

Appendix C: Geotechnical Evaluation



**GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
1500 S. RAYMOND AVENUE
FULLERTON, CALIFORNIA**

Prepared for **REXFORD INDUSTRIAL REALTY &
MANAGEMENT, INC.**
11620 WILSHIRE BOULEVARD, SUITE 1000
LOS ANGELES, CALIFORNIA 90025

Prepared by **LEIGHTON CONSULTING, INC.**
17781 COWAN
IRVINE, CALIFORNIA 92614

Project Number 13477.001

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Project No. 13477.001

Rexford Industrial Realty & Management, Inc.
11620 Wilshire Boulevard, Suite 1000
Los Angeles, California 90025

Attention: Mr. Daniel Murphy

**Subject: Geotechnical Exploration Report
Proposed Industrial Building
1500 S. Raymond Avenue
Fullerton, California**

Per our March 7, 2022 proposal, authorized on March 14, 2022, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. We understand the proposed development will include demolition of the existing building and site improvements to accommodate construction of a new one-story industrial building with a total building area of 138,500 square feet. The proposed concrete tilt-up building will be constructed at grade with dock-high truck loading, Orange County Fire Department access and vehicular surface parking. 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 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.



Jeffrey M. Pflueger, PG, CEG 2499

Associate Geologist

Extension 4257, jpflueger@leightongroup.com



Carl C. Kim, PE, GE 2620

Senior Principal Engineer

Extension: 4262, ckim@leightongroup.com

MM/JMP/CCK/lr

Distribution: (1) Addressee

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 1500 S. Raymond Avenue in the city of Fullerton, Orange County, California. The site location (latitude 33.8551°, longitude -117.9047°) and immediate vicinity are shown on Figure 1, *Site Location Map*.

The project site is irregular in shaped and covers approximately 7.15 acres. The site is bordered by a railroad easement to the north, existing industrial properties to the east, and E. Burton Street to the south and west. The site is currently occupied by the existing and active Hotel Fullerton. The hotel consists of a 3-story building that wraps around a courtyard and pool area, with a 6-story tower located on the north side of the courtyard area. Asphalt paved parking and access surround the hotel. Access to the site is via E. Burton Street on the south and west.

The project site is relatively flat with sheet flow generally directed to the southwest. Review of the United States Geological Survey (USGS) 7.5-Minute Anaheim Quadrangle (USGS, 1981) indicates the ground surface at the site is between approximately Elevation (El.) +170 and +175 feet mean sea level (msl).

Based on review of historical aerial photographs (NETR, 2022), the site was an orchard until at least 1963. Between approximately 1963 and 1972, the hotel was constructed, with the exception of the 6-story tower, which was constructed between 1972 and 1980 along with the expansion of the existing parking lot to the north. The site generally appears to have remained in the same configuration since 1980.

Based on review of the *Conceptual Site Plan* (Sheet A1-0) dated March 14, 2022, we understand the proposed development consists of a new one-story industrial building with a total building area of 138,500 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. Orange County Fire Department access is planned on the northern and eastern sides of the building and vehicular surface parking is planned on the northern and western sides of the building. A truck yard for trailer parking and additional vehicular surface parking is planned in the northern portion of the site. Ancillary improvements likely consist of utility infrastructure, pavement,

flatwork, and landscaping. In addition, the *Conceptual Site Plan* indicates an existing north-south trending underground storm drain easement is located in the eastern portion of the site, and easements for sewer, drainage, public utility and railroad traverse the site at various locations in the northern portion of the site. Preliminary structural loading information was not available at the time of this report.

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 March 18, 2022, and included drilling, logging, and sampling a total of four (4) hollow-stem auger borings (designated LB-1 through LB-4) to approximate depths between 31½ and 51½ feet below the existing ground surface (bgs). Two (2) additional borings (designated LP-1 and LP-2) were each drilled to an approximate depth of 10 feet bgs for subsequent percolation testing. The approximate locations of the explorations are shown on Figure 2, *Exploration Location Map*. The boring logs 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 (non-cohesive) 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.

- Percolation Testing – Borings LP-1 and LP-2 were converted to temporary percolation test wells upon completion of drilling and sampling. The test wells consisted of 2-inch slotted (0.020”) PVC well casing surrounded by #3 Monterey Sand placed in the annulus of the well within the test zone. In-situ percolation testing was performed on March 21, 2022 in general accordance with the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013). The results of the percolation testing are presented in Appendix B, *Percolation Test Data*. Refer to the discussion of infiltration rate presented in Section 2.4.1, *Infiltration*. Upon completion of the percolation testing, the well casing was removed from each boring and the borings were backfilled with soil cuttings and patched at the surface with cold-mix asphalt concrete to match existing site 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);
 - Direct Shear (ASTM D 3080);

-
- Consolidation (ASTM D 2435);
 - Maximum Dry Density (ASTM D 1557);
 - Expansion Index (ASTM D 4829);
 - R-value; 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 C, *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 project site is located on the lowest reach of the Santa Ana River basin within the Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province extends southward from the Los Angeles Basin to the tip of Baja California (Yerkes et al., 1965) and is characterized by elongated northwest-trending mountain ranges separated by sediment-floored valleys. The most dominant structural features of the province are the northwest trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province. East of the site are the northwest-trending Santa Ana Mountains, a large range which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea.

The area south and west of the Santa Ana Mountains is generally characterized as a broad, complex, alluvial fan, which receives sediments from the Santa Ana River and its tributaries draining the Santa Ana and San Bernardino Mountains. These sediments are comprised of relatively flat-lying, unconsolidated to loosely consolidated clastic deposits that are approximately 3,000 feet thick beneath the site (Sprotte et al., 1980; Real, 1985).

2.2 Surficial Geology

The surficial deposits at the site are mapped to primarily consist of Quaternary-aged (Holocene to Late Pleistocene) young alluvial fan deposits (Bedrossian and Roffers, 2010; Morton and Miller, 2006). The young alluvial deposits consist of unconsolidated to slightly consolidated deposits of boulders, cobbles, gravel, sand, and silt from a confined valley or canyon. The surficial geologic units mapped in the vicinity of the project site are presented 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 young alluvial fan deposits (Qyf). The artificial fill encountered in our borings at the explored locations is generally between 2 to 3 feet in thickness across the site, likely associated with the existing and previous site improvements. The fill soils consist primarily of locally derived sand, silty sand, clayey sand and sandy clay.

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 (51½ feet bgs). The alluvial sediments generally consist of tan to dark brown, moist, very loose to very dense, sand with isolated layers of silty sand and clayey sand.

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. Test results indicate an Expansion Index (EI) value of 8 (“very low” potential for expansion). The Expansion Index laboratory test results are included in Appendix C 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 C of this report.

The test results indicate soluble sulfate concentrations of 185 parts per million (ppm), chloride content of 80 ppm, pH values of 8.11, and minimum resistivity value of 2,170 ohm-cm.

The results of the resistivity test indicate the underlying soil is severely corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate content 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) near-surface samples of the onsite soils recovered from the borings were remolded to 90 percent relative compaction and 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 exhibit low to moderate compressibility potential. The results of testing are presented in Appendix C.

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 C as well as summary graphs that provide values of angle of internal friction (ϕ) 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 not encountered at the site during our subsurface exploration performed at the site to the maximum depth of 51½ feet bgs. Based on review of the *Seismic Hazard Zone Report for the Anaheim and Newport Beach Quadrangles* (CGS, 1997), the historically shallowest groundwater depth at the site is greater than 50 feet bgs. Based on review of available groundwater information from the California Department of Water Resources for a nearby groundwater monitoring well located approximately ½ mile southwest of the project site (Well Name: AMD-3/MP5), the shallowest groundwater level measured for a monitoring period between May 1991 and April 2022 was approximately 80.1 feet bgs.

Based on these findings, groundwater is not expected to pose a constraint during or after construction. 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.

2.4.1 **Infiltration**

Percolation testing was performed in temporary wells installed within borings LP-1 and LP-2 located in the northern portion of the site for a test zone at depths of approximately 5 to 10 feet bgs to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with Orange County guidelines (OCPW, 2013). Results of the percolation testing are presented in Appendix B. The test locations and zones tested are shown on Figure 2.

A boring percolation test is useful for field measurements of the infiltration rate of soils and is suited for testing when the design depth of the infiltration device is deeper than current existing grades, especially in areas where it is difficult to dig test pits, or where the depths of these test pits would be considerably deep. At the subject site, testing consisted of advancing the borings to general depths anticipated for the invert of typical infiltration devices.

The percolation tests were performed using a constant-head method similar to the well permeameter test method (USBR 7300-89), which records the volumetric flow rate of water delivered to the test zone while maintaining a

relatively constant height of water in the well. Since the subsurface materials at this location were generally favorable for percolation (sandy soils), a water source was used to deliver water to the well at a relatively constant rate while recording the water height in the well. The measured infiltration rate for the constant-head percolation test was calculated by dividing the total volume of water infiltrated by the total duration of the test and dividing by the percolation surface area.

Detailed results of the field testing data and measured infiltration rate for the test well are presented in Appendix B. The test results are summarized in the table below:

Table 1 – Measured (Unfactored) Infiltration Rate

Test Well Designation	Approximate Depth of Test Zone (feet bgs)	Measured Infiltration Rate (inches per hour)
LP-1	5 to 10	24.3
LP-2	5 to 10	110.6

Based on the results of our field percolation testing that was performed at the site, the measured (unfactored) infiltration rates for the two (2) tests performed were 24.3 inches per hour (LP-1) and 110.6 inches per hour (LP-2), respectively. According to Orange County guidelines (OCPW, 2013), the measured infiltration rate at test wells LP-1 and LP-2 are well above the minimum feasibility criteria of 0.3 inch per hour.

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* (CGS, 2018; 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 considered 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, located approximately 5.8 miles from the

site. The San Andreas Fault, which is the largest active fault in California, is approximately 37.5 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 Historical 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 2 – 2019 CBC Based Ground Motion Parameters (Mapped Values)

Categorization/Coefficient	Code-Based
Site Latitude	33.8551°
Site Longitude	-117.9047°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_s	1.574 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.555 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}	1.574 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}	1.049 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	null ¹
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.738 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 Anaheim Quadrangle (CGS, 1998), the project site is **not** located within a liquefaction hazard zone as mapped by the State of California (Figure 5, *Seismic Hazard Map*). In addition, the historically shallowest groundwater is greater than 50 feet bgs. Based on these considerations, the potential for liquefaction at the site is considered negligible.

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.

Based on our evaluation of the alluvial site soils, the total seismically-induced settlement is estimated to be about ½ inch. The differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

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 liquefaction is not considered a hazard at the site and the site is relatively flat and constrained laterally, the potential for earthquake-induced lateral spreading is also considered negligible.

2.10 **Earthquake-Induced Landsliding**

As shown on the *Seismic Hazard Zones* map for the Anaheim Quadrangle (CGS, 1998), the site is **not** mapped within a seismically-induced landslide hazard zone identified by the State of California (Figure 5, *Seismic Hazard Map*). Based on this consideration and since the site is relatively flat, the potential for seismically-induced landslides is not considered a hazard at the site.

2.11 **Flooding**

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2009), a majority of the project site is located within an area of 0.2 percent annual chance of a flood hazard (500-year flood hazard zone). In addition, the southwestern portion of the site is located within an area with a base flood depth of 1 foot (100-year flood hazard zone). As shown on Figure 6, *Flood*

Hazard Zone Map, a majority of the site is located within a 500-year flood hazard zone and the southwestern portion is located within a 100-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. According to information obtained from the State of California Division of Safety of Dams (DSOD) and shown on Figure 7, *Dam Inundation Map*, a majority of the site is mapped within a dam inundation zone associated with Prado Dam. However, due to the location and distance of the site from Prado Dam, the potential for earthquake-induced flooding to occur due to a failure of this dam is considered low. Catastrophic failure of this dam is expected to be a very unlikely event in that dam safety regulations exist and are enforced by the DOSD, Army Corps of Engineers and Department of Water Resources. Inspectors may require dam owners to perform work, maintenance or implement controls if issues are found with the safety of the dam.

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 and there are no documented oil wells onsite (CalGEM, 2022). Based on these findings, methane is not considered a hazard at the site.

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 shallow spread-type foundations established in engineered fill soils. The floor slab may be supported directly on grade. There may be existing underground utilities that will also be impacted. Information on these utilities should be provided to Leighton for evaluation.

All existing undocumented fill is recommended to be removed from below the proposed building pad prior to placement of engineered fill. We estimate removal and recompaction of existing undocumented fill materials will be on the order of approximately 2 to 3 feet bgs. Localized areas in the unexplored portions of the site should be anticipated to require deeper removals, in particular beneath the existing buildings. In addition, overexcavation should be performed so that a minimum of 3 feet of engineered fill is established below the proposed foundation elements

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 Fullerton, the County of Orange, 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

Earthwork will consist of removal of unsuitable soil materials, overexcavation, and placement of compacted 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 C 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 Fullerton requirements, whichever is more stringent. 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.

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 soil, as deemed by the geotechnical engineer, should be removed to expose suitable native soils and replaced as engineered fill below the proposed building and other structural improvements. Based on our field explorations, we estimate removals of existing undocumented fill will be approximately 2 to 3 feet below existing grade across most of the site. Localized areas will require deeper removals. Unexplored portions of the site including areas beneath existing buildings and in areas of existing utilities, and areas disturbed during demolition of existing buildings and improvements may also require deeper removals.

Removals should be performed such that all undocumented fill is removed to expose suitable natural soils (alluvium) and replaced as engineered fill. In addition, overexcavations should be performed such that a minimum of 3 feet of engineered fill is established below the proposed building foundations. The lateral extent of overexcavation beyond foundations should be equal to the depth of removals below the proposed foundations. The depth of overexcavation in non-structural areas planned for new pavement and concrete slabs on grade is recommended to be 2 feet below

the current grade or planned subgrade elevation, whichever is lower, to develop a suitable bearing subgrade for 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 optimum moisture content, and then compacted to a minimum of **95** percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor).

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. 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 **95** 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 recompact 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. Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

3.2 Foundation Design

Conventional spread footings established in engineered fill or undisturbed natural soils may be used to support proposed building. Footings should be embedded a

minimum 18 inches below the lowest adjacent grade. An allowable soil bearing pressure of 3,000 pounds per square foot (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 recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads.

The ultimate bearing capacity can be taken as 9,000 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 allowable bearing capacity for shallow footings is based on a total static settlement of 1 inch. Differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

For static loading, 50 pounds per cubic inch (pci) may be assumed as the modulus of subgrade reaction (k). For seismic loading, a k value of 150 pci may be assumed.

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. Once developed by the structural engineer, we should review total dead and sustained live loads for each column including plan location and span distance, to evaluate if differential settlements between dissimilarly loaded columns will be tolerable. Excessive differential settlement can be mitigated with the use of reduced bearing pressures, deeper footing embedment, possibly changing overexcavation schemes and using imported base material under spread footings, or possibly other methods.

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, a passive pressure of 300 psf per foot of depth to a maximum of 3,000 psf 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.

3.3 **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 severely 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 8, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 3 – 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)	25

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. In addition to the recommended earth pressures, walls below grade adjacent to existing structures or streets and areas of traffic should be designed to accommodate surcharge loads. For traffic surcharge, a uniform lateral pressure of 100 pounds per square foot acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal traffic; the traffic surcharge load may be neglected provided a minimum of 10-foot clearance between the wall and the traffic is maintained. We will provide

surcharge loading from adjacent foundations after reviewing details of the planned basement walls in relation to existing foundations

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), 2021 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), 2021 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 a near surface samples of existing onsite soils indicate a value of 65.

