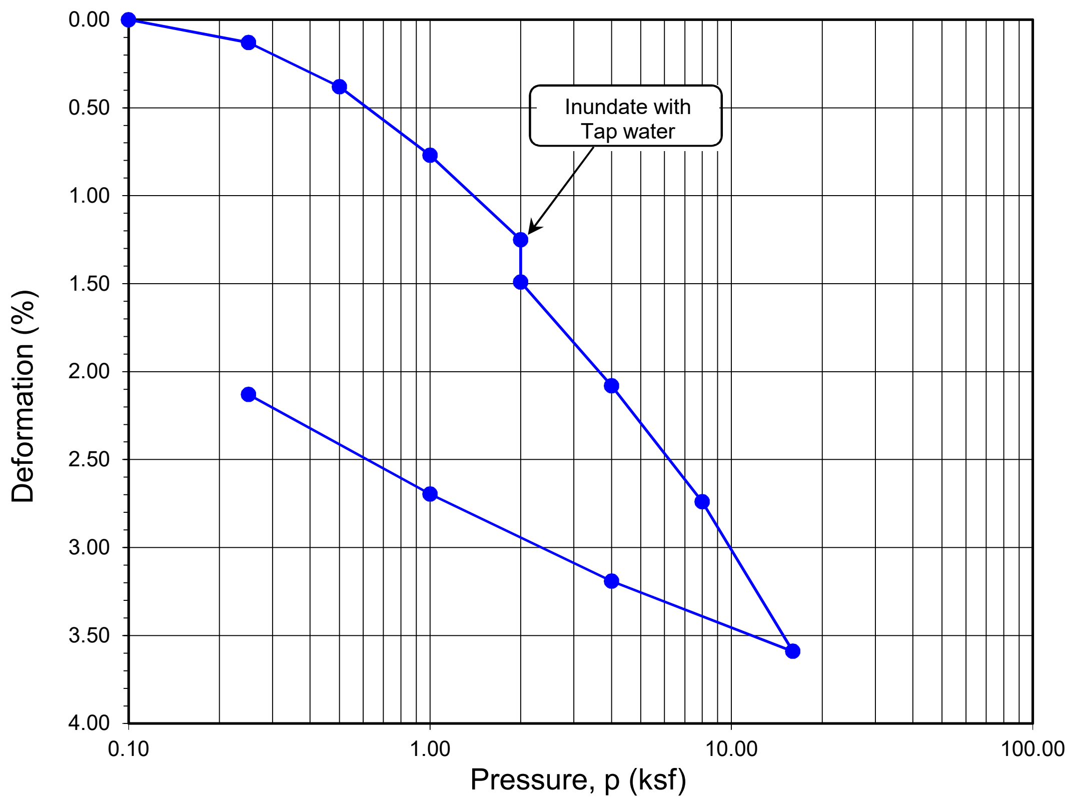
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2692	1.0000	0.00	0.00	0.631	0.00
0.25	0.2674	0.9982	0.05	0.18	0.629	0.13
0.50	0.2643	0.9951	0.11	0.49	0.625	0.38
1.00	0.2595	0.9903	0.20	0.97	0.619	0.77
2.00	0.2536	0.9844	0.31	1.56	0.611	1.25
2.00	0.2512	0.9820	0.31	1.80	0.607	1.49
4.00	0.2439	0.9747	0.45	2.53	0.597	2.08
8.00	0.2357	0.9665	0.61	3.35	0.587	2.74
16.00	0.2252	0.9560	0.81	4.40	0.573	3.59
4.00	0.2306	0.9614	0.67	3.86	0.579	3.19
1.00	0.2374	0.9682	0.49	3.18	0.587	2.69
0.25	0.2444	0.9752	0.35	2.48	0.597	2.13

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/1/22	10:50:00	0.0	0.0	0.2512
3/1/22	10:50:06	0.1	0.3	0.2465
3/1/22	10:50:15	0.2	0.5	0.2462
3/1/22	10:50:30	0.5	0.7	0.2460
3/1/22	10:51:00	1.0	1.0	0.2458
3/1/22	10:52:00	2.0	1.4	0.2456
3/1/22	10:54:00	4.0	2.0	0.2453
3/1/22	10:58:00	8.0	2.8	0.2452
3/1/22	11:05:00	15.0	3.9	0.2451
3/1/22	11:20:00	30.0	5.5	0.2449
3/1/22	11:50:00	60.0	7.7	0.2448
3/1/22	12:50:00	120.0	11.0	0.2446
3/1/22	14:50:00	240.0	15.5	0.2444
3/1/22	18:50:00	480.0	21.9	0.2442
3/2/22	10:50:00	1440.0	37.9	0.2439

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
RW-1	**R-3**	**10.0**	**6.6**	**21.4**	**103.3**	**103.4**	**0.631**	**0.597**	**28**	**92**

Soil Identification: Olive gray sandy silt s(ML)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

03-22

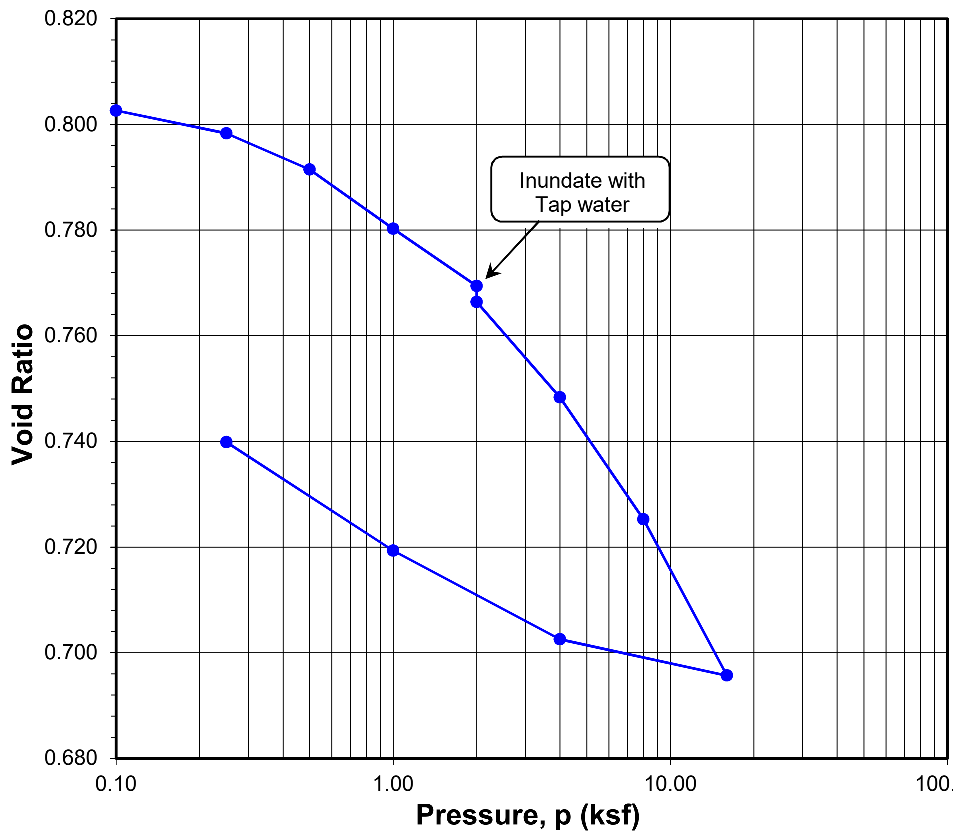
Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Bathala	Date:	02/24/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	03/09/22
Boring No.:	RW-2	Depth (ft.):	7.5		
Sample No.:	R-2	Sample Type:	Ring		
Soil Identification:	Olive gray silty clay (CL-ML)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	175.09
Weight of Ring (g)	44.22
Height after consol. (in.)	0.9651
Before Test	
Wt.Wet Sample+Cont. (g)	153.15
Wt.of Dry Sample+Cont. (g)	139.55
Weight of Container (g)	56.65
Initial Moisture Content (%)	16.4
Initial Dry Density (pcf)	93.5
Initial Saturation (%)	55
Initial Vertical Reading (in.)	0.3274
After Test	
Wt.of Wet Sample+Cont. (g)	242.27
Wt. of Dry Sample+Cont. (g)	210.00
Weight of Container (g)	61.69
Final Moisture Content (%)	31.00
Final Dry Density (pcf)	89.7
Final Saturation (%)	95
Final Vertical Reading (in.)	0.2864
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

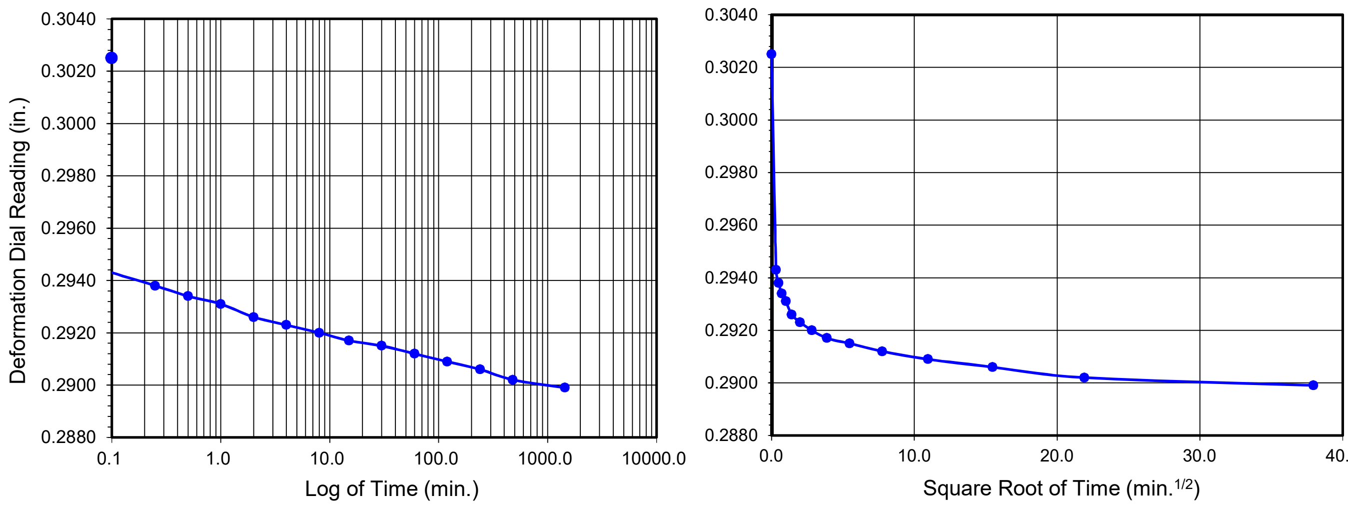


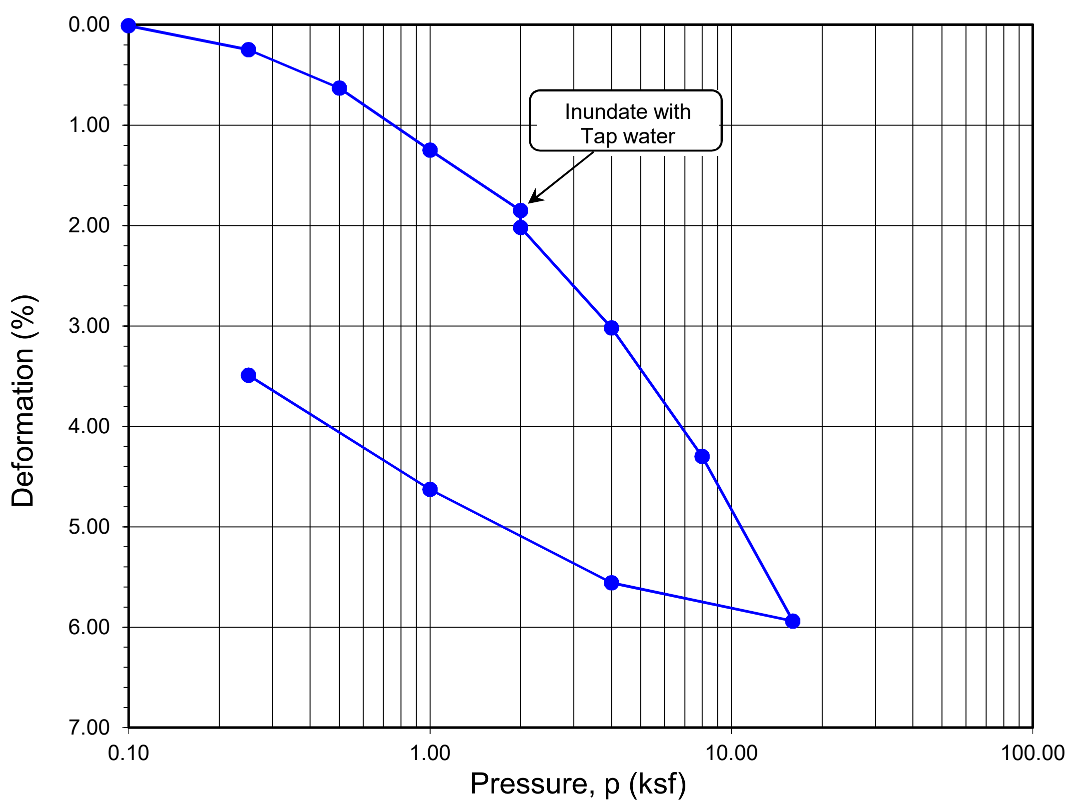
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3273	0.9999	0.00	0.01	0.803	0.01
0.25	0.3242	0.9968	0.07	0.32	0.798	0.25
0.50	0.3195	0.9921	0.16	0.79	0.791	0.63
1.00	0.3122	0.9848	0.27	1.52	0.780	1.25
2.00	0.3042	0.9768	0.47	2.32	0.769	1.85
2.00	0.3025	0.9751	0.47	2.49	0.766	2.02
4.00	0.2899	0.9625	0.73	3.75	0.748	3.02
8.00	0.2744	0.9470	1.00	5.30	0.725	4.30
16.00	0.2545	0.9271	1.35	7.29	0.696	5.94
4.00	0.2617	0.9343	1.01	6.57	0.703	5.56
1.00	0.2735	0.9461	0.76	5.39	0.719	4.63
0.25	0.2864	0.9590	0.61	4.10	0.740	3.49