Table 4 – 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 4,000 pounds per square inch.

Table 5 – 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 may be added to the paving 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 Infiltration BMP Design Considerations

It should be noted that the measured infiltration rates presented herein may degrade over time due to complete saturation of underlying soils, and fines build-up and plugging if pretreatment of the storm water is not performed. As such, a

reduction of the measured infiltration rates using a factor of safety of at least 2 or more should be considered to establish a conservative infiltration rate for the service life of the system. This factor should not be less than 2, but may be higher at the discretion of the design engineer.

In general, a vast majority of geotechnical distress issues are related to improper drainage. Distress in the form of foundation movement could occur. Direct infiltration to the subsurface is not recommended adjacent to curb and gutter, public pavements or within 10 feet away from the design saturation zone as soil saturation could lead to a loss of soil support, settlement or collapse, and internal erosion (piping). The design saturation zone may be assumed as a 1:1 plane projected downward from the top of an infiltration device's discharge zone. Additionally, infiltration water will migrate along pipe backfill (typically sand or gravel bedding) affecting improvements far from the point of infiltration. Proposed direct open bottom infiltration systems, should be located as far away from existing or proposed foundations, rigid improvements and utilities as is practical in order to reduce the geotechnical distress issues related to water. Where sufficient distance from improvements cannot be achieved, additional recommendations may be warranted and can be provided during plan review.

Prior to construction of any infiltration device intended for the site, the plans should be reviewed by the geotechnical consultant to verify that our geotechnical recommendations have been appropriately incorporated into the plans and not compromised by the addition of an infiltration system to the site. The designer of any infiltration system should contact the geotechnical consultant for geotechnical input during the design process as they feel necessary.

3.8 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 $\frac{3}{4}$ H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and $1\frac{1}{2}$ H:1V for Type C soils. Near-surface onsite soils are to be considered Type C 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.

3.9 **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 ($\leq$) 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 ($\geq$) 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.

3.10 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.

3.11 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.

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

- Grading and excavation of the site;
- 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 Orange County. We do not make any warranty, either expressed or implied.

5.0 REFERENCES

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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 site-wide 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

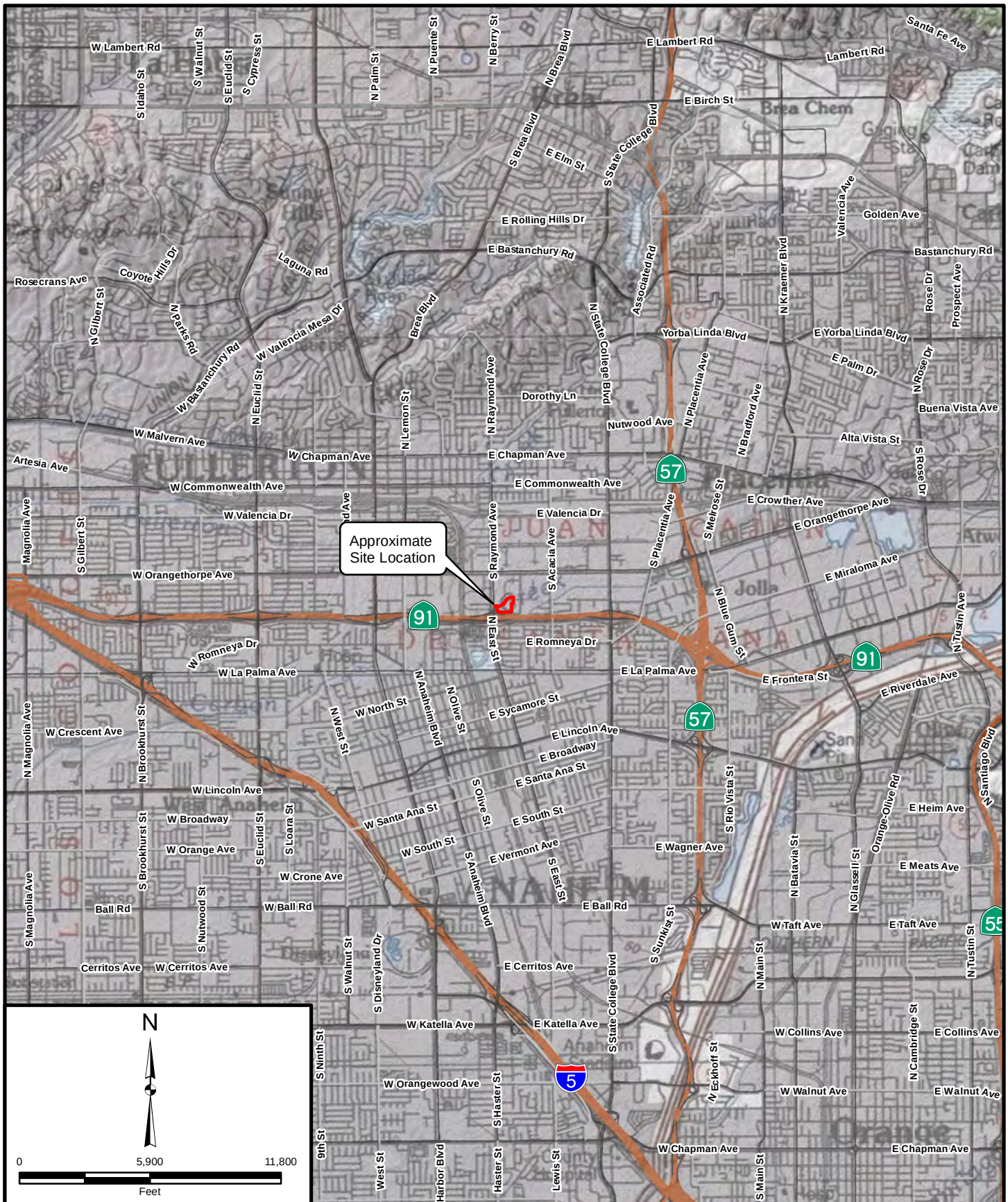
While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the 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

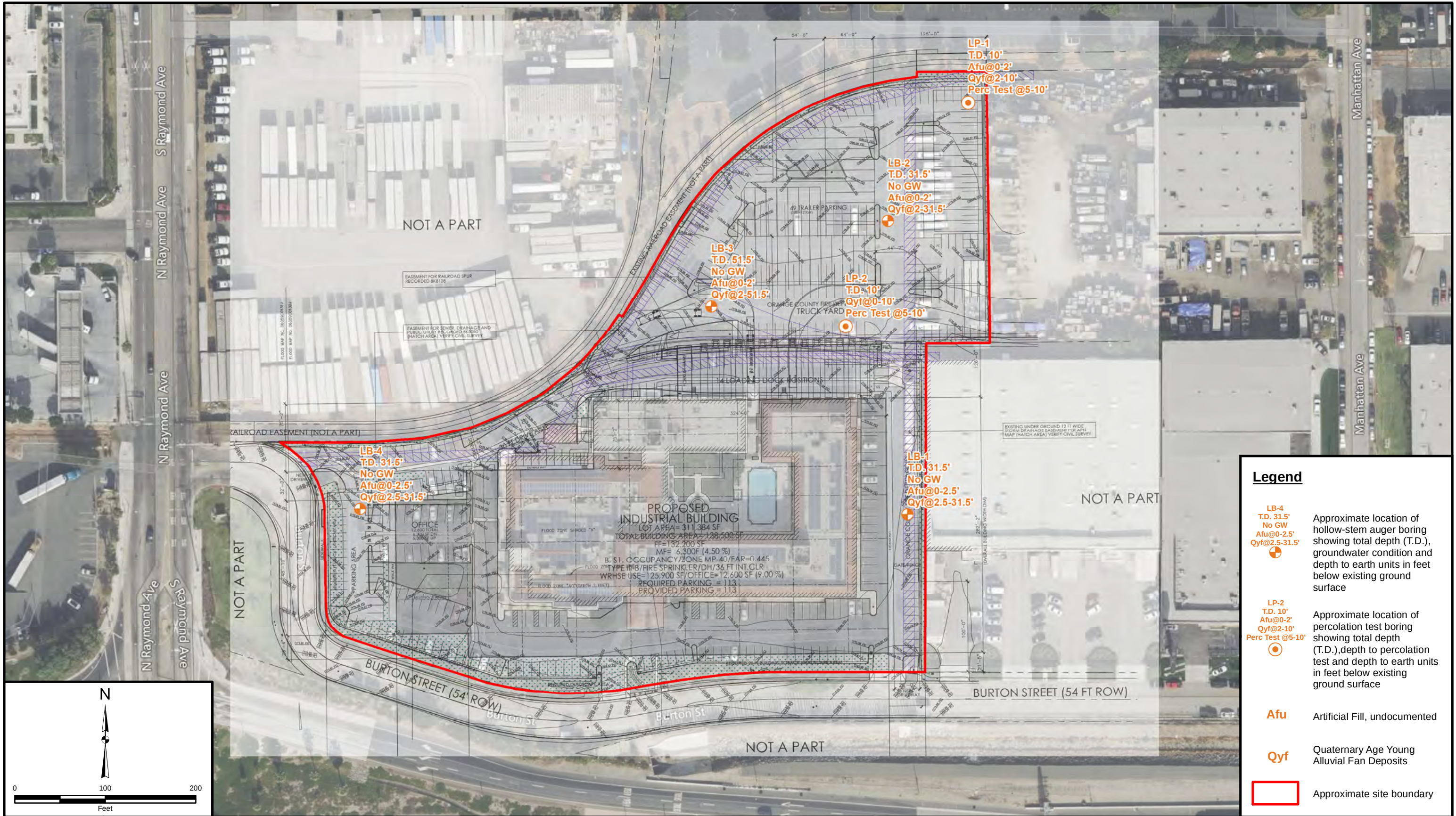


Project: 13477.001	Eng/Geol: CCK/JMP
Scale: 1" = 6,014'	Date: March 2022
Reference: Copyright: © 2013 National Geographic Society, i-cubed	

SITE LOCATION MAP Proposed Industrial Building 1500 South Raymond Avenue Fullerton, California

FIGURE 1



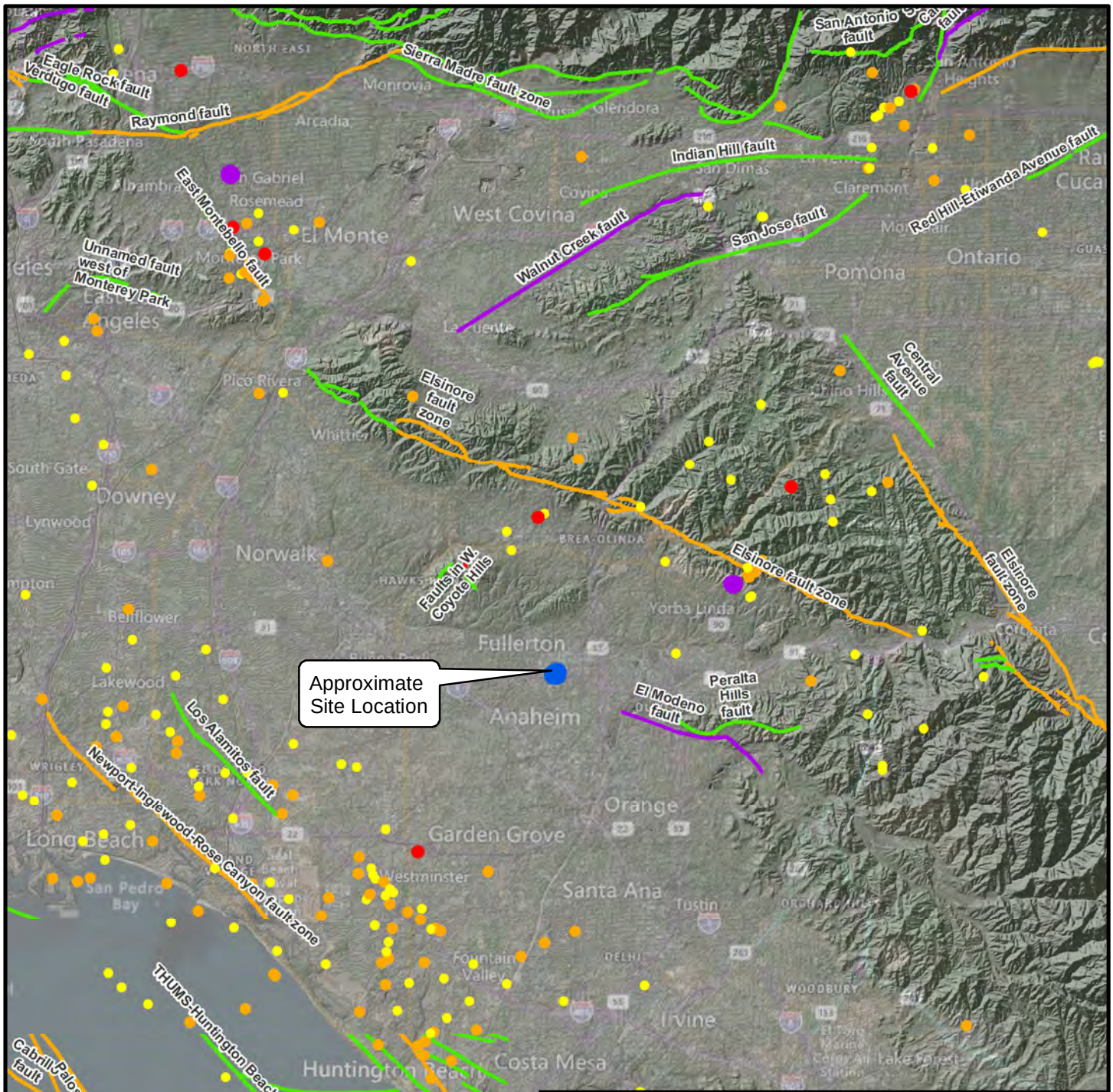


Project: 13477.001 Eng/Geol: CCK/JMP
Scale: 1" = 100' Date: May 2022
Reference: © 2022 Microsoft Corporation © 2022
Maxar ©CNES (2022) Distribution Airbus DS © 2022
TomTom

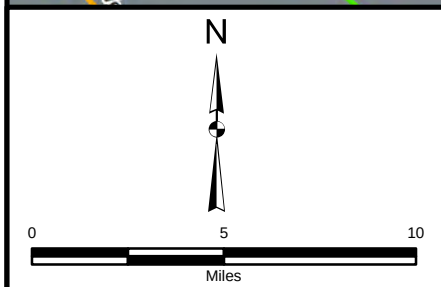
EXPLORATION LOCATION MAP

Proposed Industrial Building
1500 South Raymond Avenue
Fullerton, California





Approximate
Site Location



Legend

Fault activity

Recency of Movement

- Historic (<200 years)
- Holocene (<11,700 years)
- Late Quaternary (last 700,000 years)
- Quaternary (<1.6M years)

Historical Earthquakes ($\geq M3.5$)