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/1/22	10:55:00	0.0	0.0	0.3025
3/1/22	10:55:06	0.1	0.3	0.2943
3/1/22	10:55:15	0.2	0.5	0.2938
3/1/22	10:55:30	0.5	0.7	0.2934
3/1/22	10:56:00	1.0	1.0	0.2931
3/1/22	10:57:00	2.0	1.4	0.2926
3/1/22	10:59:00	4.0	2.0	0.2923
3/1/22	11:03:00	8.0	2.8	0.2920
3/1/22	11:10:00	15.0	3.9	0.2917
3/1/22	11:25:00	30.0	5.5	0.2915
3/1/22	11:55:00	60.0	7.7	0.2912
3/1/22	12:55:00	120.0	11.0	0.2909
3/1/22	14:55:00	240.0	15.5	0.2906
3/1/22	18:55:00	480.0	21.9	0.2902
3/2/22	10:55:00	1440.0	37.9	0.2899

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
RW-2	**R-2**	**7.5**	**16.4**	**31.0**	**93.5**	**89.7**	**0.803**	**0.740**	**55**	**95**

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

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

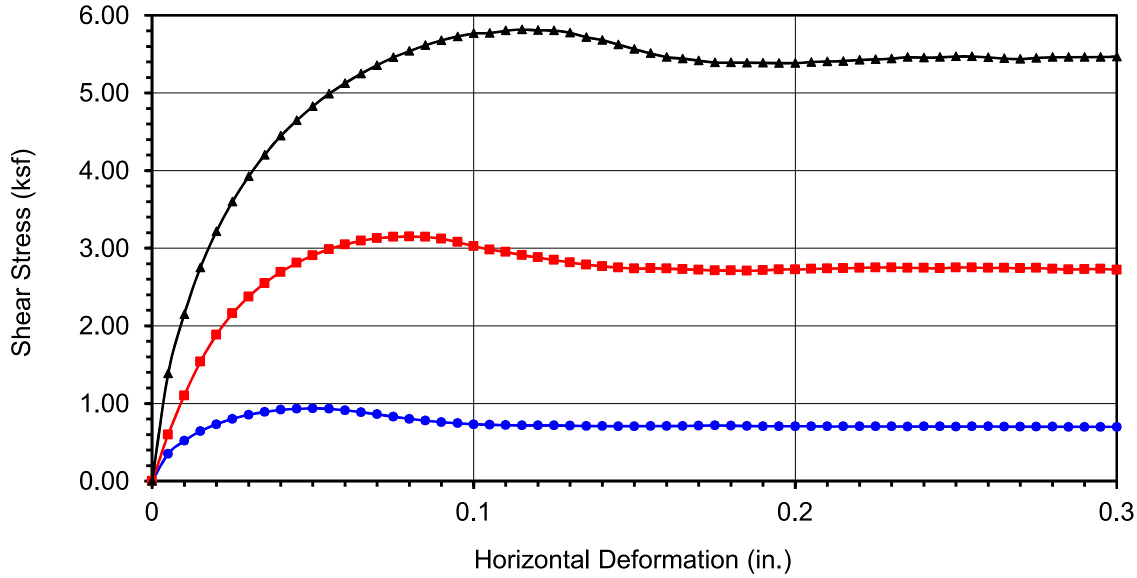
03-22

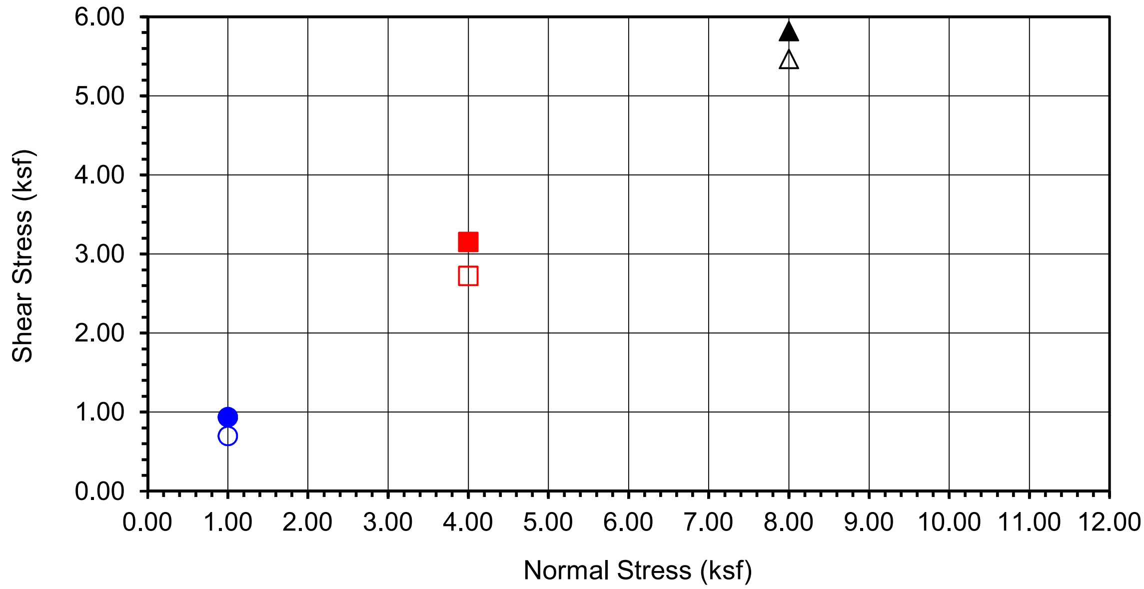
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Bathala	Date:	02/28/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	03/22/22
Boring No.:	CPT-2	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		
Soil Identification:	Olive brown sandy silt s(ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	185.01	184.82	184.95
Weight of Ring(gm):	45.66	45.46	45.51
Before Shearing			
Weight of Wet Sample+Cont.(gm):	155.00	155.00	155.00
Weight of Dry Sample+Cont.(gm):	143.60	143.60	143.60
Weight of Container(gm):	58.11	58.11	58.11
Vertical Rdg.(in): Initial	0.0000	0.2563	0.2516
Vertical Rdg.(in): Final	-0.0120	0.2784	0.2790
After Shearing			
Weight of Wet Sample+Cont.(gm):	210.71	202.81	204.17
Weight of Dry Sample+Cont.(gm):	186.15	178.78	180.75
Weight of Container(gm):	66.07	58.11	59.15
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**CPT-2**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:

Olive brown sandy silt s(ML)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.937	■ 3.150	▲ 5.816
Shear Stress @ End of Test (ksf)	○ 0.698	□ 2.723	△ 5.467
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	13.33	13.33	13.33
Dry Density (pcf)	102.3	102.3	102.3
Saturation (%)	55.5	55.5	55.6
Soil Height Before Shearing (in.)	0.9880	0.9779	0.9726
Final Moisture Content (%)	20.5	19.9	19.3

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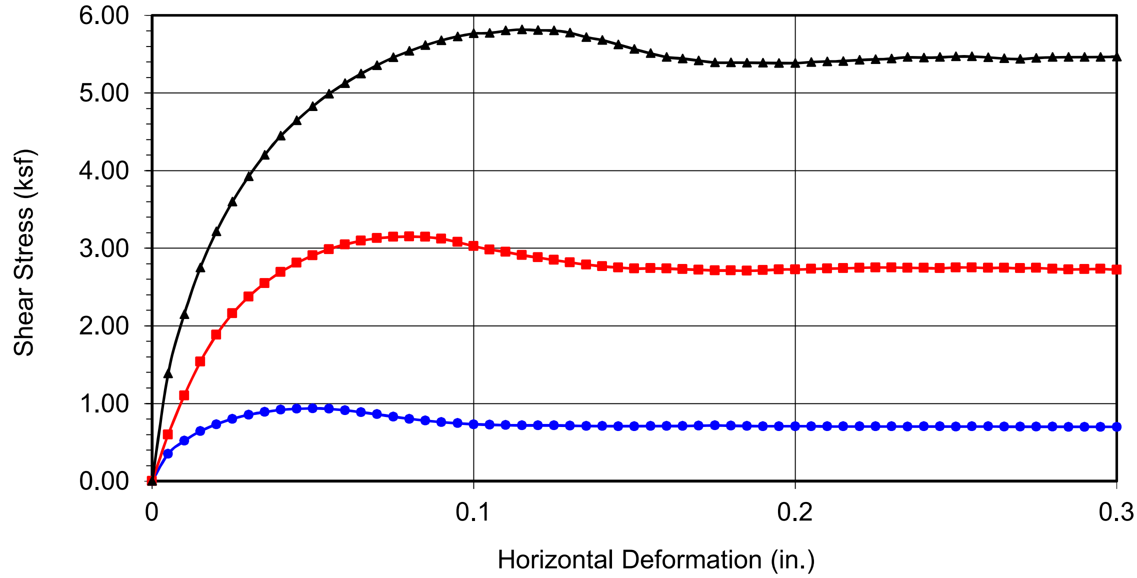
DIRECT SHEAR TEST RESULTS

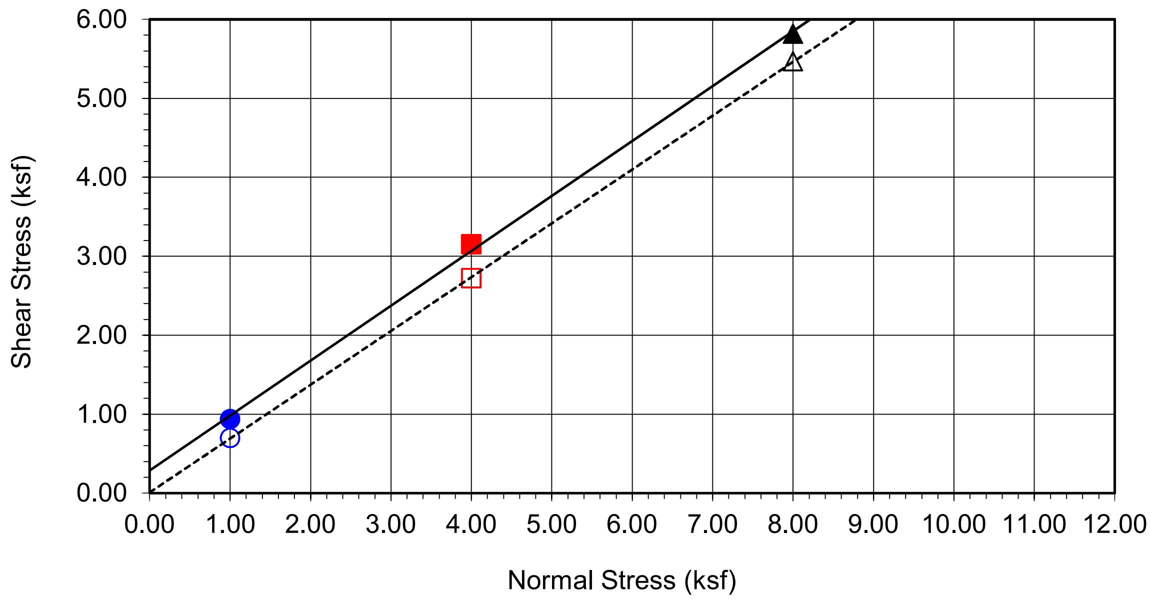
Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

02-22





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

Soil Identification:
Olive brown sandy silt s(ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	288	35
Ultimate	9	34

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.937	■ 3.150	▲ 5.816
Shear Stress @ End of Test (ksf)	○ 0.698	□ 2.723	△ 5.467
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	13.33	13.33	13.33
Dry Density (pcf)	102.3	102.3	102.3
Saturation (%)	55.5	55.5	55.6
Soil Height Before Shearing (in.)	0.9880	0.9779	0.9726
Final Moisture Content (%)	20.5	19.9	19.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

02-22

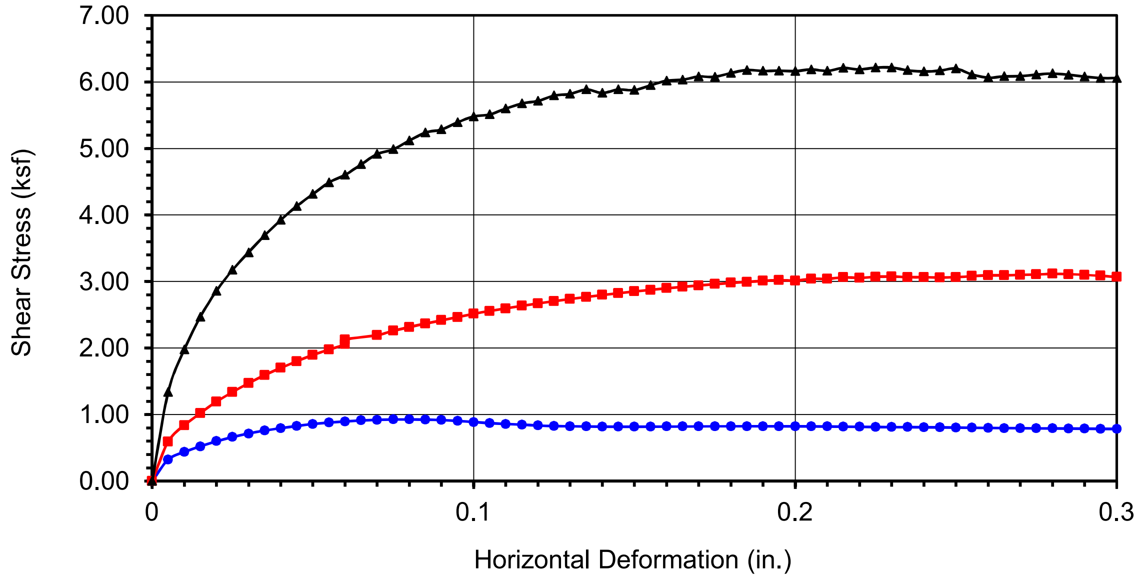
Leighton

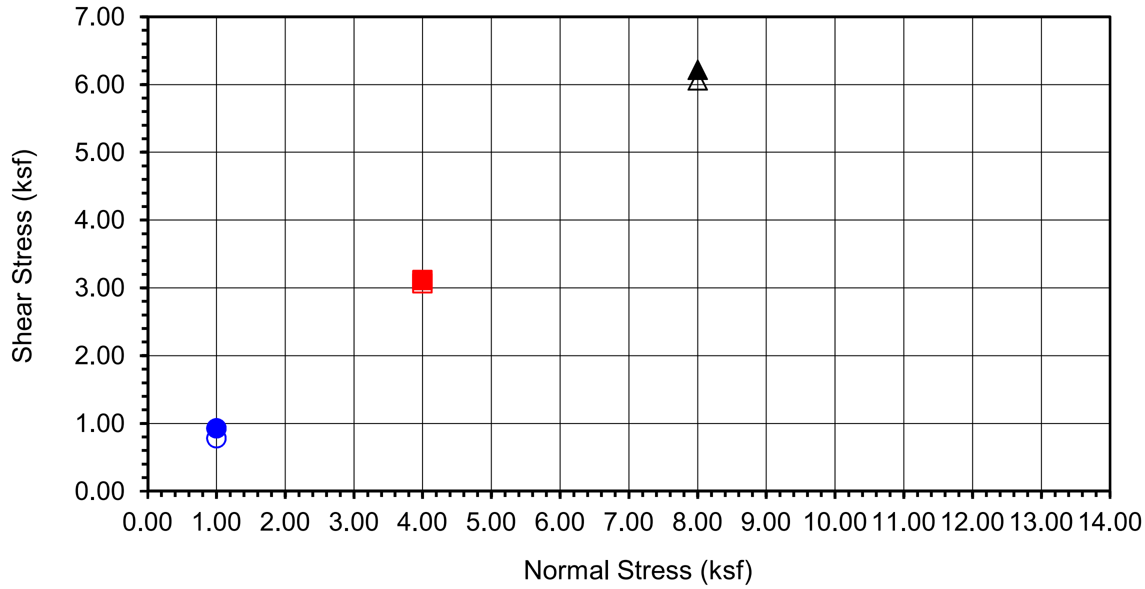
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Bathala	Date:	03/02/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	03/09/22
Boring No.:	RW-2	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.):	7.5		
Soil Identification:	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):	167.21	175.61	186.12
Weight of Ring(gm):	43.29	44.78	42.47
Before Shearing			
Weight of Wet Sample+Cont.(gm):	153.15	153.15	153.15
Weight of Dry Sample+Cont.(gm):	139.55	139.55	139.55
Weight of Container(gm):	56.65	56.65	56.65
Vertical Rdg.(in): Initial	0.0000	0.2687	0.2376
Vertical Rdg.(in): Final	-0.0114	0.3123	0.2800
After Shearing			
Weight of Wet Sample+Cont.(gm):	173.10	197.56	199.53
Weight of Dry Sample+Cont.(gm):	139.57	169.52	170.13
Weight of Container(gm):	39.01	67.12	59.16
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**RW-2**
Sample No.	**R-2**
Depth (ft)	**7.5**

Sample Type:

Ring

Soil Identification:

Olive gray silty clay (CL-ML)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.927	■ 3.115	▲ 6.215
Shear Stress @ End of Test (ksf)	○ 0.783	□ 3.068	△ 6.058
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 (%)	16.41	16.41	16.41
Dry Density (pcf)	88.5	93.5	102.6
Saturation (%)	49.0	55.1	69.0
Soil Height Before Shearing (in.)	0.9886	0.9564	0.9576
Final Moisture Content (%)	33.3	27.4	26.5

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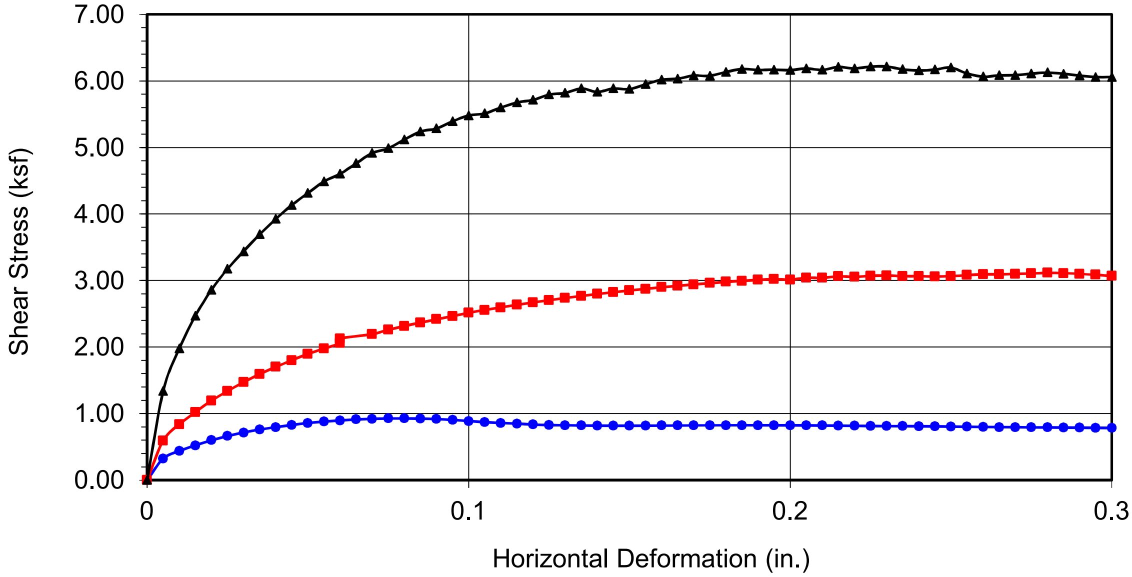
DIRECT SHEAR TEST RESULTS

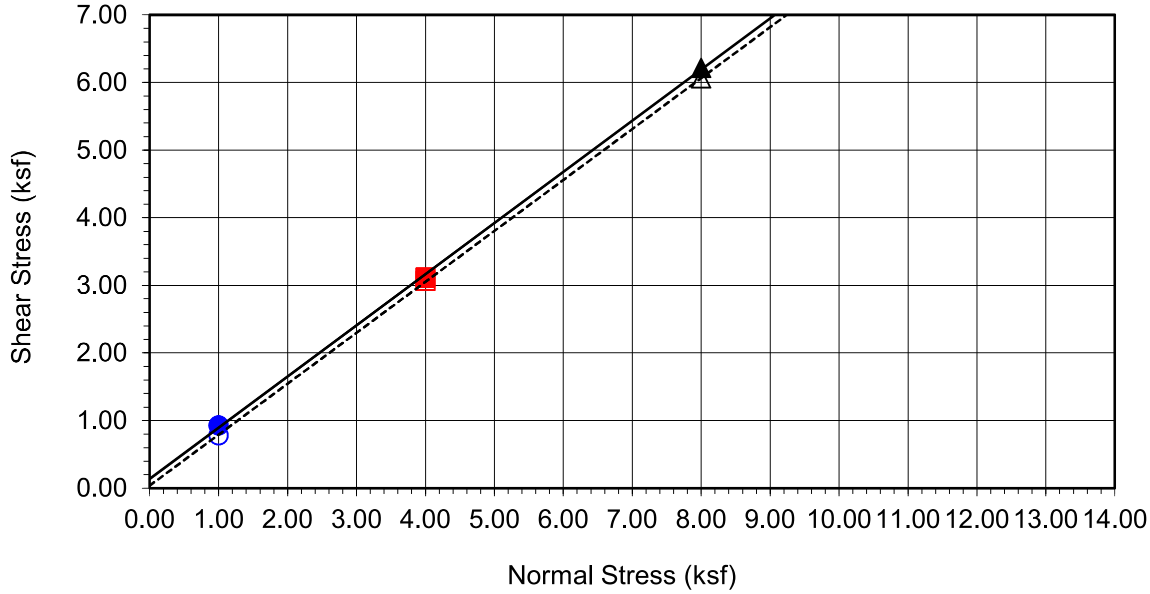
Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

03-22





Boring No.	**RW-2**
Sample No.	**R-2**
Depth (ft)	**7.5**
Sample Type:	Ring
Soil Identification: Olive gray silty clay (CL-ML)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	141	37
Ultimate	39	37

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.927	■ 3.115	▲ 6.215
Shear Stress @ End of Test (ksf)	○ 0.783	□ 3.068	△ 6.058
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 (%)	16.41	16.41	16.41
Dry Density (pcf)	88.5	93.5	102.6
Saturation (%)	49.0	55.1	69.0
Soil Height Before Shearing (in.)	0.9886	0.9564	0.9576
Final Moisture Content (%)	33.3	27.4	26.5