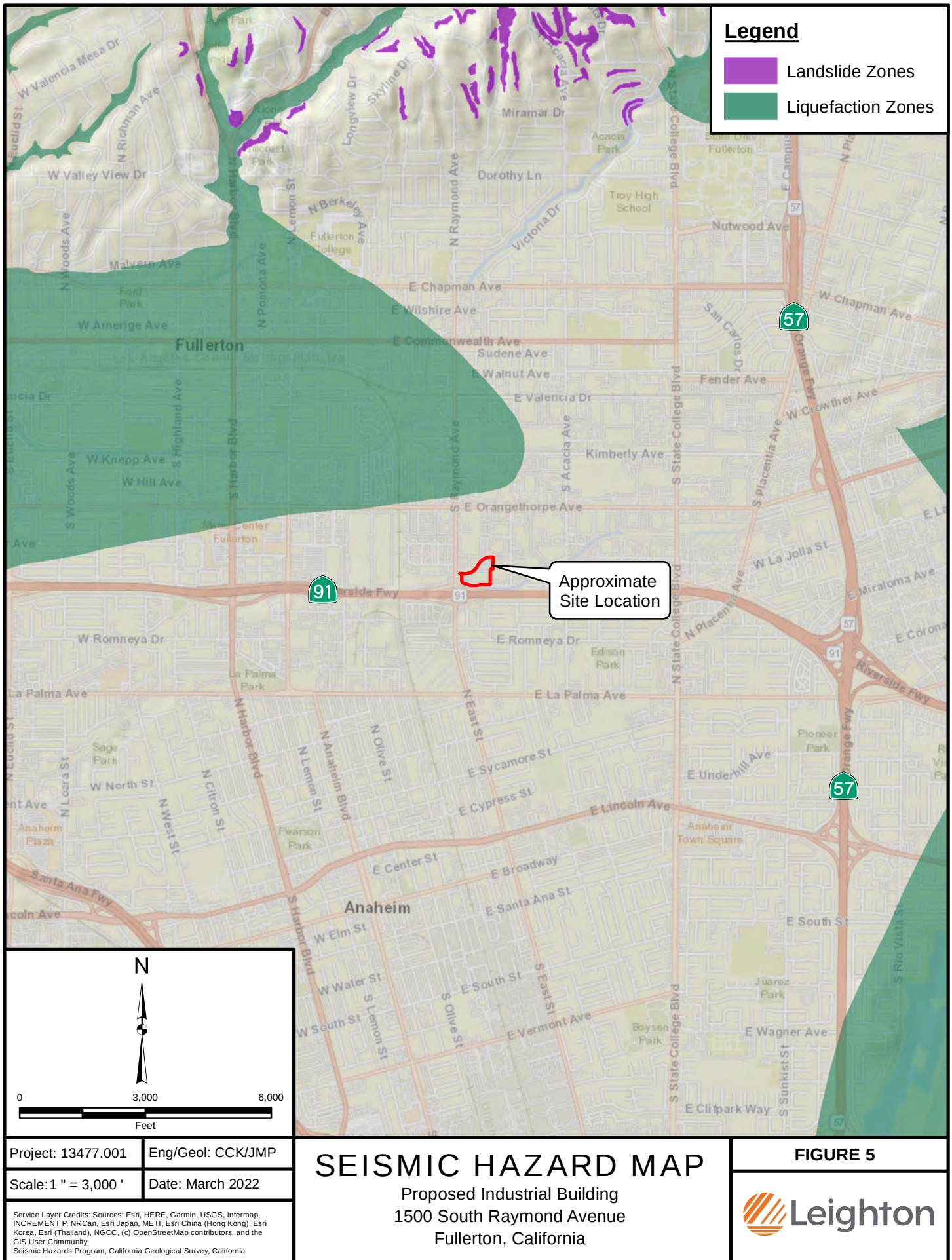
- 3.5 - 3.99
- 4.0 - 4.99
- 5.0 - 5.99
- 6.0 - 6.99

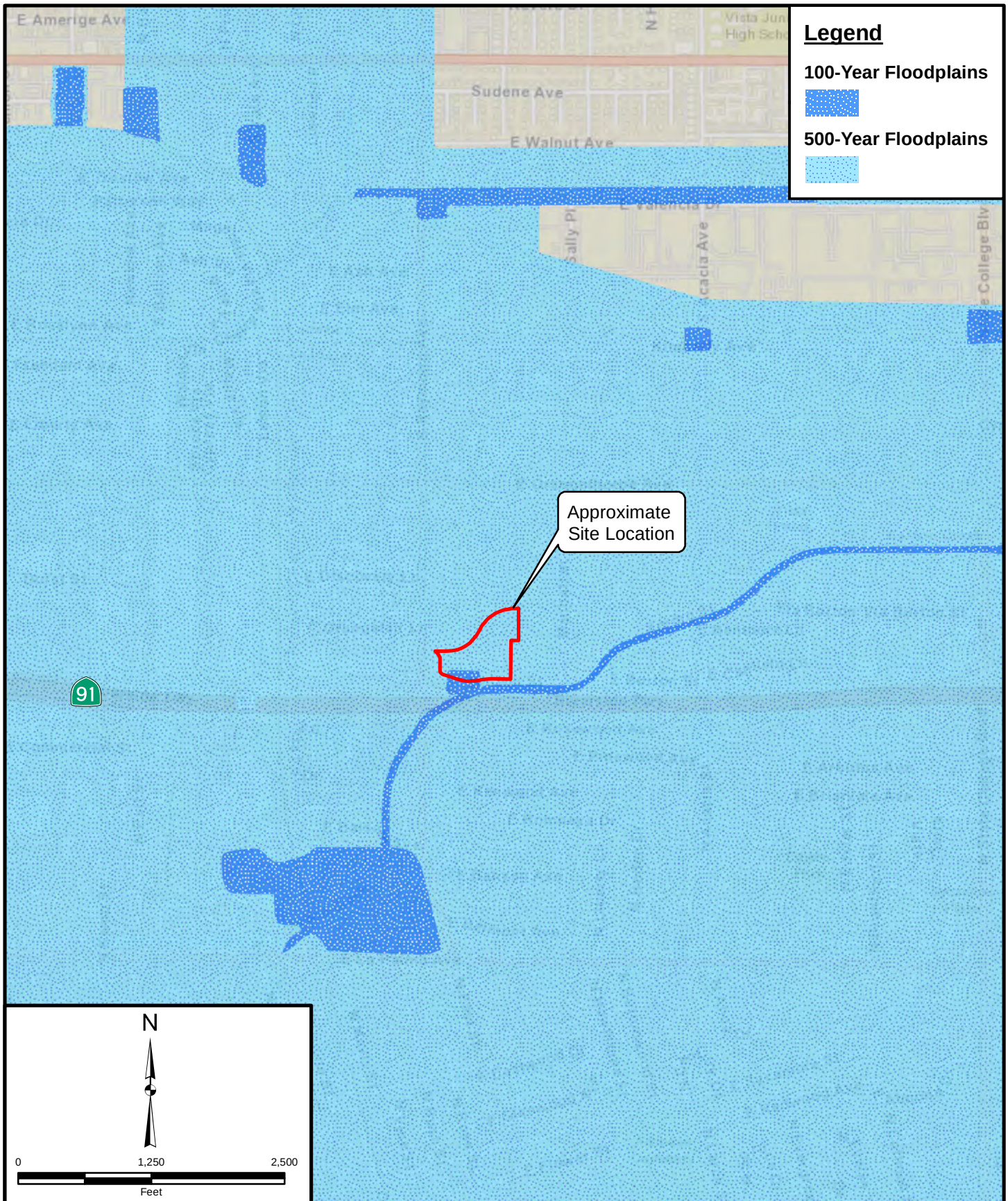
Project: 13477.001 Eng/Geol: CCK/JMP
 Scale: 1" = 5 miles Date: May 2022
 Basemap Reference: © 2022 Microsoft Corporation
 Earthstar Geographics SIO © 2022 TomTom
 Seismicity Data Reference: maps.conservation.ca.gov

**REGIONAL FAULT AND
HISTORIC SEISMICITY MAP**
 Proposed Industrial Building
 1500 South Raymond Avenue
 Fullerton, California

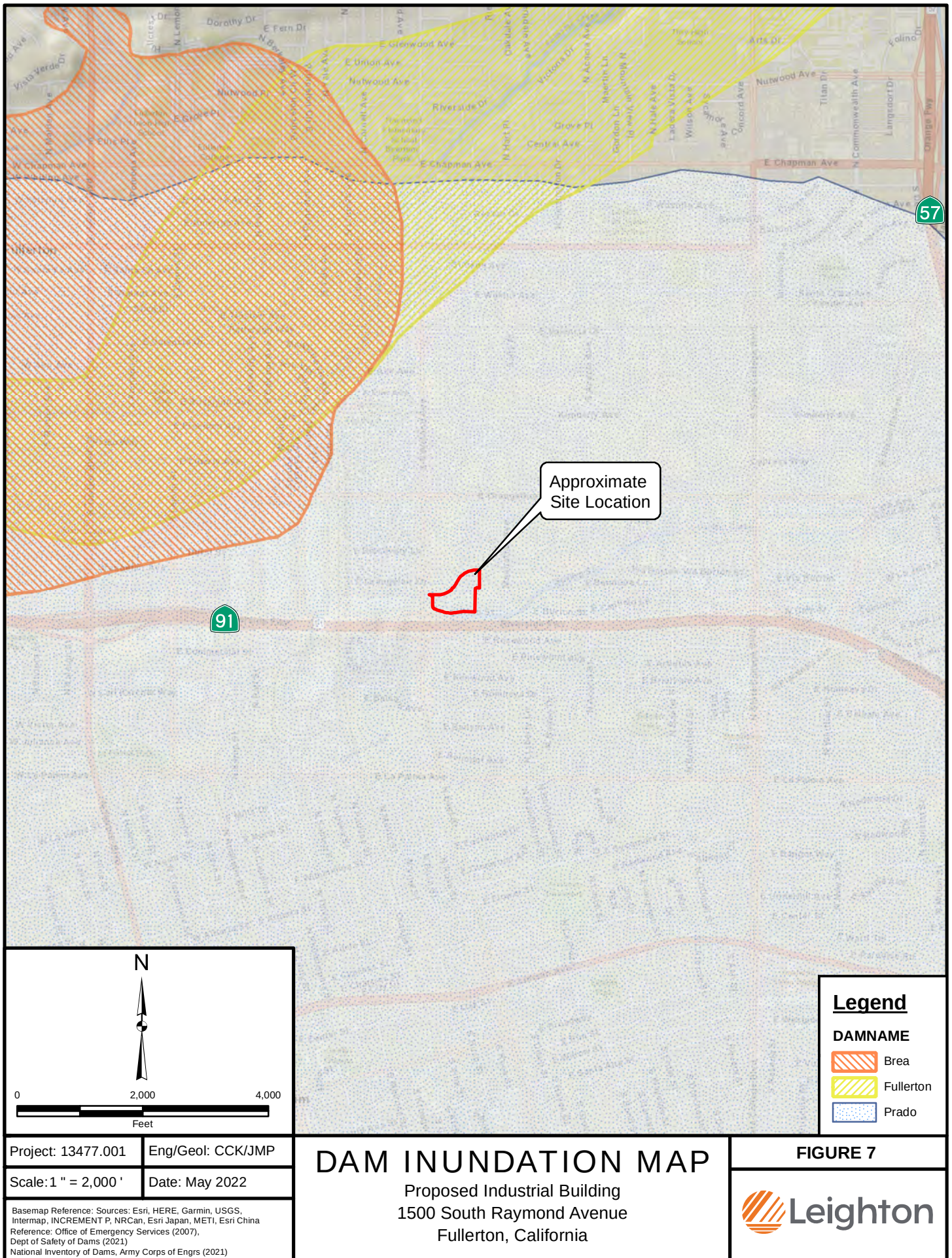
FIGURE 4

Leighton



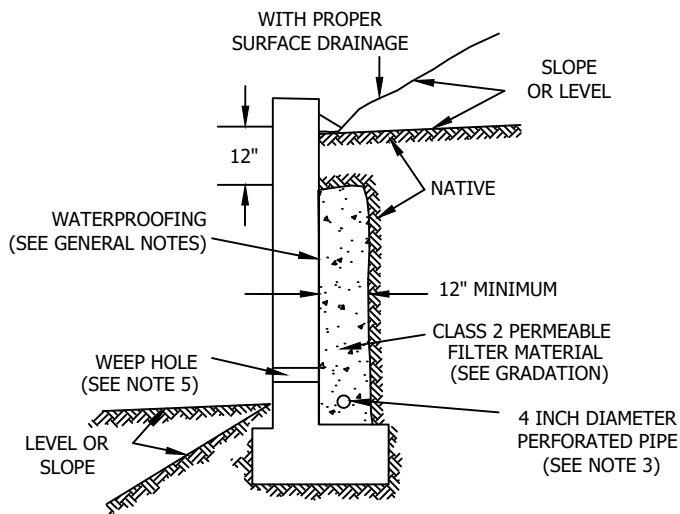


Project: 13477.001	Eng/Geol: CCK/JMP	FLOOD HAZARD ZONE MAP Proposed Industrial Building 1500 South Raymond Avenue Fullerton, California	FIGURE 6
Scale: 1" = 1,250'	Date: May 2022		Leighton
Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community FEMA (http://www.fema.gov/index.shtm), DWR (http://www.dwr.ca.gov)			

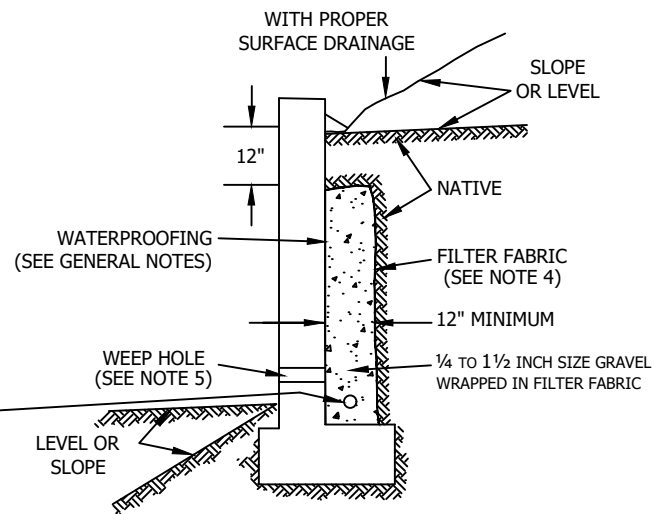


SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



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

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No. 13477.001
Project Rexford - 1500 S. Raymond Avenue, Fullerton
Drilling Co. Gregg Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
Logged By BTM
Hole Diameter 8"
Ground Elevation 173'
Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
				B-1				SM	@Surface: 3 inches AC over 3 inches Base	
								SM	Artificial fill, undocumented (Afu)	
									@0.5': Silty SAND, medium brown, moist, very fine to fine sand, trace coarse sand	
170								SM	Quaternary Age Young Alluvial Fan Deposits (Qyf)	
								SP	@2.5': Silty SAND, light brown, moist, very fine to fine sand, few medium sand	
5				S-1A S-1B	2 3 4		5 3		@3.5': SAND, tan, slightly moist, predominately fine sand, some medium sand, trace coarse sand, trace fine subangular gravel	
									@5': SAND, loose to medium dense, dark brown, moist, fine sand, few coarse sand to fine gravel, few clay; grades to SAND, tan, moist, fine to medium sand, few coarse sand, trace iron oxidation stains	
165				R-2	6 4 6	96	2		@7.5': SAND, loose, tan, slightly moist, predominately fine sand, few medium sand, trace coarse sand and fine gravel	
10				S-3	4 5 7		4		@10': SAND, medium dense, tan, slightly moist, fine sand, slightly micaceous; grades to yellowish tan with few medium sand	
160										
15				R-4	5 8 9	103	10		@15': SAND, medium dense, tan, slightly moist, predominately fine sand, few medium sand, small 1" pocket of Sandy CLAY	
									@15.5', brown, low plasticity	
155										
20				S-5	4 5 8		3		@20': SAND, medium dense, tan, slightly moist, predominately fine sand, few medium sand	
150										
25				R-6	4 8 10	105	13	CL SP	@25': Sandy CLAY, stiff, light to medium brown, moist, low plasticity, fine sand, trace fine gravel, slightly micaceous	
									@26': SAND, medium dense, tan, slightly moist, predominately fine sand, few medium sand, slightly micaceous	
145										
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 										

SAMPLE TYPES:

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

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-1

Project No. 13477.001
Project Rexford - 1500 S. Raymond Avenue, Fullerton
Drilling Co. Gregg Drilling
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Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
Logged By BTM
Hole Diameter 8"
Ground Elevation 173'
Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
30				S-7	5 6 11		6		@30': SAND, medium dense, light brown, moist, fine sand, slightly micaceous, pockets of Sandy CLAY to Clayey SAND	
140									Total Depth: 31.5' bgs No groundwater encountered during drilling. Backfilled with soil cuttings and patched with cold-mix asphalt.	
35										
135										
40										
130										
45										
125										
50										
120										
55										
115										
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
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 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-2

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 175'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
<i>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.</i>										
175	0	N S							@Surface: 6 inches AC over 3 inches Base	
				B-1				SC	Artificial fill, undocumented (Afu)	
								SC/SM	@0.75': Clayey SAND, dark brown, moist, fine sand, low plasticity, trace medium to coarse sand, asphalt chunks	
								SC	Quaternary Age Young Alluvial Fan Deposits (Qyf)	
								SP	@2': Interbedded Silty SAND, light brown, slightly moist, very fine to fine sand, slightly micaceous; and Clayey SAND, medium to dark brown, moist, fine sand, low plasticity	
170	5			R-1	3 4 6	93	3		@3.5': Clayey SAND, medium grayish brown, moist, fine sand, low plasticity, slightly micaceous	CN, DS
				S-2	2 2 3		2		@4.5': SAND, tan, predominately fine sand, few medium sand, slightly micaceous	
									@5': SAND, loose, brown, moist, predominately fine sand, trace medium sand, slightly micaceous, chunks of asphalt (slough); grades to SAND, tan, fine to medium sand, slightly micaceous	
									@7.5': SAND, loose, tan, slightly moist, predominately fine sand, few medium sand, trace coarse sand, micaceous	
165	10			R-3	6 5 7	125	2		@10': SAND, medium dense, yellowish brown, slightly moist, fine to medium sand, trace coarse sand, slightly micaceous, top of sample disturbed	
160	15			S-4	4 6 8		2		@15': SAND, medium dense, tan to light brown, slightly moist, fine to medium sand, trace coarse sand, micaceous	
155	20			R-5	4 12 18	98	3		@20': Same as above; medium dense, grades to predominately fine sand @ ~21.7'	
150	25			S-6	5 8 10		4	SM	@25.2': Silty SAND, medium dense, light brown, moist, very fine sand, slightly micaceous	
145	30									
SAMPLE TYPES:										
TYPE OF TESTS:										
DS DIRECT SHEAR										
SE SAND EQUIVALENT										
SG SPECIFIC GRAVITY										
UC UNCONFINED COMPRESSIVE STRENGTH										
AL ATTERBERG LIMITS										
CN CONSOLIDATION										
CO COLLAPSE										
CR CORROSION										
CU UNDRAINED TRIAXIAL										
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MD MAXIMUM DENSITY										
PP POCKET PENETROMETER										
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G GRAB SAMPLE										
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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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SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-2

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 175'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
145	30			R-7	5 10 15	112	7	SP SC	@30': SAND, medium dense, light brown, moist, predominately fine sand, few medium sand, trace coarse sand, slightly micaceous @31': Clayey SAND, medium dense, medium to dark brown, very fine sand, low plasticity, slightly micaceous Total Depth: 31.5' bgs No groundwater encountered during drilling. Backfilled with soil cuttings and patched with cold-mix asphalt.	
140	35									
135	40									
130	45									
125	50									
120	55									
115	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-3

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 174'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
									@Surface: 7 inches AC over 3 inches Base	
				B-1		106	11	SC	Artificial fill, undocumented (Afu) @0.83': Clayey SAND, light to dark brown, fine to medium sand, low plasticity, trace coarse sand	CN, CR, DS, EI, MD, RV
								SM	Quaternary Age Young Alluvial Fan Deposits (Qyf) @2': Silty SAND, brown, moist, very fine sand, slightly micaceous, interbedded with thin layers of Sandy CLAY and Clayey SAND @4': Silty SAND, tan, slightly moist, very fine to fine sand, slightly micaceous	
170	5			S-1	3 3 4		5	SC SP	@5': Clayey SAND, loose to medium dense, dark brown, moist, predominately fine sand, few medium sand, low plasticity, slightly micaceous @6': SAND, loose to medium dense, tan, slightly moist, very fine sand, micaceous	
				R-2	6 6 9	99	1		@7.5': SAND with silt, medium dense, slightly moist, fine to medium sand, few coarse sand, slightly micaceous; grades to predominately fine sand	CN, DS
165	10			S-3	2 2 4		2		@10': SAND, loose, tan, slightly moist to dry, fine to medium sand, few coarse sand, trace fine subround to subangular gravel	
160	15			R-4	8 12 15	107	2		@15': Same as above; medium dense, slightly micaceous	
155	20			S-5	5 9 9		2	SM	@20': SAND, medium dense, light brown, slightly moist, fine to medium sand, few coarse sand, trace clay inclusions, slightly micaceous @21.4': Silty SAND, tan, fine to medium sand, slightly micaceous	
150	25			R-6	8 12 18	114	7	SP	@25': SAND, medium dense, tan, slightly moist, fine sand, slightly micaceous; grades to SAND, light brown, fine sand, trace medium to coarse sand, some clayey pockets, slightly micaceous	
145	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



GEOTECHNICAL BORING LOG LB-3

Project No. 13477.001
Project Rexford - 1500 S. Raymond Avenue, Fullerton
Drilling Co. Gregg Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
Logged By BTM
Hole Diameter 8"
Ground Elevation 174'
Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>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.</i>	Type of Tests
30		N S		S-7	4 7 10		10	CL SP	@30': SAND, medium dense, slightly moist, very fine to fine sand, few medium sand, trace coarse sand, slightly micaceous @30.5': ~4 -inches of Sandy CLAY, brown, low plasticity, fine sand	
140				R-8	8 17 24	104	2		@35': SAND, dense, tan, slightly moist, very fine to fine sand, trace medium sand, slightly micaceous	
135				S-9	6 9 11		4		@40': Same as above; medium dense	
130				R-10	8 14 18	100	2		@45': Same as above; medium dense	
125				S-11	7 11 13		4		@50': Same as above; dense	
120									Total Depth: 51.5' bgs No groundwater encountered during drilling. Backfilled with soil cuttings and patched with cold-mix asphalt.	
115										
60										

SAMPLE TYPES:

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

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-4

Project No. 13477.001
Project Rexford - 1500 S. Raymond Avenue, Fullerton
Drilling Co. Gregg Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
Logged By BTM
Hole Diameter 8"
Ground Elevation 171'
Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
170				B-1				SP	@Surface: 6 inches AC over 3 inches Base	
									Artificial fill, undocumented (Afu)	
									@0.75': SAND with silt and clay, brown, moist, very fine to fine sand	
								SP	Quaternary Age Young Alluvial Fan Deposits (Qyf)	
									@2.5': SAND, light brown, moist, very fine to fine sand, some medium to coarse sand	
	5			R-1	2 3 6	95	6		@4': SAND with silt, yellowish brown, moist, very fine to fine sand, trace medium sand, trace rootlets, slightly micaceous	
165								SC	@5': SAND with silt, loose, yellowish brown, moist, very fine to fine sand, some fine subangular gravel	
				S-2	2 3 4		2	SP	@6': Clayey SAND, loose, medium brown, moist, fine sand, low plasticity, trace medium sand	
									@7.5': SAND, loose to medium dense, pale yellow brown, moist, very fine to medium sand	
	10			R-3	5 8 13	101	2		@10': Same as above; medium dense, trace angular gravel	
160										
	15			S-4	4 6 8		6		@15': SAND, medium dense, yellowish brown, moist, fine to coarse sand, some fine subangular gravel	
155										
	20			R-5	15 24 31	103	2		@20': Same as above; dense, fine to medium sand, trace coarse sand and fine gravel	
150										
	25			S-6	7 10 13		5		@25': SAND, dense, yellowish brown, moist, fine sand, slightly micaceous	
145										
	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										
										

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 LB-4

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 171'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>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.</i>	
140	30			R-7	9 27 46	97	3		@30': SAND, very dense, yellowish brown, slightly moist, fine to medium sand, slightly micaceous	
									Total Depth: 31.5' bgs No groundwater encountered during drilling. Backfilled with soil cuttings and patched with cold-mix asphalt.	
135	35									
130	40									
125	45									
120	50									
115	55									
60	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

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 H HYDROMETER
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 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LP-1

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 177'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
									@Surface: 6 inches AC over 3 inches Base	
175				B-1				SC-CL	Artificial fill, undocumented (Afu) @0.75': Sandy CLAY to Clayey SAND, dark brown, moist, very fine sand, low plasticity	
								SM	Quaternary Age Young Alluvial Deposits (Qyf) @2': Silty SAND, light to medium brown, moist, very fine sand, slightly micaceous	
5				S-1A	2		7	SP	@5': SAND with clay, very loose, medium brown, moist, very fine to fine sand, slightly micaceous, some coarse sand and fine gravel, asphalt chunks (likely slough)	
170				S-1B	1		19		@6.2': SAND with clay, medium brown, moist, very fine to fine sand, slightly micaceous	
				R-2	4 5 15	96	6		@8.5': SAND, medium dense, tan to light brown, moist, fine sand, slightly micaceous, pockets of low to medium plasticity clay and sandy clay	
10										
165										
15										
160										
20										
155										
25										
150										
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										
Total Depth: 10' bgs No groundwater encountered during drilling. Temporary percolation well installed. Blank 2-inch PVC installed from 0 to 5 feet and 0.020-inch slotted 2-inch PVC installed from 5 to 10 feet. Annulus filled with No. 3 Monterey SAND from 4 to 10 feet. Percolation test performed on 3-21-2021. Upon completion of percolation test, well casing removed and boring backfilled with soil cuttings and patched with cold-mix asphalt. 										



GEOTECHNICAL BORING LOG LP-2

Project No. 13477.001
 Project Rexford - 1500 S. Raymond Avenue, Fullerton
 Drilling Co. Gregg Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2 - Exploration Location Map

Date Drilled 3-18-22
 Logged By BTM
 Hole Diameter 8"
 Ground Elevation 174'
 Sampled By BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
				B-1				SM	@Surface: 6 inches AC over 4 inches Base	
								SP	Quaternary Age Young Alluvial Fan Deposits (Qyf) @0.83': Silty SAND, medium brown, slightly moist, very fine sand, slightly micaceous @2.5': SAND, tan, slightly moist, predominately very fine to fine sand, few medium sand, slightly micaceous @3.5': Same as above; little medium sand	
170	5			R-1	3 4 7	116	3	SM	@5': Silty SAND, loose, tan, slightly moist to dry, predominately fine sand, few medium sand, slightly micaceous	
165	10			S-2	2 2 3				@8.5': NO RECOVERY; loose	
160	15								Total Depth: 10' bgs No groundwater encountered during drilling. Temporary percolation well installed. Blank 2-inch PVC installed from 0 to 5 feet and 0.020-inch slotted 2-inch PVC installed from 5 to 10 feet. Annulus filled with No. 3 Monterey SAND from 4 to 10 feet. Percolation test performed on 3-21-2021. Upon completion of percolation test, well casing removed and boring backfilled with soil cuttings and patched with cold-mix asphalt.	
155	20									
150	25									
145	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



APPENDIX B

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	13477.001	Test Hole Number:	LP-1
Project Name:	Rexford Raymond Fullerton	Date Excavated:	3/18/2022
Earth Description:	Alluvium	Date Tested:	3/21/2022
Liquid Description:	Tap water	Depth of boring (ft):	10
Tested By:	BTM	Radius of boring, r (in):	4
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	6
		Porosity of Annulus Material, n :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	7:41	-	-	-	0.0
2	7:56	15	5.58	53.0	32.8
3	8:11	15	5.59	52.9	65.7
4	8:26	15	5.59	52.9	98.5
5	8:41	15	5.66	52.1	131.4
6	8:56	15	6.15	46.2	164.2
7	9:11	15	6.26	44.9	197.1
8	9:26	15	6.40	43.2	229.9
9	9:41	15	6.53	41.6	262.8
10	9:56	15	6.52	41.8	295.6

Total Volume of Water Delivered (gallons)	295.6
Total Volume of Water Delivered (cubic inches)	68288.2893
Average Water Height (inches)	47.6
Average Percolation Surface Area (cubic Inches)	1247.3
Duration of Test (minutes)	135
Duration of Test (hours)	2.25

Measured Infiltration Rate = (Total Volume)/(Test Duration)/(Surface Area)

Measured Infiltration Rate (inches per hour) = 24.3

Boring Percolation Test Data Sheet

Project Number:	13477.001	Test Hole Number:	LP-2
Project Name:	Rexford Raymond Fullerton	Date Excavated:	3/18/2022
Earth Description:	Alluvium	Date Tested:	3/21/2022
Liquid Description:	Tap water	Depth of boring (ft):	10
Tested By:	BTM	Radius of boring, r (in):	4
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	6
		Porosity of Annulus Material, n :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	12:15	-	-	-	0.0
2	12:30	15	7.46	30.5	98.9
3	12:45	15	7.41	31.1	197.8
4	1:00	15	7.42	31.0	296.8
5	1:15	15	7.43	30.8	395.7
6	1:30	15	7.44	30.7	494.6
7	1:45	15	7.40	31.2	593.5
8	2:00	15	7.43	30.8	692.4
9	2:15	15	7.41	31.1	791.4
10	2:30	15	7.43	30.8	890.3

Total Volume of Water Delivered (gallons)	890.3
Total Volume of Water Delivered (cubic inches)	205654.68
Average Water Height (inches)	30.9
Average Percolation Surface Area (cubic Inches)	826.7
Duration of Test (minutes)	135
Duration of Test (hours)	2.25

Measured Infiltration Rate = (Total Volume)/(Test Duration)/(Surface Area)

Measured Infiltration Rate (inches per hour) = 110.6

APPENDIX C

LABORATORY TEST RESULTS



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Rexford Raymond, Fullerton Tested By: J. Gonzalez Date: 03/22/22
Project No.: 13477.001 Checked By: A. Santos Date: 03/23/22
Boring No.: LB-3 Depth (ft.): 0-5
Sample No.: B-1
Soil Identification: Dark brown silty clayey sand (SC-SM)

Preparation Method:

☒ Moist
☐ Dry

☒ 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)	3760	3865	3871	3785		
Weight of Mold (g)	1826	1826	1826	1826		
Net Weight of Soil (g)	1934	2039	2045	1959		
Wet Weight of Soil + Cont. (g)	523.8	494.5	457.3	456.9		
Dry Weight of Soil + Cont. (g)	485.9	448.9	405.2	397.0		
Weight of Container (g)	39.0	36.4	39.3	38.8		
Moisture Content (%)	8.48	11.05	14.24	16.72		
Wet Density (pcf)	128.0	135.0	135.4	129.7		
Dry Density (pcf)	118.0	121.6	118.5	111.1		

Maximum Dry Density (pcf)

121.6

Optimum Moisture Content (%)