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

03-22

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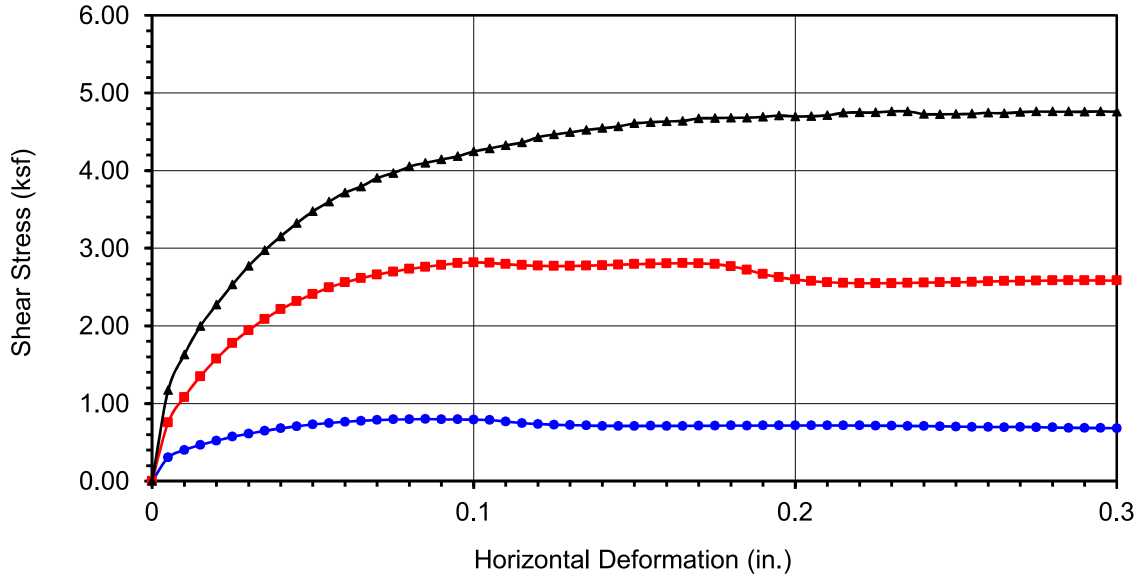
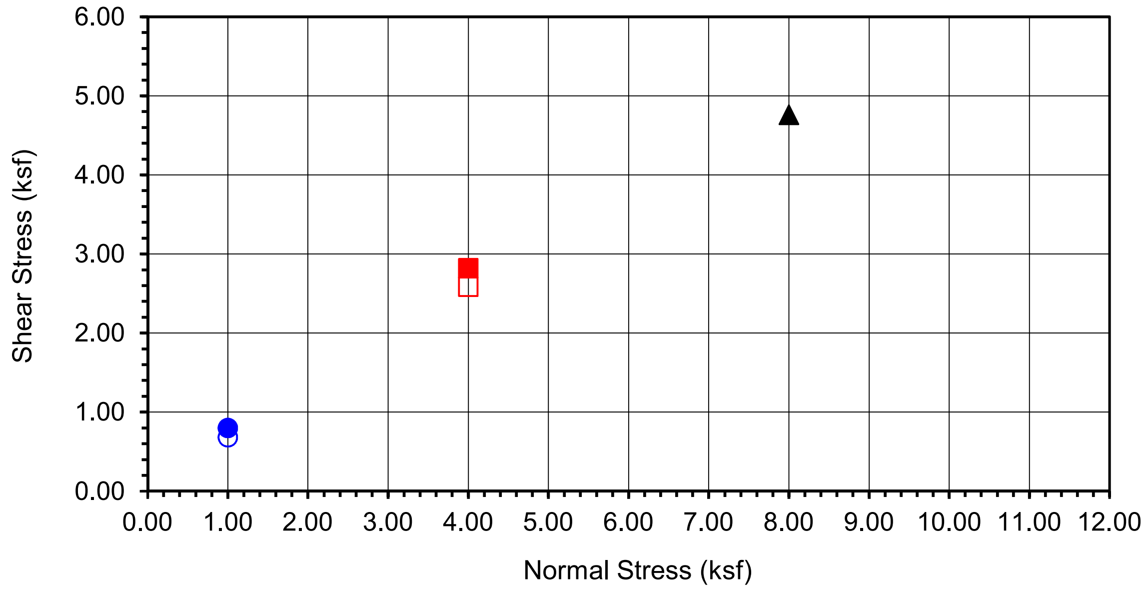
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Rexford Freeway Santa Fe Springs	Tested By:	G. Bathala	Date:	02/23/22
Project No.:	13429.001	Checked By:	A. Santos	Date:	02/28/22
Boring No.:	RW-2	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.):	15.0		
Soil Identification:	Olive 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):	190.43	193.42	195.63
Weight of Ring(gm):	42.10	43.98	45.58
Before Shearing			
Weight of Wet Sample+Cont.(gm):	203.92	203.92	203.92
Weight of Dry Sample+Cont.(gm):	180.04	180.04	180.04
Weight of Container(gm):	67.12	67.12	67.12
Vertical Rdg.(in): Initial	0.2226	0.2351	0.0000
Vertical Rdg.(in): Final	0.2392	0.2605	-0.0720
After Shearing			
Weight of Wet Sample+Cont.(gm):	211.71	206.16	207.98
Weight of Dry Sample+Cont.(gm):	182.12	178.94	180.49
Weight of Container(gm):	66.35	58.10	65.83
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**RW-2**
Sample No.	**R-4**
Depth (ft)	**15**

Sample Type:

Ring

Soil Identification:

Olive silty clay (CL-ML)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.799	■ 2.817	▲ 4.763
Shear Stress @ End of Test (ksf)	○ 0.682	□ 2.584	△ 4.757
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 (%)	21.15	21.15	21.15
Dry Density (pcf)	101.8	102.6	103.0
Saturation (%)	87.1	88.8	89.7
Soil Height Before Shearing (in.)	0.9834	0.9746	0.9280
Final Moisture Content (%)	25.6	22.5	24.0

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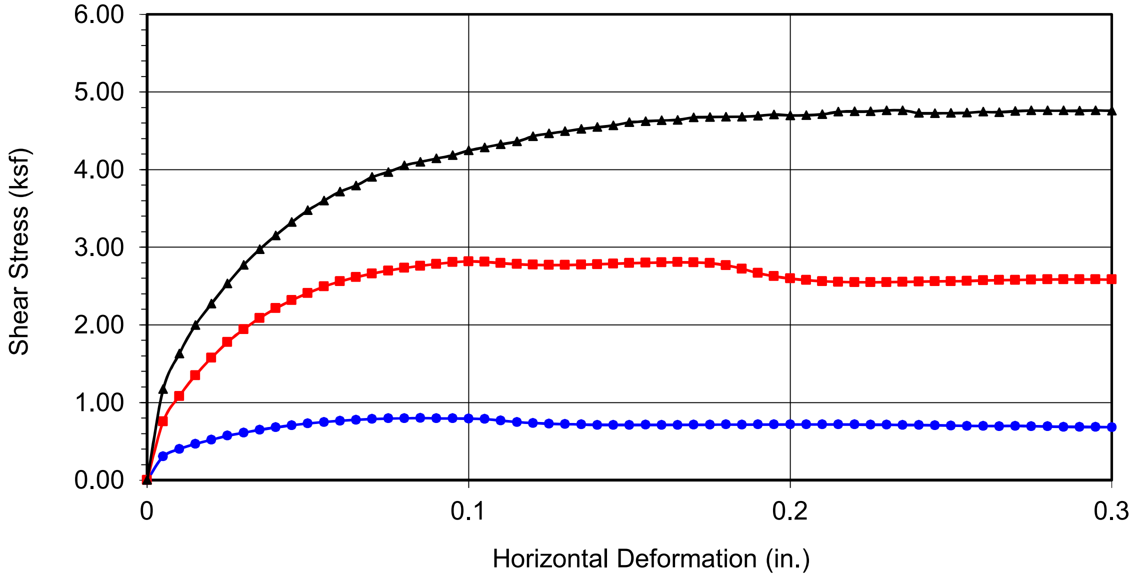
DIRECT SHEAR TEST RESULTS

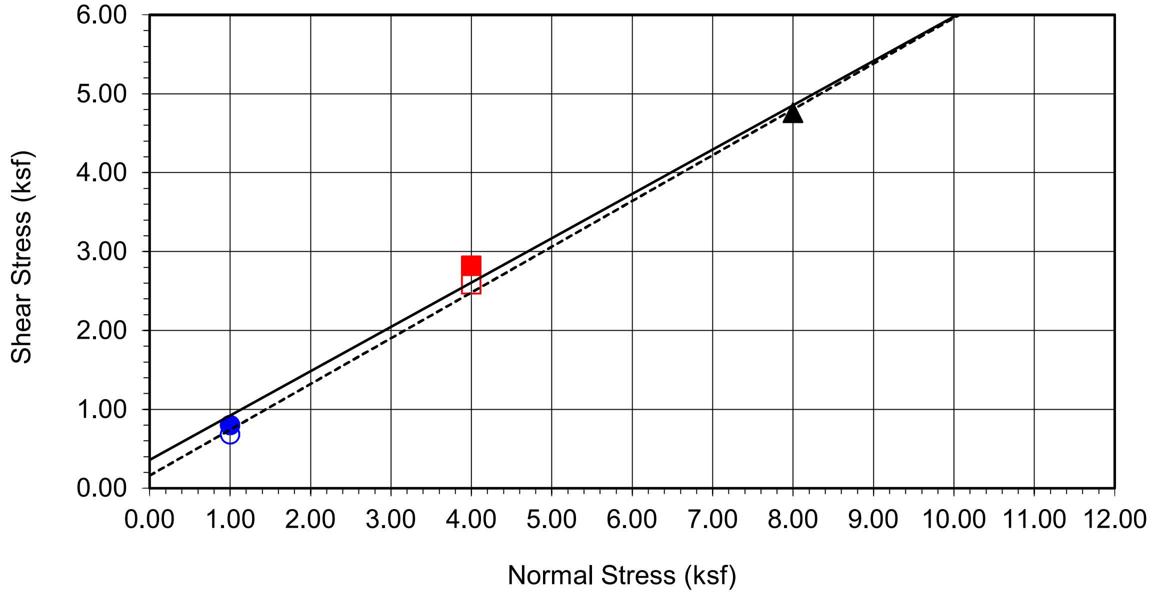
Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

02-22





Boring No.	**RW-2**
Sample No.	**R-4**
Depth (ft)	**15**
Sample Type:	Ring
Soil Identification: Olive silty clay (CL-ML)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	358	29
Ultimate	161	30

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.799	■ 2.817	▲ 4.763
Shear Stress @ End of Test (ksf)	○ 0.682	□ 2.584	△ 4.757
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 (%)	21.15	21.15	21.15
Dry Density (pcf)	101.8	102.6	103.0
Saturation (%)	87.1	88.8	89.7
Soil Height Before Shearing (in.)	0.9834	0.9746	0.9280
Final Moisture Content (%)	25.6	22.5	24.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13429.001

Rexford Freeway Santa Fe Springs

02-22

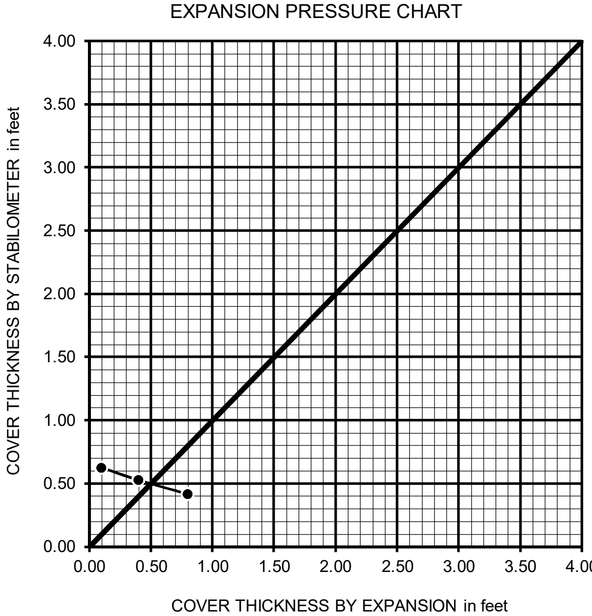
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Rexford Freeway Santa Fe Springs	PROJECT NUMBER:	13429.001
BORING NUMBER:	CPT-2	DEPTH (FT.):	0 - 5.0
SAMPLE NUMBER:	B-1	TECHNICIAN:	F. Mina
SAMPLE DESCRIPTION:	Olive brown sandy silt s(ML)	DATE COMPLETED:	3/8/2022

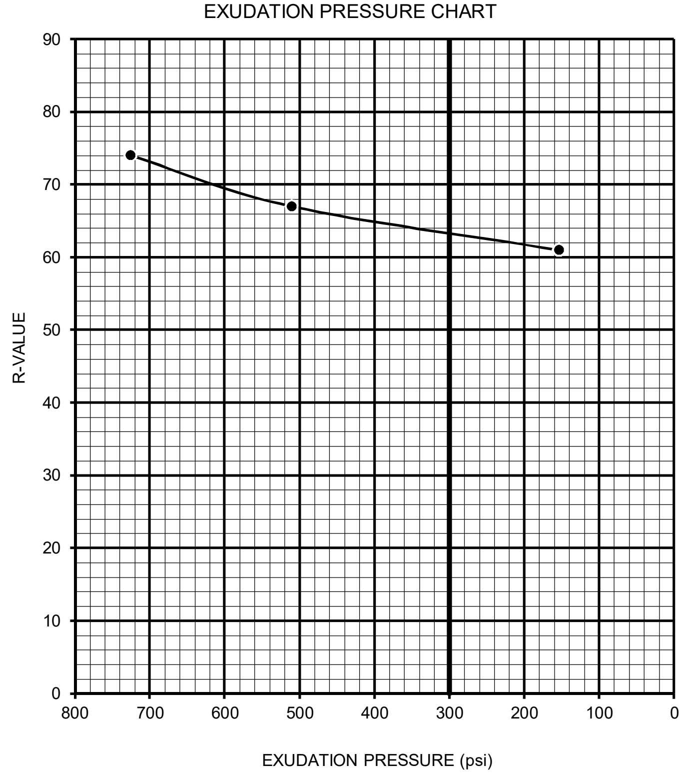
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.2	12.2	13.2
HEIGHT OF SAMPLE, Inches	2.54	2.52	2.55
DRY DENSITY, pcf	100.9	105.8	103.1
COMPACTOR PRESSURE, psi	350	350	350
EXUDATION PRESSURE, psi	726	510	154
EXPANSION, Inches x 10exp-4	24	12	3
STABILITY Ph 2,000 lbs (160 psi)	25	31	37
TURNS DISPLACEMENT	4.85	5.14	5.38
R-VALUE UNCORRECTED	74	67	61
R-VALUE CORRECTED	74	67	61