11.5

PROCEDURE USED

☒ 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

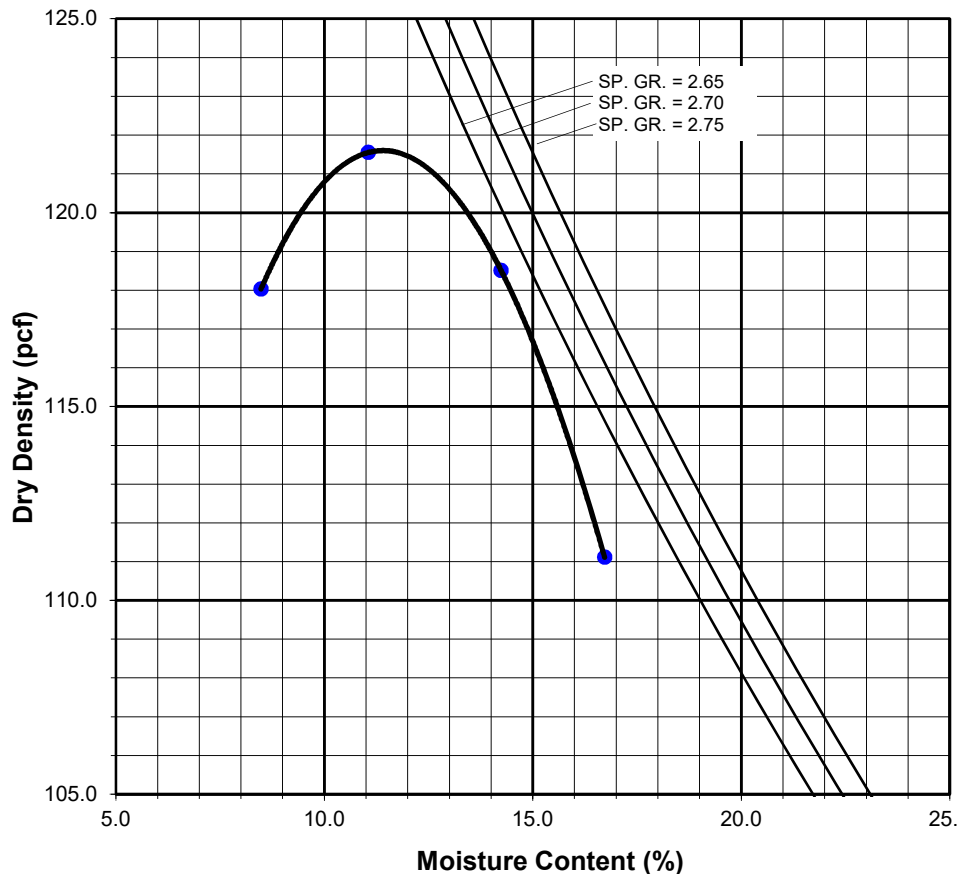
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL, PL, PI





EXPANSION INDEX of SOILS
ASTM D 4829

Project Name: Rexford Raymond, Fullerton Tested By: A. Santos Date: 03/29/22
Project No.: 13477.001 Checked By: G. Bathala Date: 04/12/22
Boring No.: LB-3 Depth (ft.): 0-5
Sample No.: B-1
Soil Identification: Dark brown silty clayey sand (SC-SM)

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	1.0075
Wt. Comp. Soil + Mold (g)	619.50	442.70
Wt. of Mold (g)	202.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	842.70	644.80
Dry Wt. of Soil + Cont. (g)	780.30	588.58
Wt. of Container (g)	0.00	202.10
Moisture Content (%)	8.00	14.55
Wet Density (pcf)	125.9	132.5
Dry Density (pcf)	116.6	115.7
Void Ratio	0.446	0.457
Total Porosity	0.308	0.314
Pore Volume (cc)	63.8	65.4
Degree of Saturation (%) [S _{meas}]	48.4	86.0

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
03/29/22	9:55	1.0	0	0.6410
03/29/22	10:05	1.0	10	0.6400
Add Distilled Water to the Specimen				
03/29/22	10:10	1.0	5	0.6420
03/30/22	6:43	1.0	1238	0.6485
03/30/22	8:35	1.0	1350	0.6485

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	8
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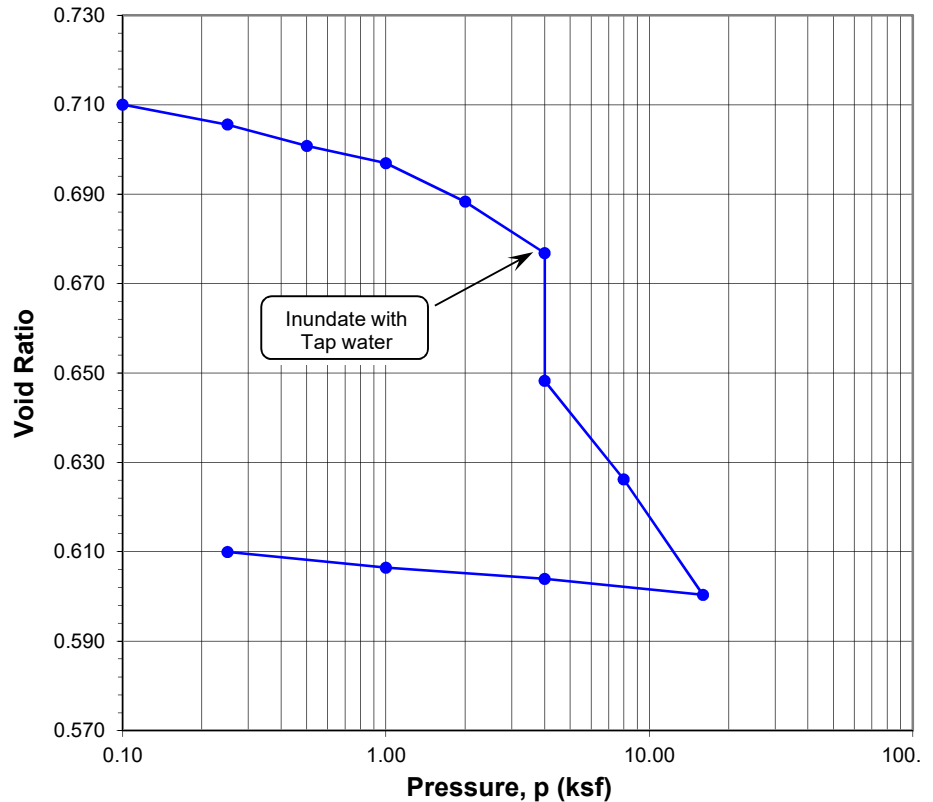


**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project Name: Rexford Raymond, Fullerton
Project No.: 13477.001
Boring No.: LB-2
Sample No.: R-1
Soil Identification: Light brown poorly-graded sand with silt (SP-SM)

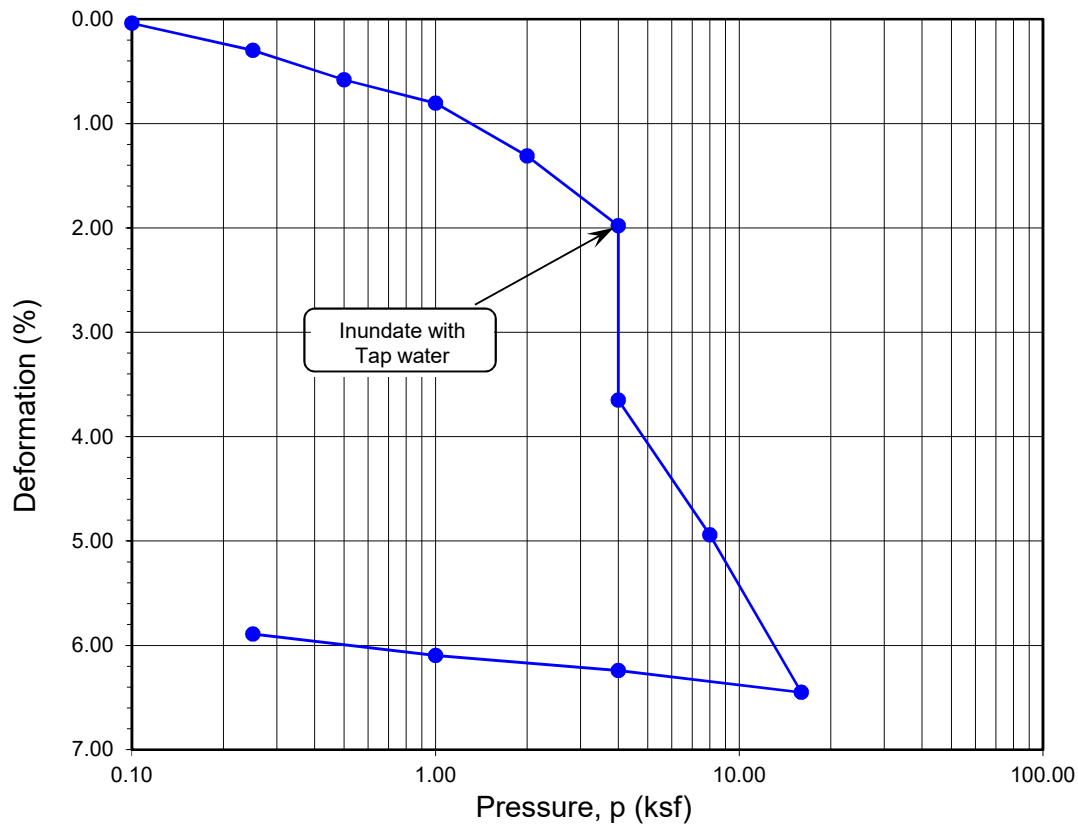
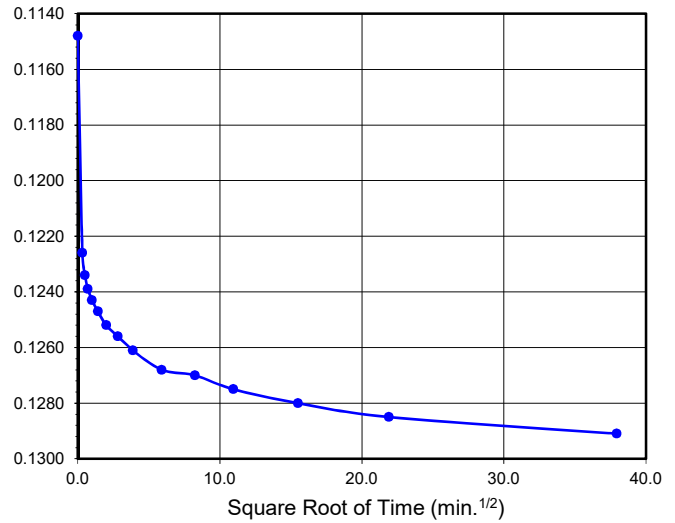
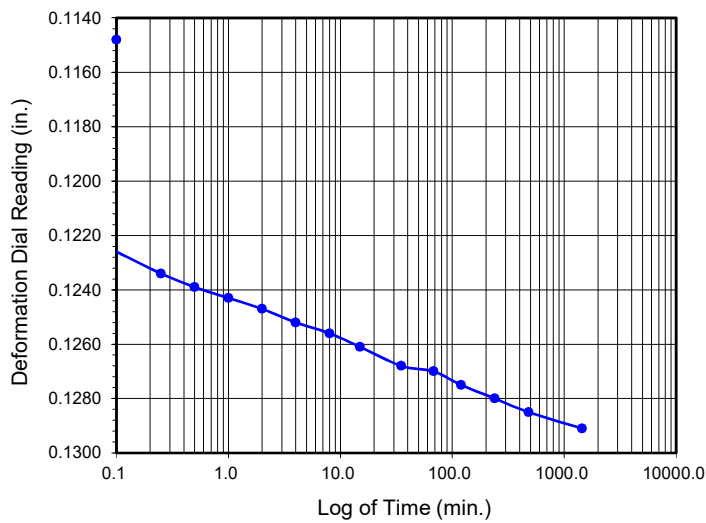
Tested By: G. Bathala Date: 03/23/22
Checked By: A. Santos Date: 04/12/22
Depth (ft.): 5.0
Sample Type: Ring

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	163.34
Weight of Ring (g):	41.41
Height after consol. (in.):	0.9411
Before Test	
Wt. of Wet Sample+Cont. (g):	181.72
Wt. of Dry Sample+Cont. (g):	178.65
Weight of Container (g):	57.47
Initial Moisture Content (%)	2.5
Initial Dry Density (pcf)	98.9
Initial Saturation (%):	10
Initial Vertical Reading (in.)	0.0726
After Test	
Wt. of Wet Sample+Cont. (g):	246.91
Wt. of Dry Sample+Cont. (g):	227.24
Weight of Container (g):	68.90
Final Moisture Content (%)	16.82
Final Dry Density (pcf):	103.3
Final Saturation (%):	72
Final Vertical Reading (in.)	0.1361
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)	Time Readings @ 8.0 ksf				
							Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
0.10	0.0730	0.9996	0.00	0.04	0.710	0.04	3/28/22	8:42:00	0.0	0.0	0.1148
0.25	0.0763	0.9963	0.07	0.37	0.706	0.30	3/28/22	8:42:06	0.1	0.3	0.1226
0.50	0.0800	0.9926	0.16	0.74	0.701	0.58	3/28/22	8:42:15	0.2	0.5	0.1234
1.00	0.0837	0.9890	0.30	1.11	0.697	0.81	3/28/22	8:42:30	0.5	0.7	0.1239
2.00	0.0901	0.9825	0.44	1.75	0.688	1.31	3/28/22	8:43:00	1.0	1.0	0.1243
4.00	0.0981	0.9745	0.57	2.55	0.677	1.98	3/28/22	8:44:00	2.0	1.4	0.1247
4.00	0.1148	0.9578	0.57	4.22	0.648	3.65	3/28/22	8:46:00	4.0	2.0	0.1252
8.00	0.1291	0.9435	0.71	5.65	0.626	4.94	3/28/22	8:50:00	8.0	2.8	0.1256
16.00	0.1458	0.9268	0.87	7.32	0.600	6.45	3/28/22	8:57:00	15.0	3.9	0.1261
4.00	0.1421	0.9305	0.71	6.95	0.604	6.24	3/28/22	9:17:00	35.0	5.9	0.1268
1.00	0.1393	0.9334	0.57	6.67	0.606	6.10	3/28/22	9:50:00	68.0	8.2	0.1270
0.25	0.1361	0.9365	0.46	6.35	0.610	5.89	3/28/22	10:42:00	120.0	11.0	0.1275
							3/28/22	12:42:00	240.0	15.5	0.1280
							3/28/22	16:42:00	480.0	21.9	0.1285
							3/29/22	8:42:00	1440.0	37.9	0.1291

Time Readings @ 8.0 ksf



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	R-1	5	2.5	16.8	98.9	103.3	0.711	0.610	10	72

Soil Identification: Light brown poorly-graded sand with silt (SP-SM)



ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project No.: 13477.001

Rexford Raymond, Fullerton

04-22



**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project Name: Rexford Raymond, Fullerton

Project No.: 13477.001

Boring No.: LB-3

Sample No.: B-1

Soil Identification: Dark brown silty clayey sand (SC-SM)