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.42	0.53	0.62
EXPANSION PRESSURE THICKNESS, ft.	0.80	0.40	0.10

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	69
R-VALUE BY EXUDATION:	63
EQUILIBRIUM R-VALUE:	63

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Rexford Freeway Santa Fe Springs	Tested By :	G. Berdy	Date:	03/01/22
Project No. :	13429.001	Checked By:	A. Santos	Date:	03/22/22

Boring No.	CPT-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown s(ML)			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.19			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	305			
Crucible No.	16			
Furnace Temperature (°C)	860			
Time In / Time Out	8:00/8:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.4782			
Wt. of Crucible (g)	18.4758			
Wt. of Residue (g) (A)	0.0024			
PPM of Sulfate (A) x 41150	98.76			
PPM of Sulfate, Dry Weight Basis	**99**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**60**			

pH TEST, DOT California Test 643

pH Value	**7.82**			
Temperature °C	22.6			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Rexford Freeway Santa Fe Springs	Tested By :	G. Berdy Date: 03/01/22
Project No. :	13429.001	Checked By:	A. Santos Date: 03/22/22
Boring No.:	CPT-2	Depth (ft.) :	0-5
Sample No. :	B-1		

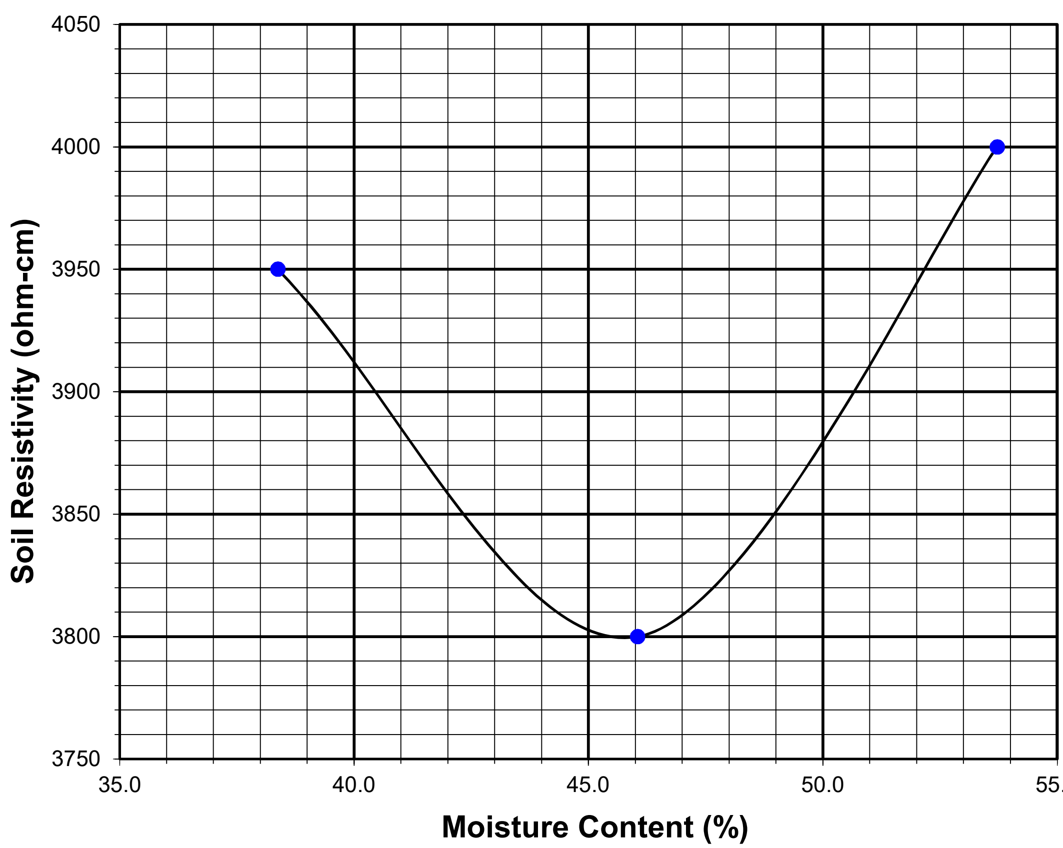
Soil Identification:* Olive brown s(ML)

*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	50	38.37	3950	3950
2	60	46.05	3800	3800
3	70	53.72	4000	4000
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.30
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	
3800	**45.8**	**99**	**60**	**7.82**	**22.6**



APPENDIX C

LIQUEFACTION ANALYSIS

Leighton

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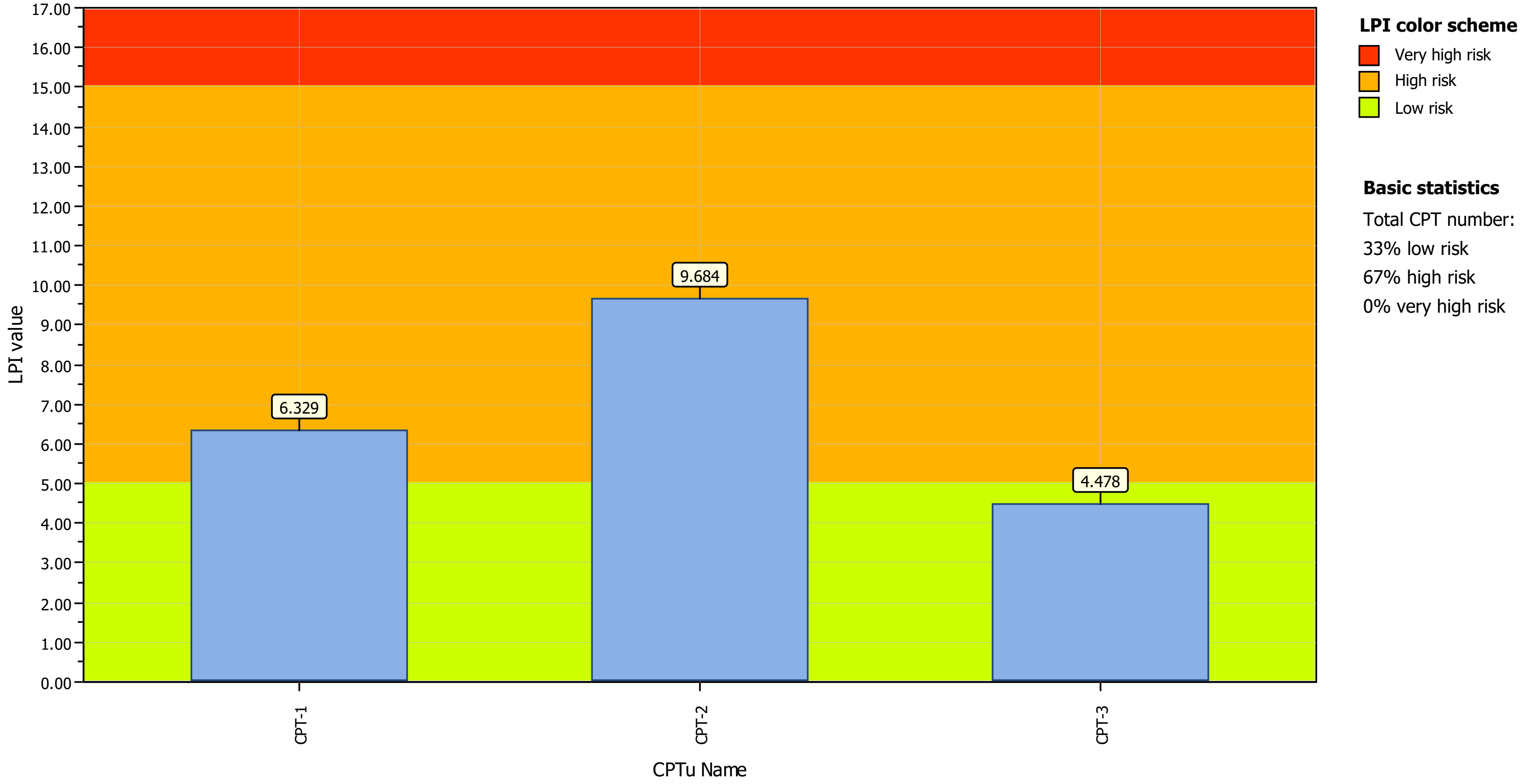
Irvine, CA 92614
949-250-1421
www.leightongroup.com