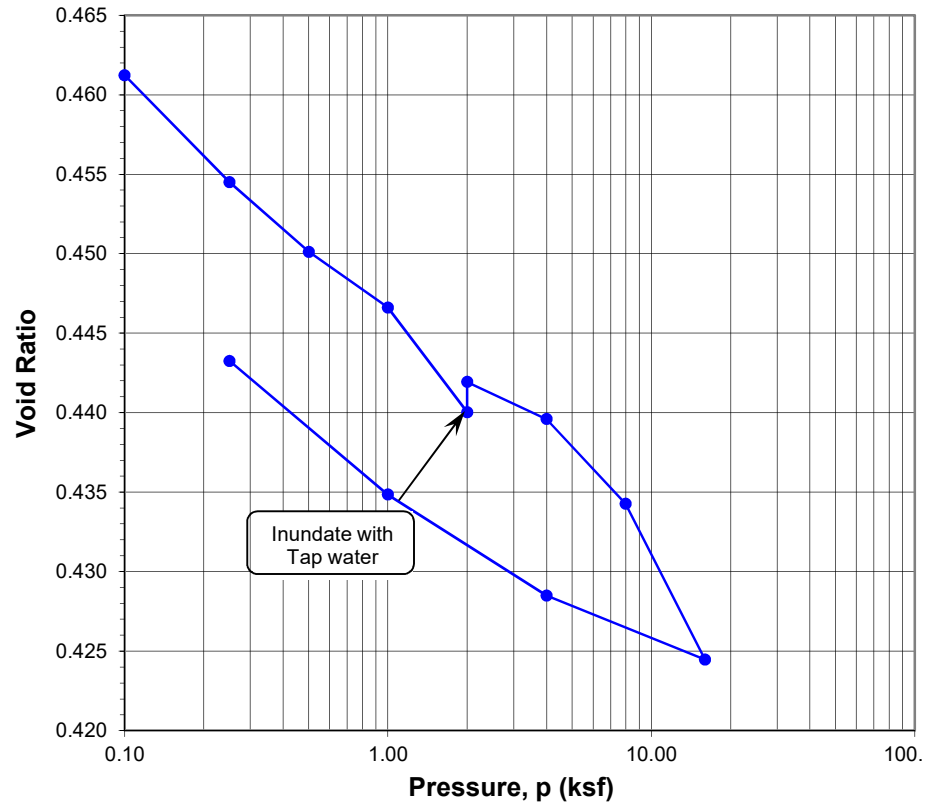
Tested By: G. Bathala Date: 03/23/22

Checked By: A. Santos Date: 04/12/22

Depth (ft.): 0-5

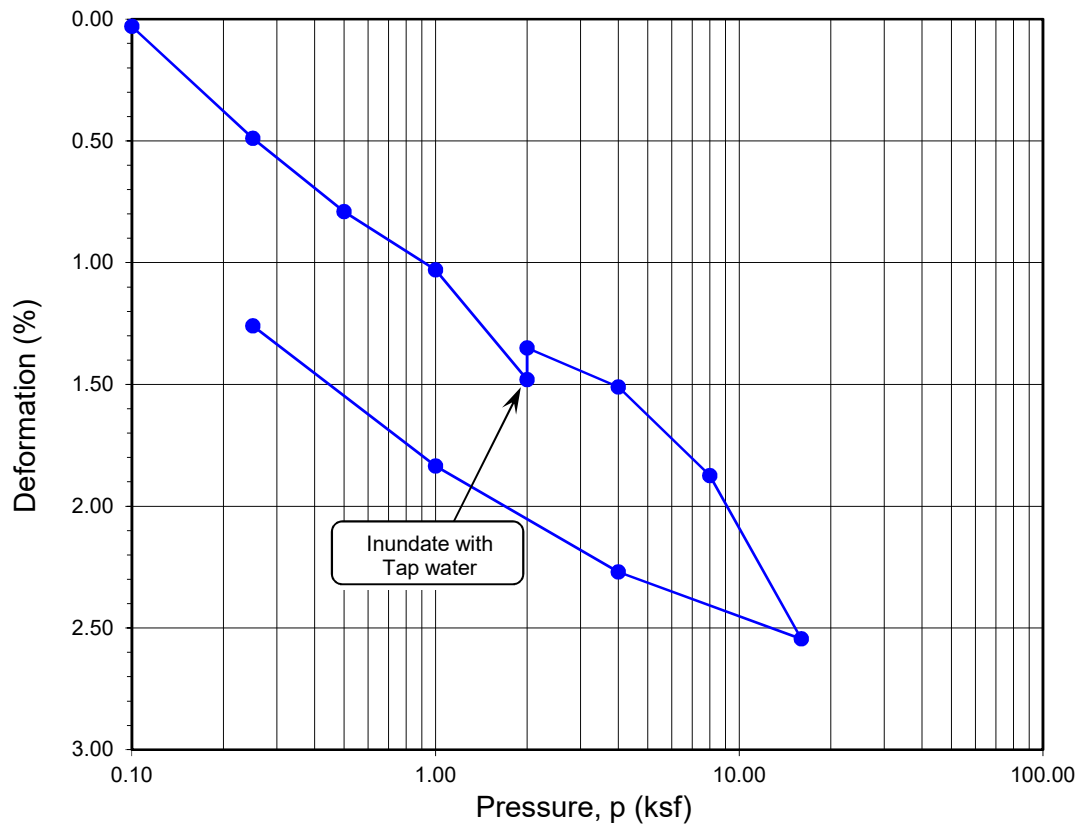
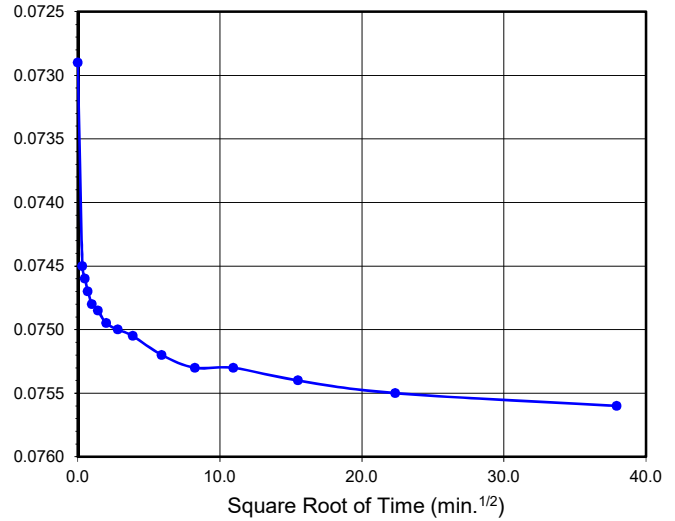
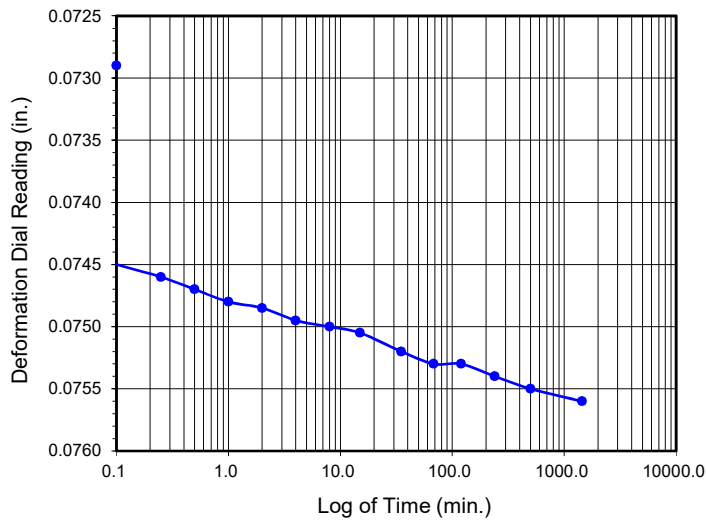
Sample Type: 95% Remold

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	200.54
Weight of Ring (g):	45.45
Height after consol. (in.):	0.9874
Before Test	
Wt. of Wet Sample+Cont. (g):	180.48
Wt. of Dry Sample+Cont. (g):	169.13
Weight of Container (g):	69.86
Initial Moisture Content (%)	11.4
Initial Dry Density (pcf)	115.7
Initial Saturation (%):	67
Initial Vertical Reading (in.)	0.0570
After Test	
Wt. of Wet Sample+Cont. (g):	261.82
Wt. of Dry Sample+Cont. (g):	241.51
Weight of Container (g):	56.66
Final Moisture Content (%)	14.57
Final Dry Density (pcf):	117.4
Final Saturation (%):	90
Final Vertical Reading (in.)	0.0720
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)	Time Readings @ 4.0 ksf				
							Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
0.10	0.0573	0.9997	0.00	0.03	0.461	0.03	3/28/22	8:10:00	0.0	0.0	0.0729
0.25	0.0623	0.9947	0.04	0.53	0.455	0.49	3/28/22	8:10:06	0.1	0.3	0.0745
0.50	0.0658	0.9912	0.09	0.88	0.450	0.79	3/28/22	8:10:15	0.2	0.5	0.0746
1.00	0.0689	0.9881	0.16	1.19	0.447	1.03	3/28/22	8:10:30	0.5	0.7	0.0747
2.00	0.0742	0.9828	0.24	1.72	0.440	1.48	3/28/22	8:11:00	1.0	1.0	0.0748
2.00	0.0729	0.9841	0.24	1.59	0.442	1.35	3/28/22	8:12:00	2.0	1.4	0.0749
4.00	0.0756	0.9814	0.35	1.86	0.440	1.51	3/28/22	8:14:00	4.0	2.0	0.0750
8.00	0.0806	0.9765	0.48	2.36	0.434	1.88	3/28/22	8:18:00	8.0	2.8	0.0750
16.00	0.0888	0.9683	0.63	3.17	0.424	2.54	3/28/22	8:25:00	15.0	3.9	0.0751
4.00	0.0843	0.9727	0.46	2.73	0.428	2.27	3/28/22	8:45:00	35.0	5.9	0.0752
1.00	0.0787	0.9784	0.33	2.16	0.435	1.83	3/28/22	9:18:00	68.0	8.2	0.0753
0.25	0.0720	0.9850	0.24	1.50	0.443	1.26	3/28/22	10:10:00	120.0	11.0	0.0753
							3/28/22	12:10:00	240.0	15.5	0.0754
							3/28/22	16:30:00	500.0	22.4	0.0755
							3/29/22	8:10:00	1440.0	37.9	0.0756

Time Readings @ 4.0 ksf



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	B-1	0-5	11.4	14.6	115.7	117.4	0.462	0.443	67	90

Soil Identification: Dark brown silty clayey sand (SC-SM)



ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435

Project No.: 13477.001

Rexford Raymond, Fullerton

04-22

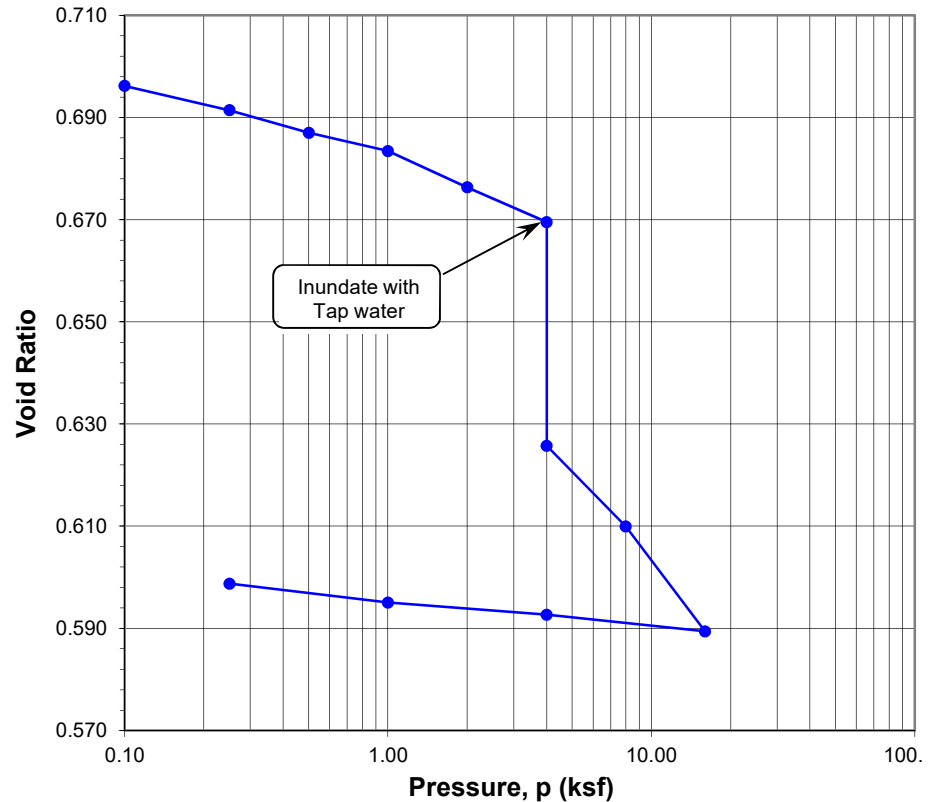


**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project Name: Rexford Raymond, Fullerton
Project No.: 13477.001
Boring No.: LB-3
Sample No.: R-2
Soil Identification: Light olive brown poorly-graded sand with silt (SP-SM)

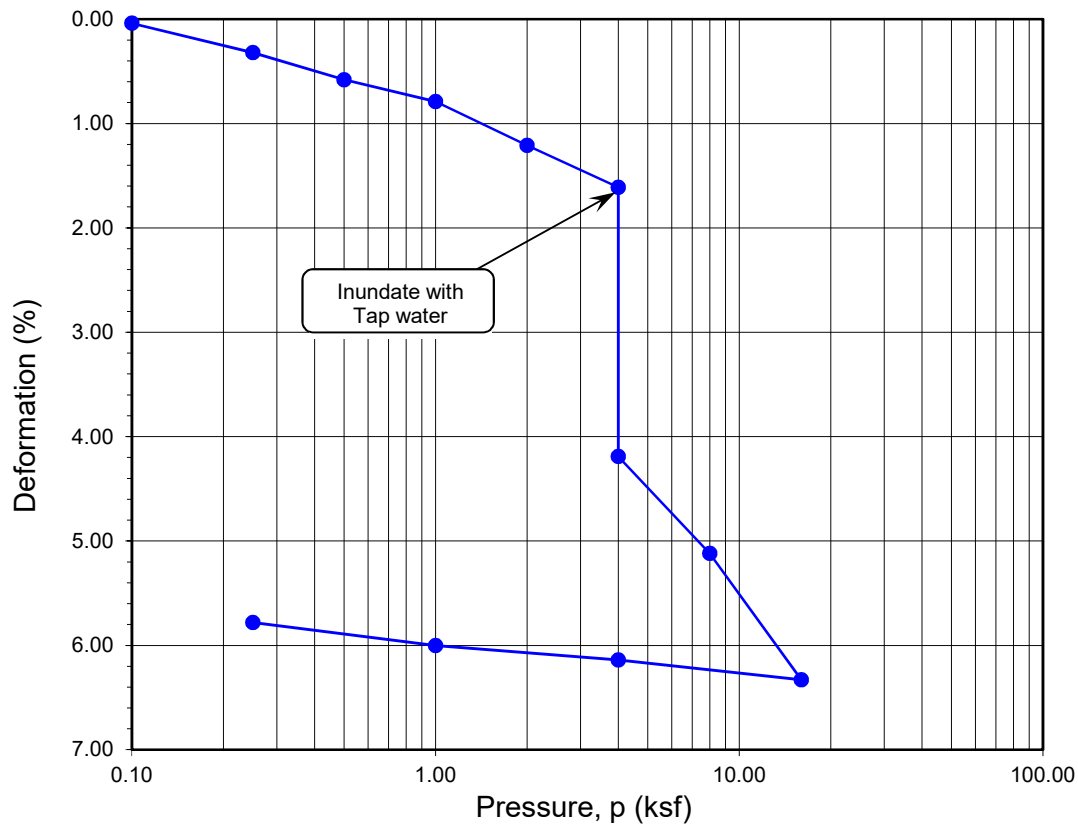
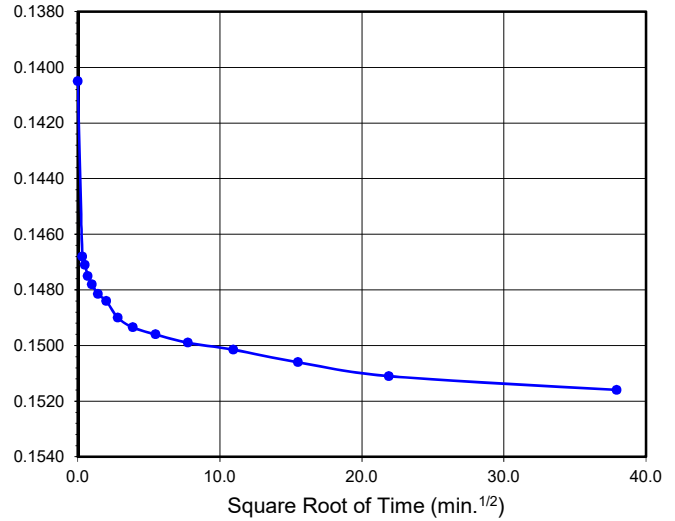
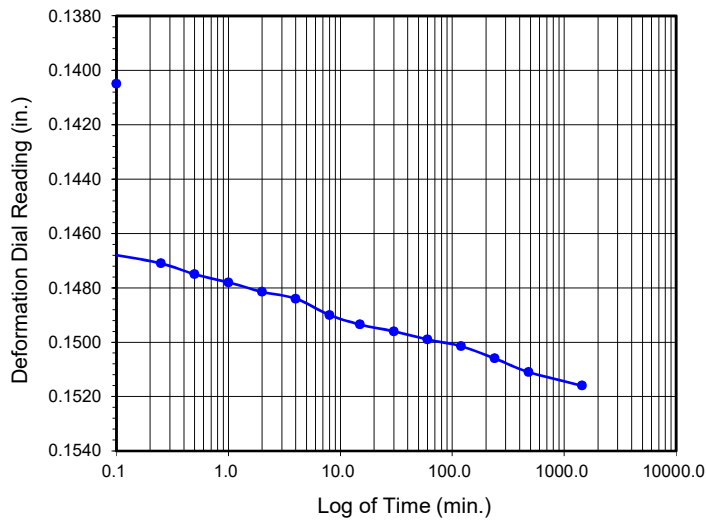
Tested By: G. Bathala Date: 03/23/22
Checked By: A. Santos Date: 04/13/22
Depth (ft.): 7.5
Sample Type: Ring

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	164.97
Weight of Ring (g):	44.44
Height after consol. (in.):	0.9422
Before Test	
Wt. of Wet Sample+Cont. (g):	206.48
Wt. of Dry Sample+Cont. (g):	205.70
Weight of Container (g):	60.22
Initial Moisture Content (%)	0.5
Initial Dry Density (pcf)	99.7
Initial Saturation (%):	2
Initial Vertical Reading (in.)	0.0941
After Test	
Wt. of Wet Sample+Cont. (g):	241.39
Wt. of Dry Sample+Cont. (g):	222.21
Weight of Container (g):	60.23
Final Moisture Content (%)	16.32
Final Dry Density (pcf):	103.8
Final Saturation (%):	70
Final Vertical Reading (in.)	0.1548
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43



							Time Readings @ 8.0 ksf				
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)	Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
0.10	0.0945	0.9996	0.00	0.04	0.696	0.04	3/28/22	8:32:00	0.0	0.0	0.1405
0.25	0.0979	0.9962	0.06	0.38	0.691	0.32	3/28/22	8:32:06	0.1	0.3	0.1468
0.50	0.1013	0.9928	0.14	0.72	0.687	0.58	3/28/22	8:32:15	0.2	0.5	0.1471
1.00	0.1044	0.9897	0.24	1.03	0.683	0.79	3/28/22	8:32:30	0.5	0.7	0.1475
2.00	0.1096	0.9845	0.34	1.55	0.676	1.21	3/28/22	8:33:00	1.0	1.0	0.1478
4.00	0.1147	0.9794	0.45	2.06	0.670	1.61	3/28/22	8:34:00	2.0	1.4	0.1482
4.00	0.1405	0.9536	0.45	4.64	0.626	4.19	3/28/22	8:36:00	4.0	2.0	0.1484
8.00	0.1511	0.9430	0.58	5.70	0.610	5.12	3/28/22	8:40:00	8.0	2.8	0.1490
16.00	0.1647	0.9294	0.73	7.06	0.589	6.33	3/28/22	8:47:00	15.0	3.9	0.1494
4.00	0.1611	0.9330	0.56	6.70	0.593	6.14	3/28/22	9:02:00	30.0	5.5	0.1496
1.00	0.1581	0.9360	0.40	6.40	0.595	6.00	3/28/22	9:32:00	60.0	7.7	0.1499
0.25	0.1548	0.9393	0.29	6.07	0.599	5.78	3/28/22	10:32:00	120.0	11.0	0.1502
							3/28/22	12:32:00	240.0	15.5	0.1506
							3/28/22	16:32:00	480.0	21.9	0.1511
							3/29/22	8:32:00	1440.0	37.9	0.1516

Time Readings @ 8.0 ksf



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	R-2	7.5	0.5	16.3	99.7	103.8	0.697	0.599	2	70

Soil Identification: Light olive brown poorly-graded sand with silt (SP-SM)



ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435

Project No.: 13477.001

Rexford Raymond, Fullerton

04-22



DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Rexford Raymond, Fullerton](#)

Tested By: [G. Bathala](#)

Date: [03/24/22](#)

Project No.: [13477.001](#)

Checked By: [A. Santos](#)

Date: [04/12/22](#)

Boring No.: [LB-2](#)

Sample Type: [Ring](#)

Sample No.: [R-1](#)

Depth (ft.): [5.0](#)

Soil Identification: [Light brown poorly-graded sand with silt \(SP-SM\)](#)

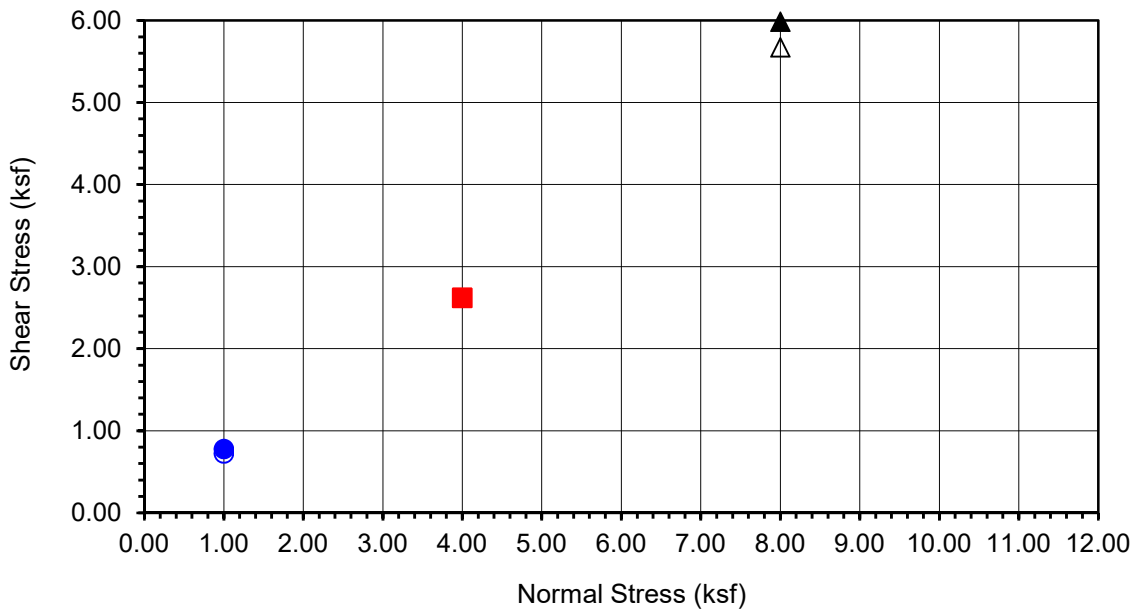
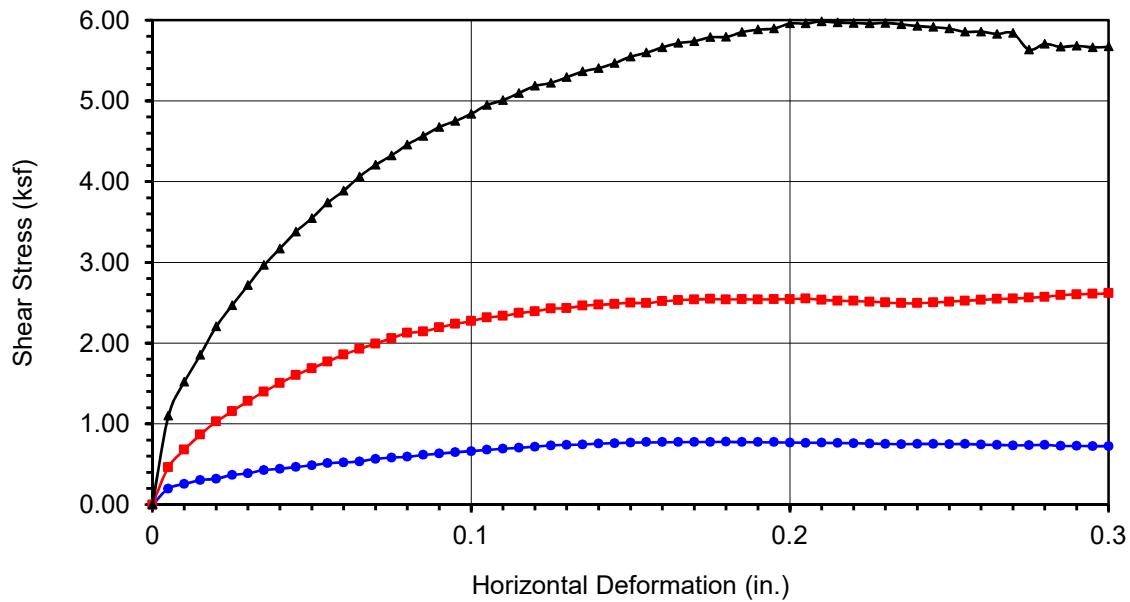
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	159.04	172.56	166.88
Weight of Ring(gm):	44.52	57.44	41.98

Before Shearing

Weight of Wet Sample+Cont.(gm):	181.72	181.72	181.72
Weight of Dry Sample+Cont.(gm):	178.65	178.65	178.65
Weight of Container(gm):	57.47	57.47	57.47
Vertical Rdg.(in): Initial	0.2421	0.2717	0.0000
Vertical Rdg.(in): Final	0.2829	0.3097	-0.0355

After Shearing

Weight of Wet Sample+Cont.(gm):	184.80	182.35	196.72
Weight of Dry Sample+Cont.(gm):	166.90	162.48	178.91
Weight of Container(gm):	60.23	56.91	61.68
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	LB-2
Sample No.	R-1
Depth (ft)	5
<u>Sample Type:</u>	
Ring	
<u>Soil Identification:</u>	
Light brown poorly-graded sand with silt (SP-SM)	

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 0.777	■ 2.616	▲ 5.983
Shear Stress @ End of Test (ksf)	○ 0.723	□ 2.616	△ 5.671
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	2.53	2.53	2.53
Dry Density (pcf)	92.9	93.4	101.3
Saturation (%)	8.4	8.5	10.3
Soil Height Before Shearing (in.)	0.9592	0.9620	0.9645
Final Moisture Content (%)	16.8	18.8	15.2



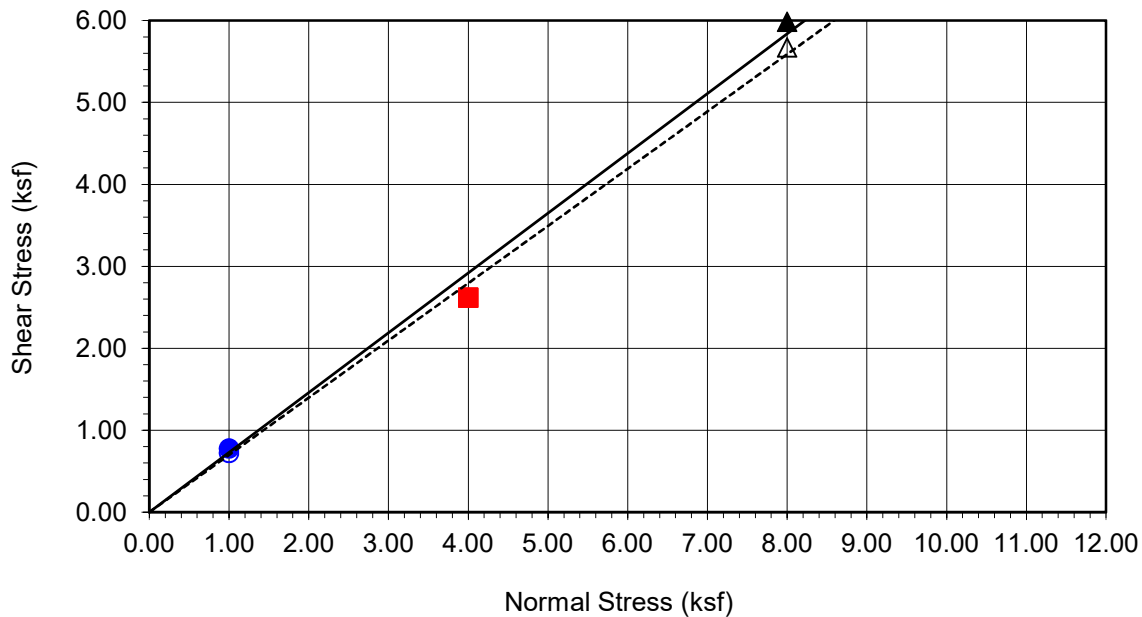
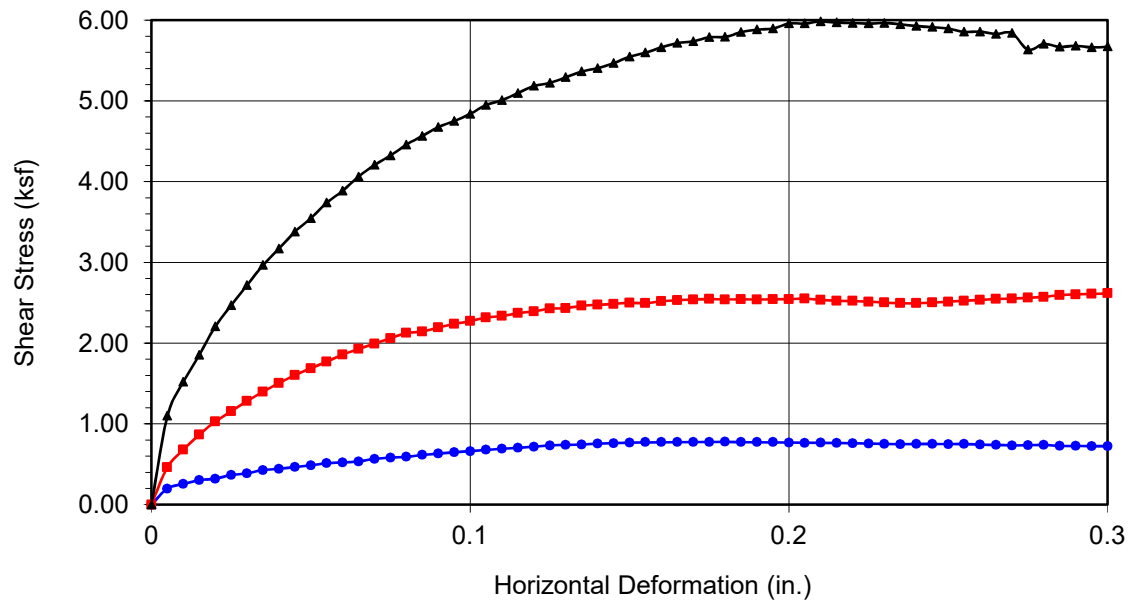
DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