Project title : Rexford Freeway Dr

Location : 13711 Freeway Drive, Santa Fe Springs, California

Overall Liquefaction Potential Index report



Basic statistics

Total CPT number: 3

33% low risk

67% high risk

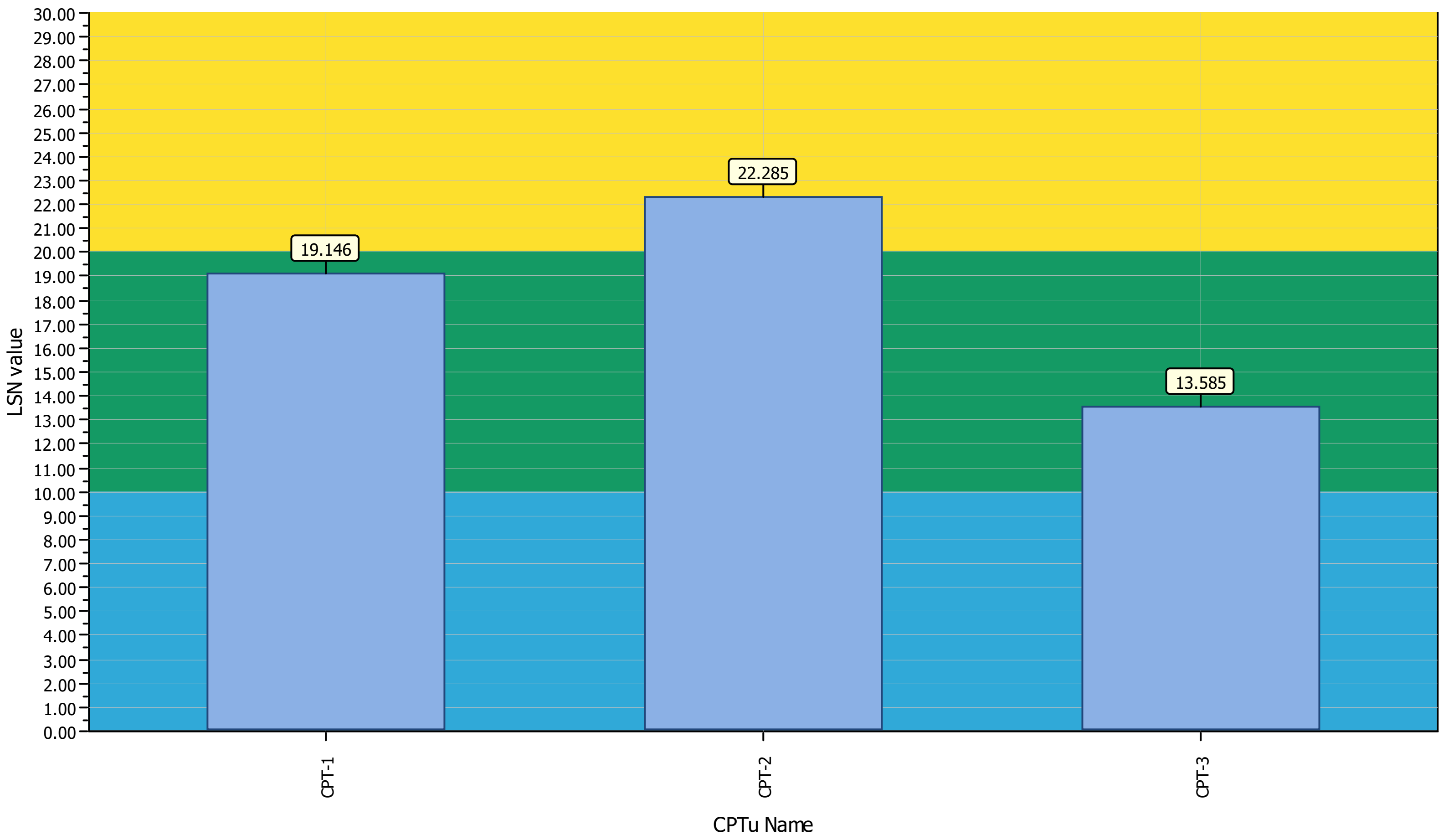
0% very high risk

17781 Cowan
Irvine, CA 92614
949-250-1421
www.leightongroup.com

Project title : Rexford Freeway Dr

Location : 13711 Freeway Drive, Santa Fe Springs, California

Overall Liquefaction Severity Number report



LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

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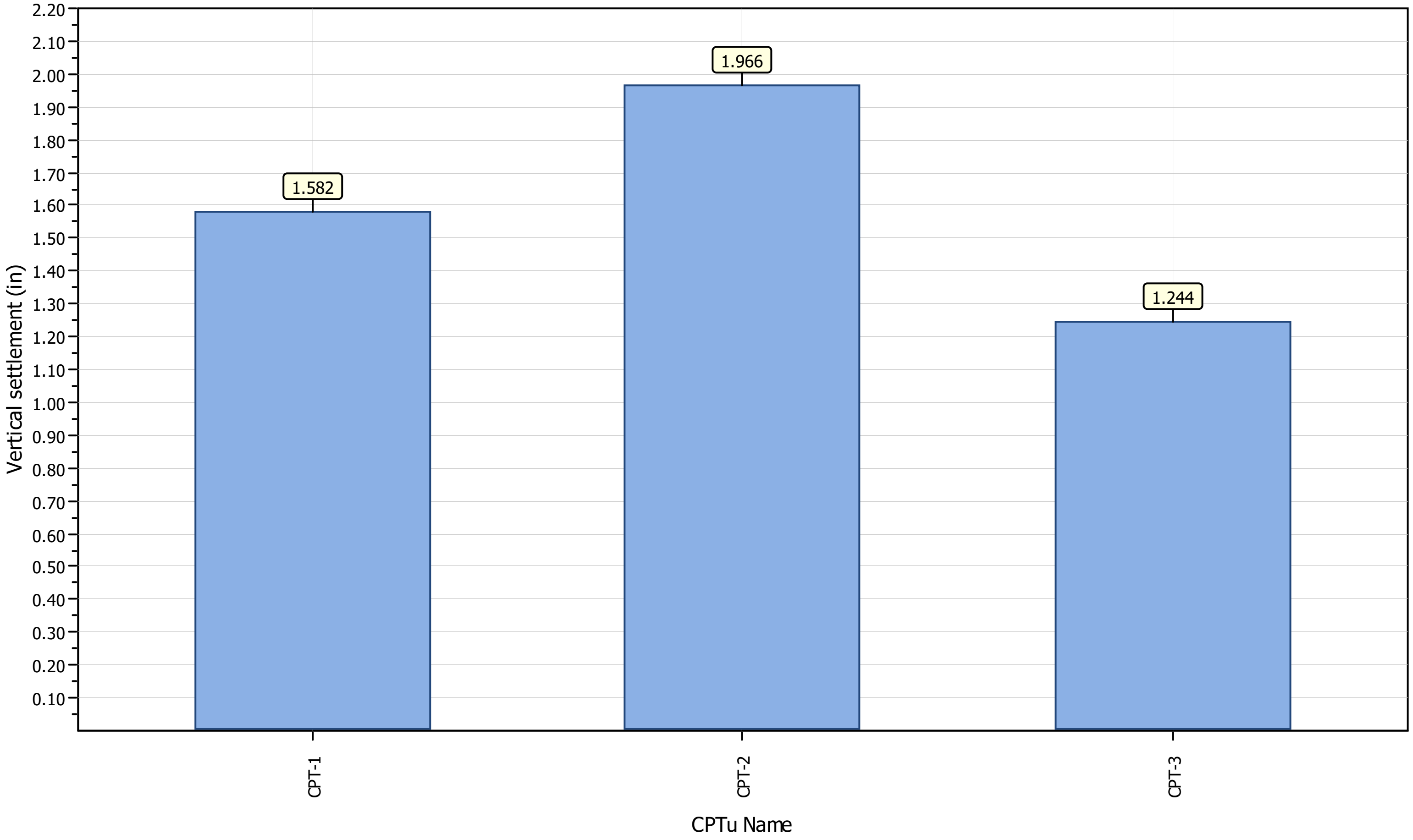
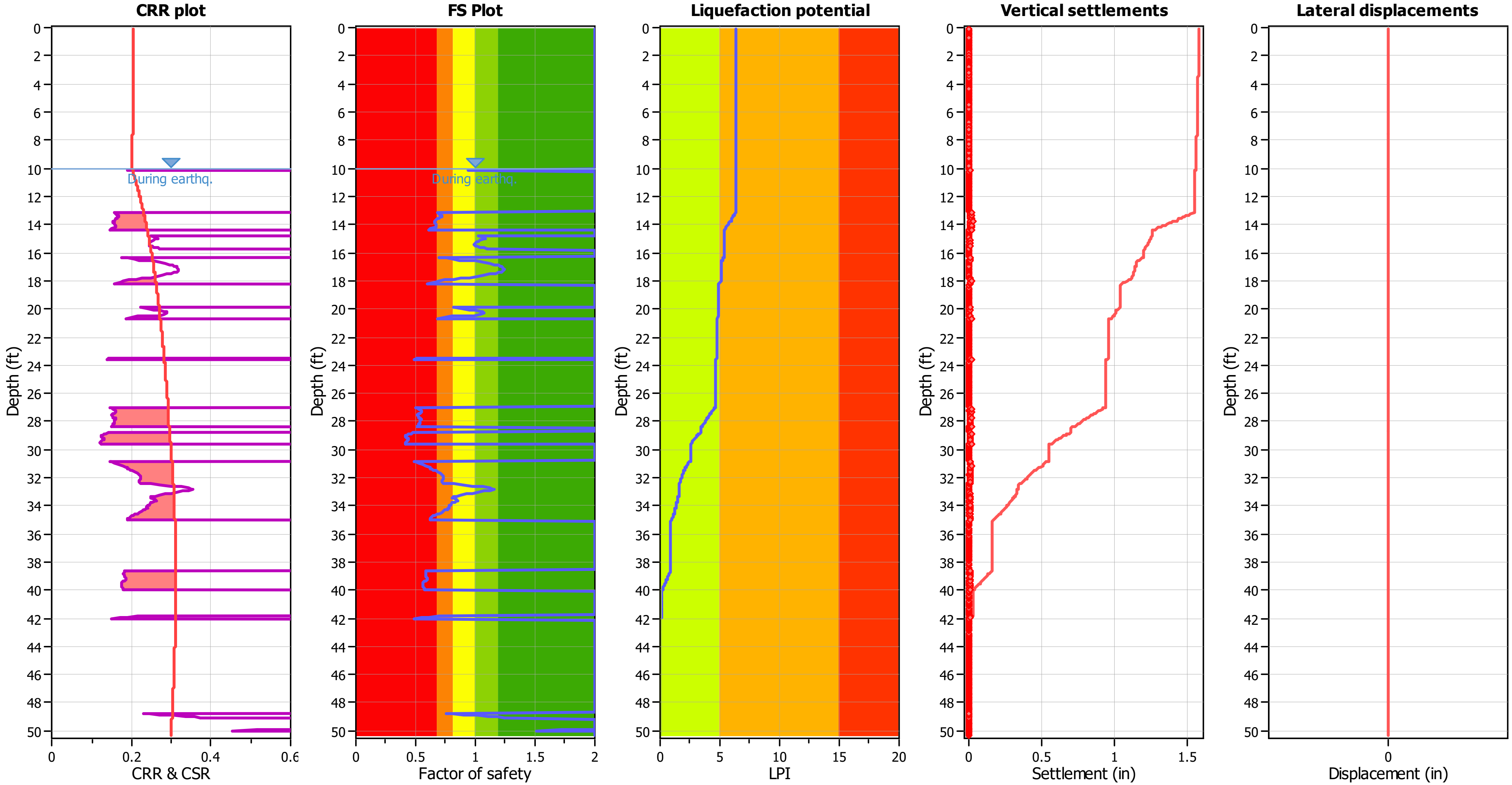
Overall vertical settlements report



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:	6.66	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

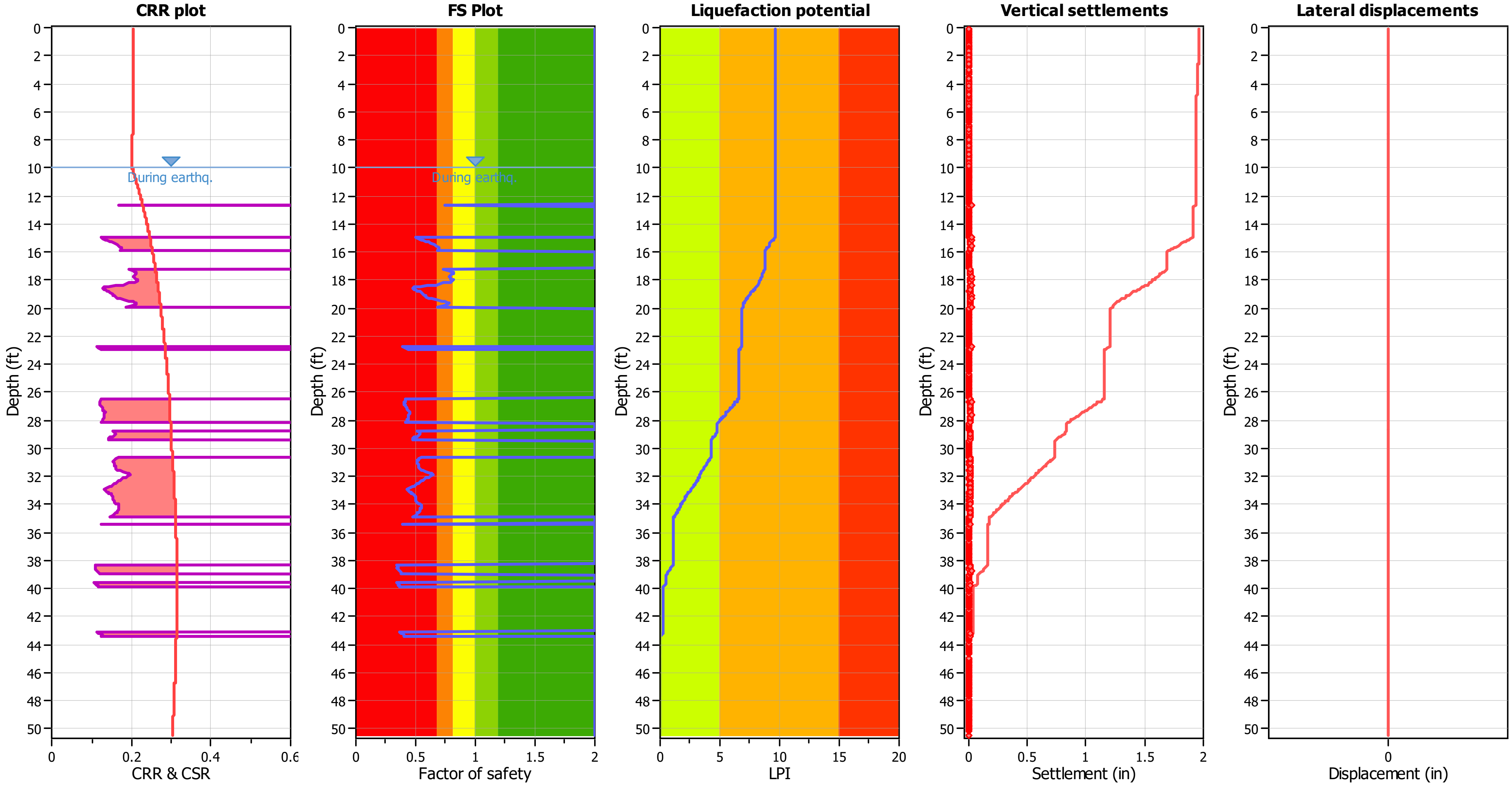
F.S. color scheme

- 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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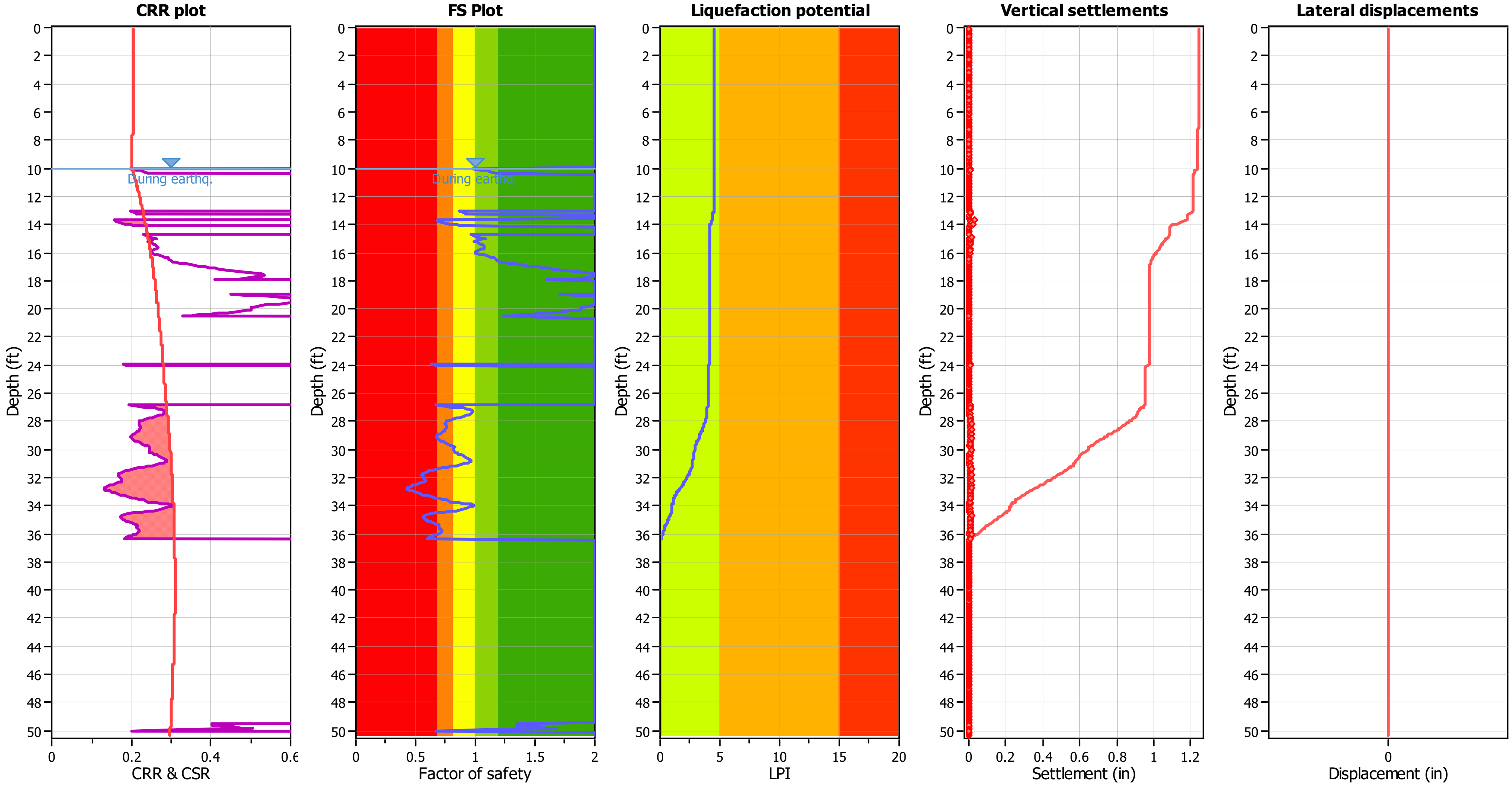

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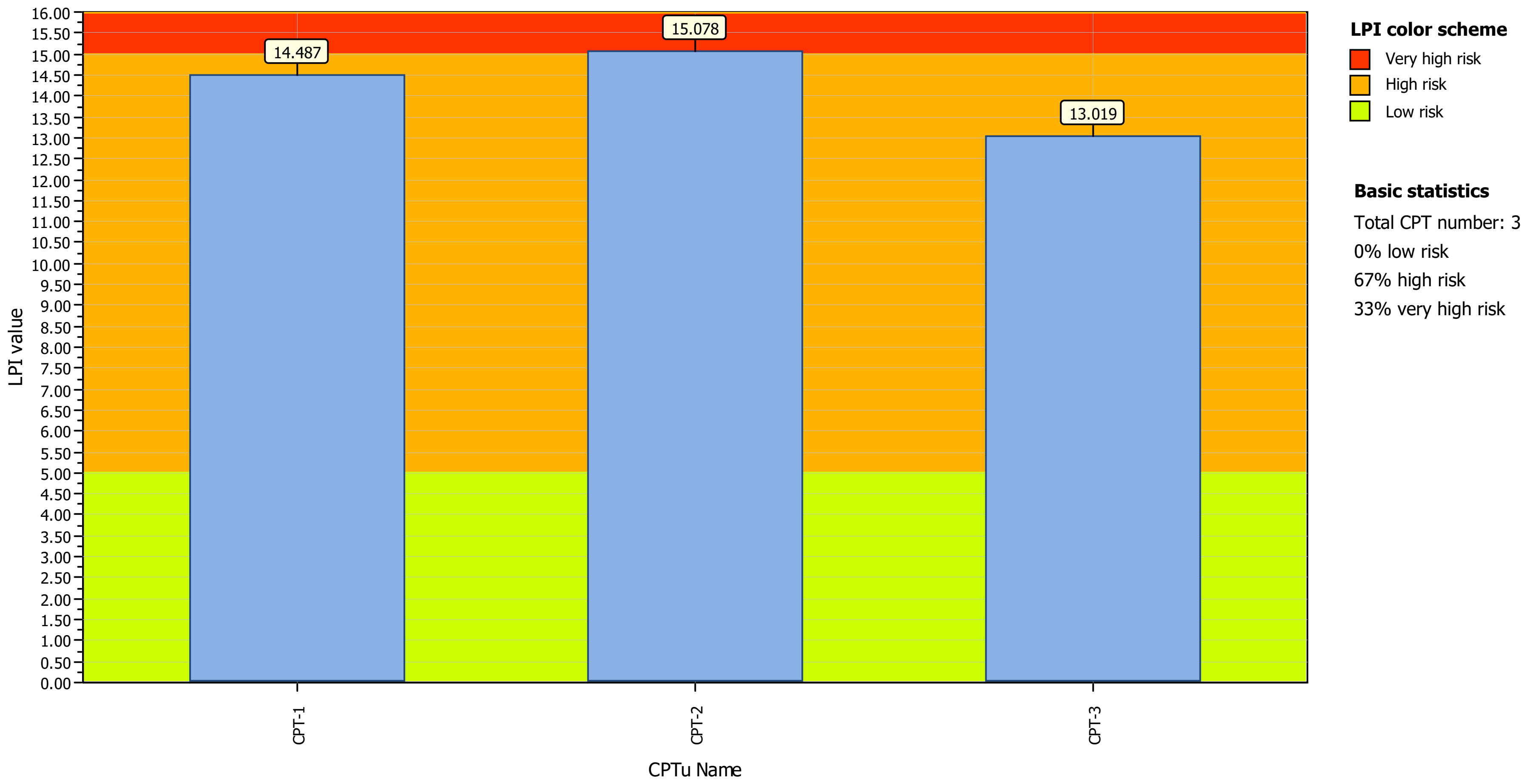

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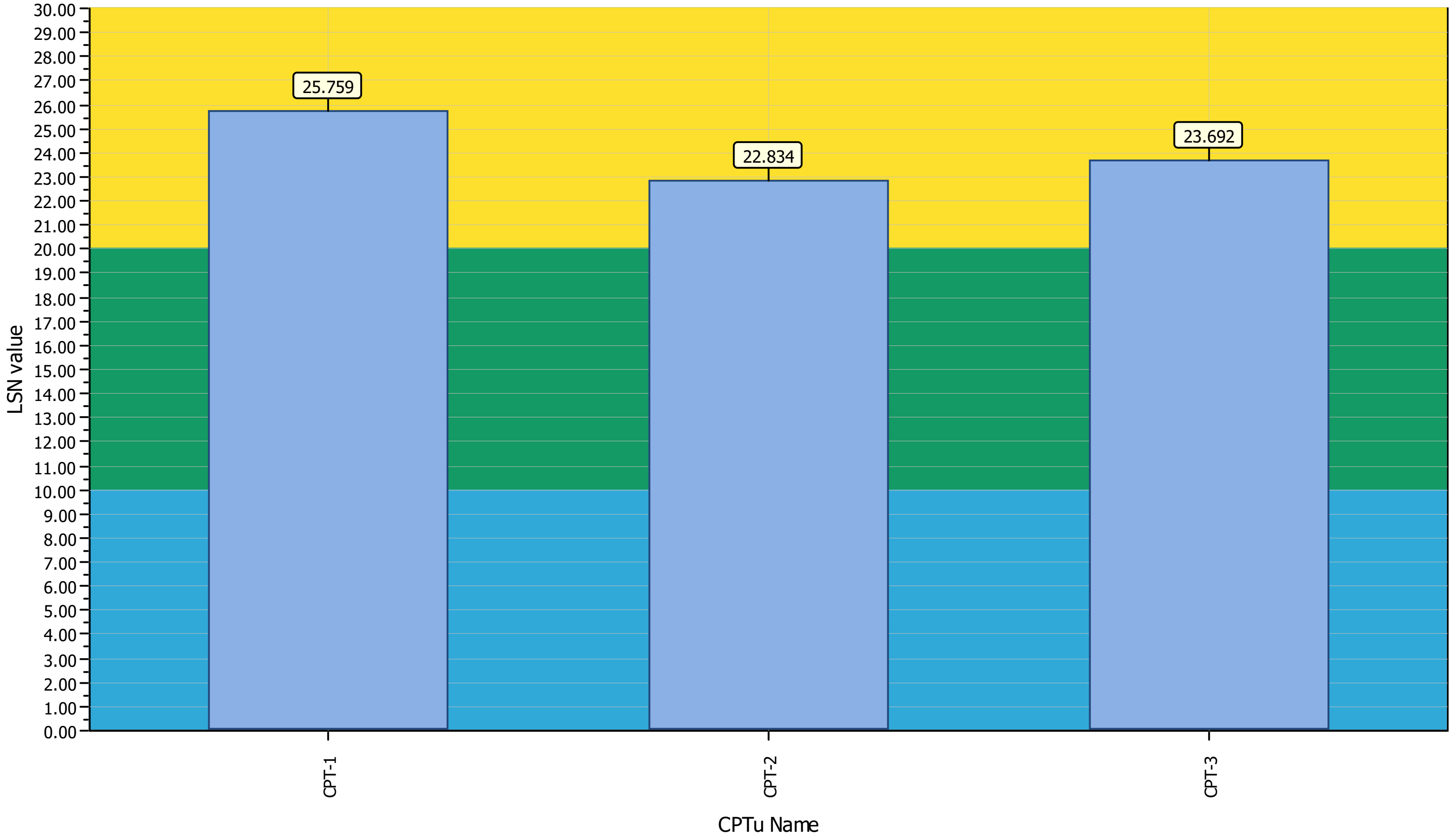

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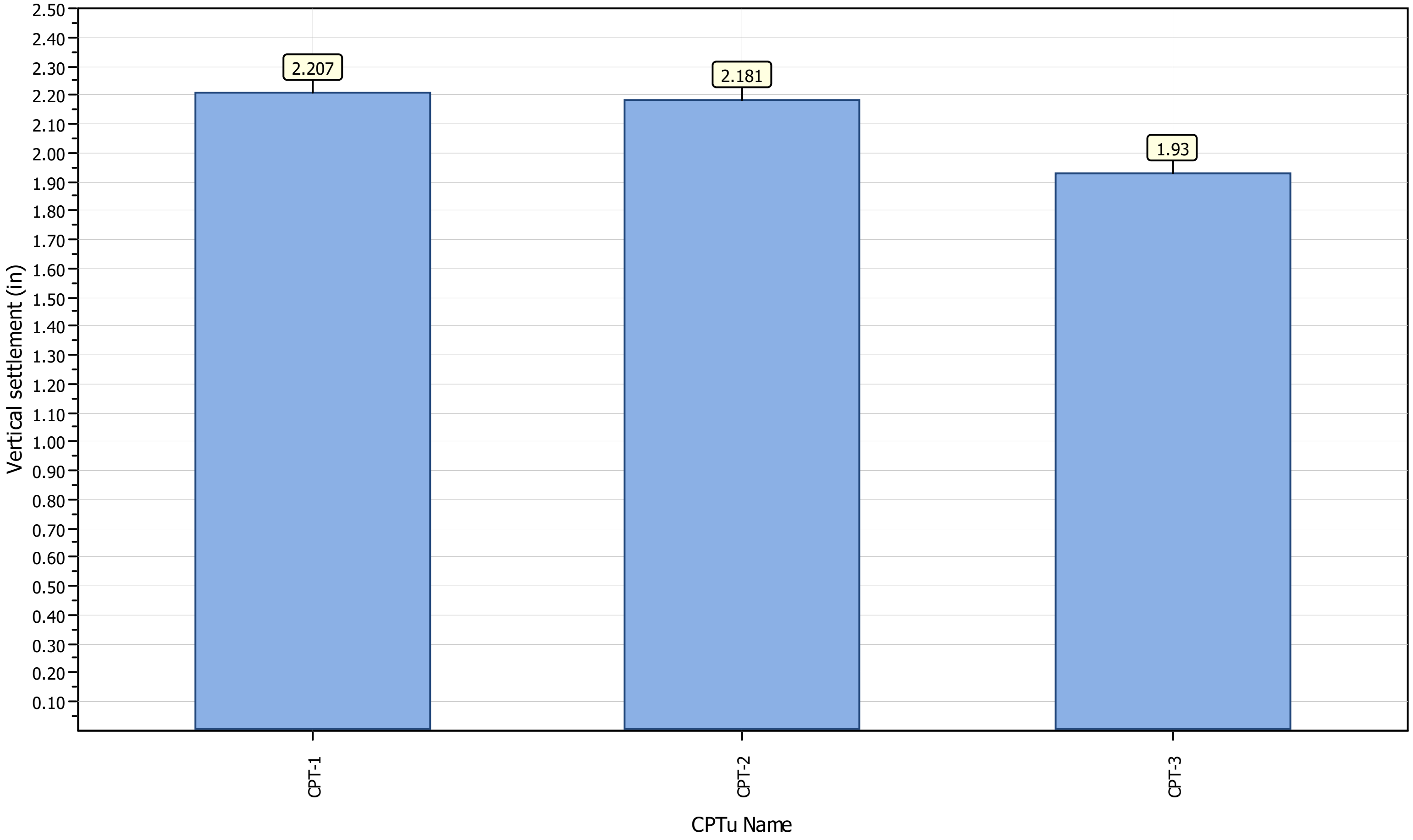
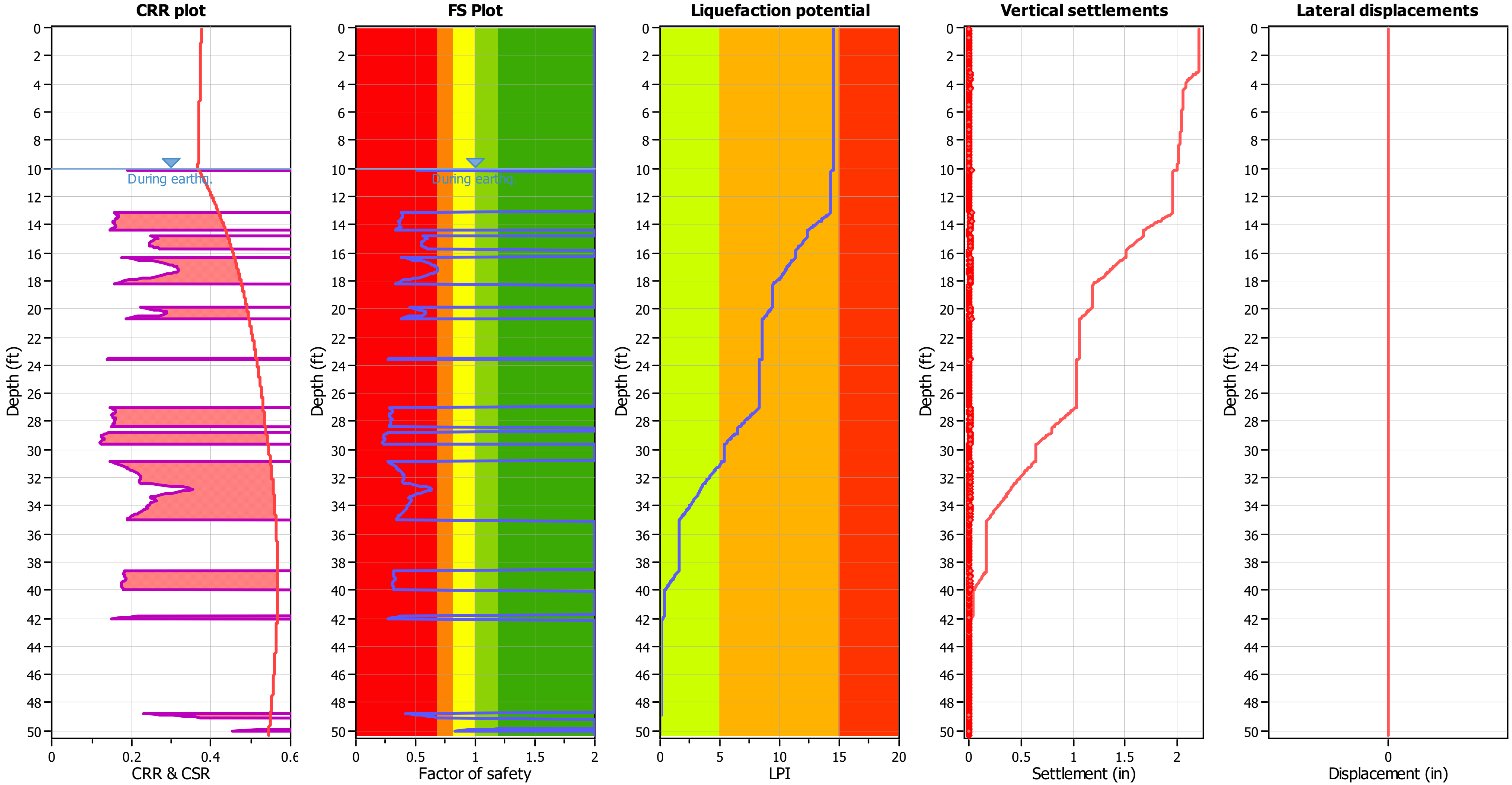


Liquefaction analysis overall plots



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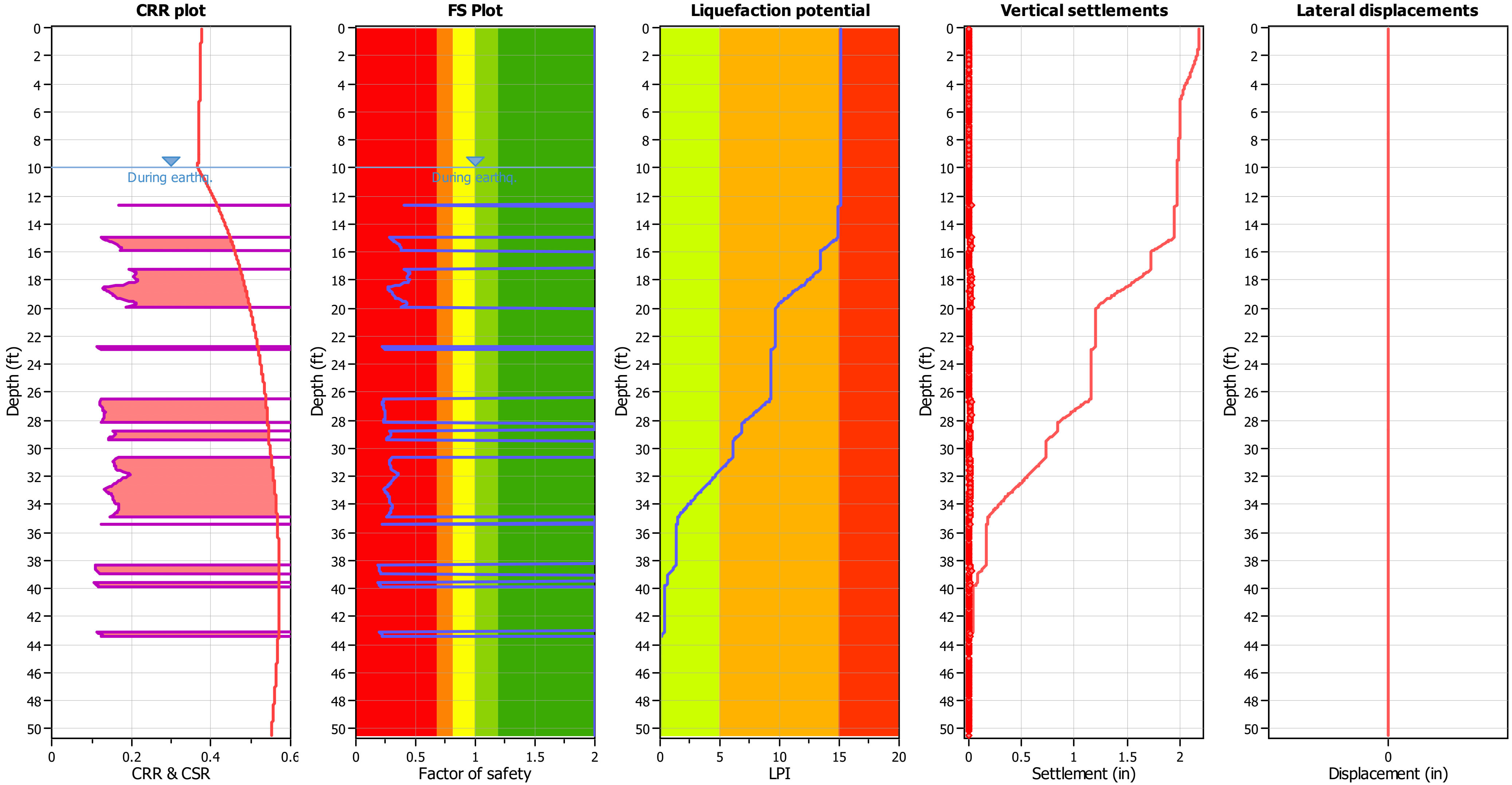

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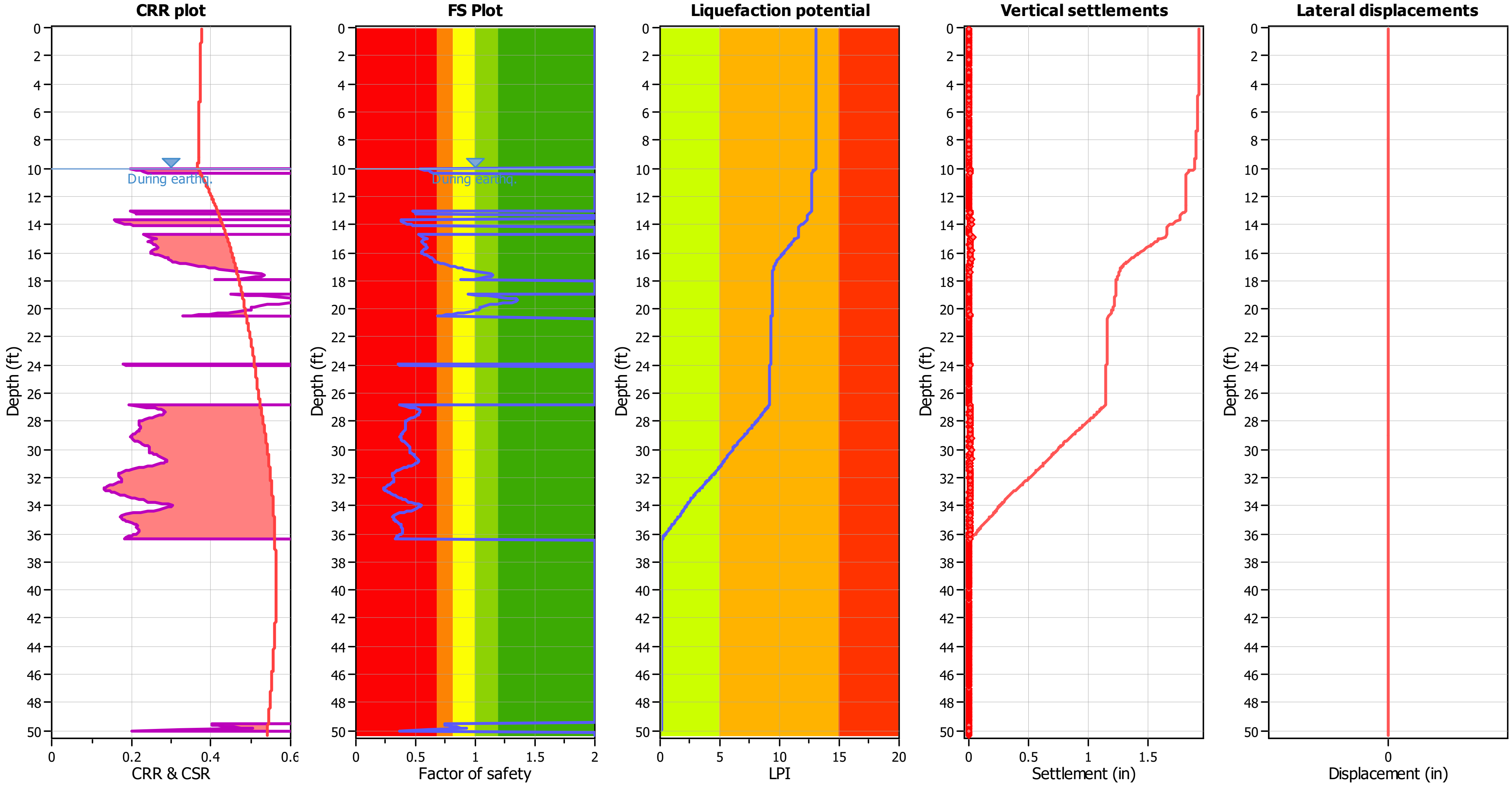

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APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

D-1.0 GENERAL

D-1.1 <u>Intent</u>

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).

D-1.2 <u>Role of Leighton Consulting, Inc.</u>

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.

D-1.3 <u>The Earthwork Contractor</u>

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.

D-2.0 PREPARATION OF AREAS TO BE FILLED

D-2.1 Clearing and Grubbing

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.

D-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.

D-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

D-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.

D-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.

D-3.0 FILL MATERIAL

D-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.

D-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.

D-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.

D-4.0 FILL PLACEMENT AND COMPACTION

D-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.

D-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.

D-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.

D-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.

D-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).

D-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.

D-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..

D-6.0 TRENCH BACKFILLS

D-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).

D-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 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 2018 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..

D-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.