Boring No.	LB-2	
Sample No.	R-1	
Depth (ft)	5	
<u>Sample Type:</u> Ring		
<u>Soil Identification:</u> Light brown poorly-graded sand with silt (SP-SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	0	36
Ultimate	0	35

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 0.777	■ 2.616	▲ 5.983
Shear Stress @ End of Test (ksf)	○ 0.723	□ 2.616	△ 5.671
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	2.53	2.53	2.53
Dry Density (pcf)	92.9	93.4	101.3
Saturation (%)	8.4	8.5	10.3
Soil Height Before Shearing (in.)	0.9592	0.9620	0.9645
Final Moisture Content (%)	16.8	18.8	15.2



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Rexford Raymond, Fullerton](#)

Tested By: [G. Bathala](#)

Date: [03/24/22](#)

Project No.: [13477.001](#)

Checked By: [A. Santos](#)

Date: [04/12/22](#)

Boring No.: [LB-3](#)

Sample Type: [95% Remold](#)

Sample No.: [B-1](#)

Depth (ft.): [0-5](#)

Soil Identification: [Dark brown silty clayey sand \(SC-SM\)](#)

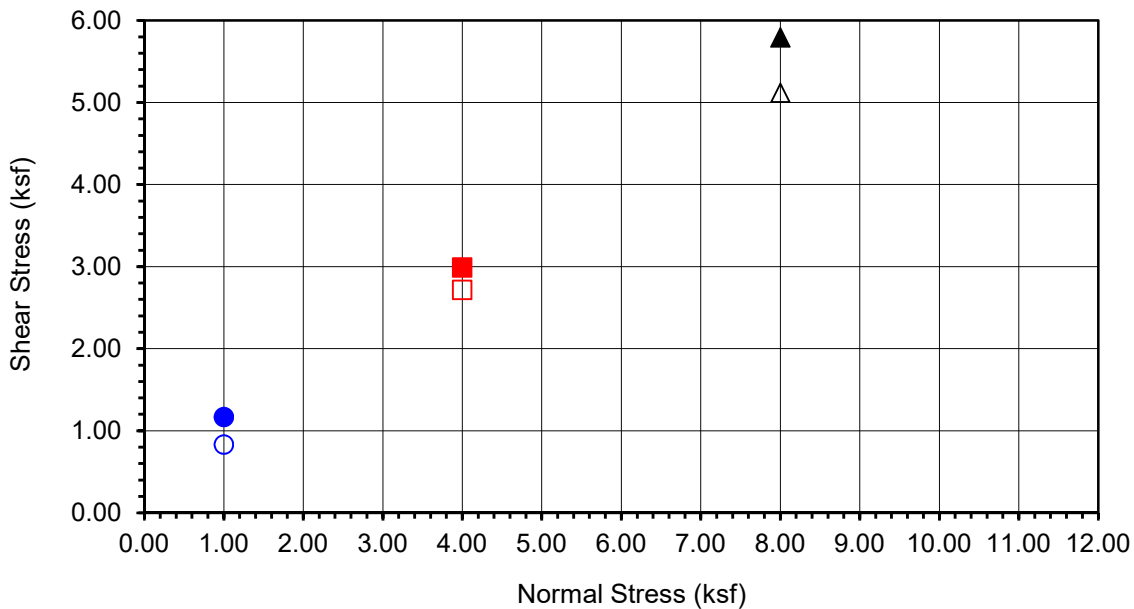
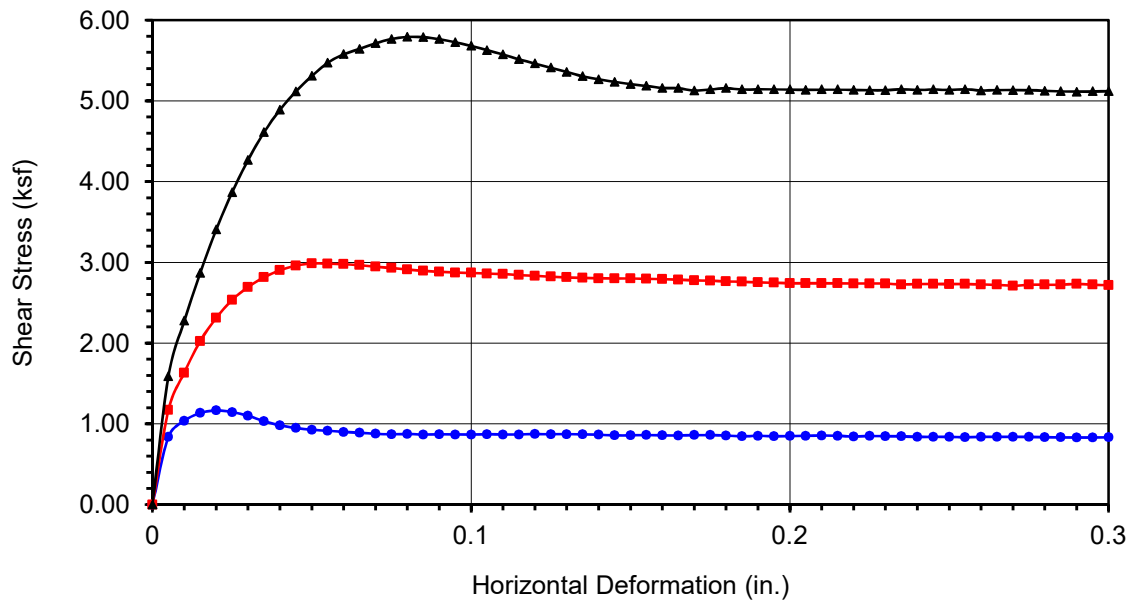
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	200.52	200.82	200.62
Weight of Ring(gm):	45.43	45.61	45.37

Before Shearing

Weight of Wet Sample+Cont.(gm):	180.48	180.48	180.48
Weight of Dry Sample+Cont.(gm):	169.13	169.13	169.13
Weight of Container(gm):	69.86	69.86	69.86
Vertical Rdg.(in): Initial	0.2454	0.2568	0.0000
Vertical Rdg.(in): Final	0.2515	0.2759	-0.0281

After Shearing

Weight of Wet Sample+Cont.(gm):	222.11	217.54	227.12
Weight of Dry Sample+Cont.(gm):	199.98	196.00	207.15
Weight of Container(gm):	66.92	58.11	69.44
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	LB-3
Sample No.	B-1
Depth (ft)	0-5
<u>Sample Type:</u>	
95% Remold	
<u>Soil Identification:</u>	
Dark brown silty clayey sand (SC-SM)	

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 1.166	■ 2.987	▲ 5.791
Shear Stress @ End of Test (ksf)	○ 0.833	□ 2.716	△ 5.118
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 (%)	11.43	11.43	11.43
Dry Density (pcf)	115.7	115.8	115.9
Saturation (%)	67.7	67.8	67.9
Soil Height Before Shearing (in.)	0.9939	0.9809	0.9719
Final Moisture Content (%)	16.6	15.6	14.5



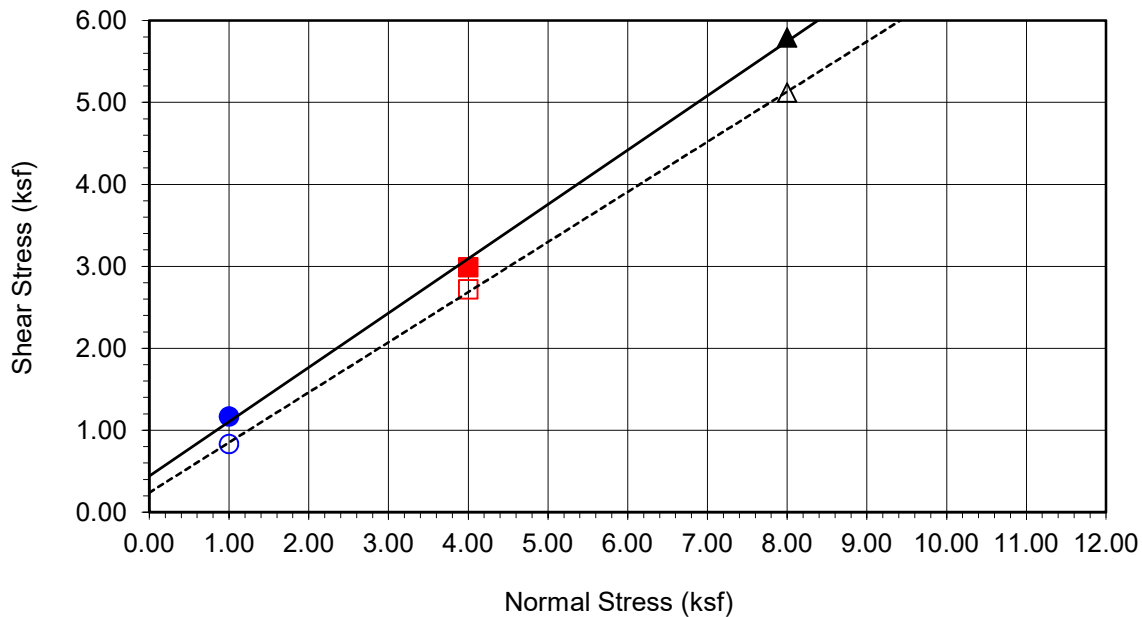
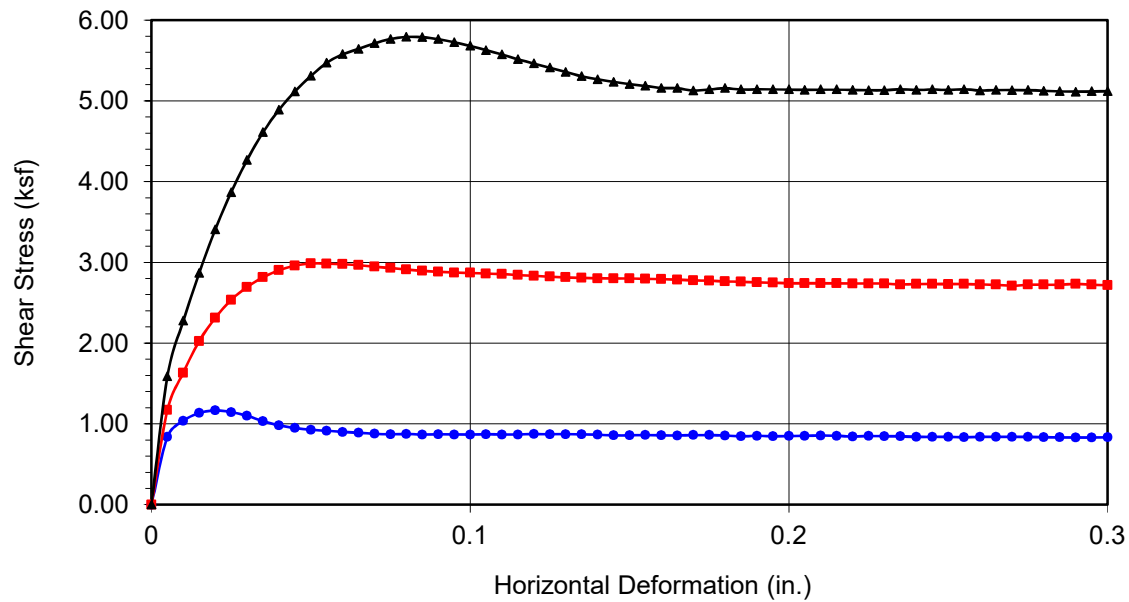
DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



Boring No.	LB-3	
Sample No.	B-1	
Depth (ft)	0-5	
<u>Sample Type:</u> 95% Remold		
<u>Soil Identification:</u> Dark brown silty clayey sand (SC-SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	442	34
Ultimate	239	31

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 1.166	■ 2.987	▲ 5.791
Shear Stress @ End of Test (ksf)	○ 0.833	□ 2.716	△ 5.118
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 (%)	11.43	11.43	11.43
Dry Density (pcf)	115.7	115.8	115.9
Saturation (%)	67.7	67.8	67.9
Soil Height Before Shearing (in.)	0.9939	0.9809	0.9719
Final Moisture Content (%)	16.6	15.6	14.5



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Rexford Raymond, Fullerton](#)

Tested By: [G. Bathala](#)

Date: [03/23/22](#)

Project No.: [13477.001](#)

Checked By: [A. Santos](#)

Date: [04/12/22](#)

Boring No.: [LB-3](#)

Sample Type: [Ring](#)

Sample No.: [R-2](#)

Depth (ft.): [7.5](#)

Soil Identification: [Light olive poorly-graded sand with silt \(SP-SM\)](#)

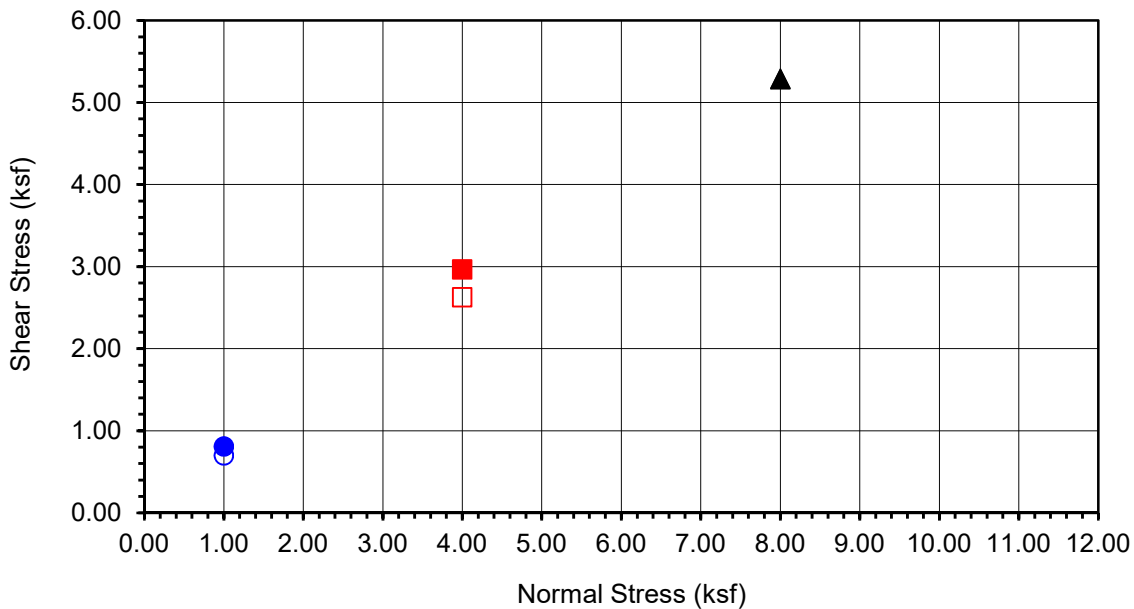
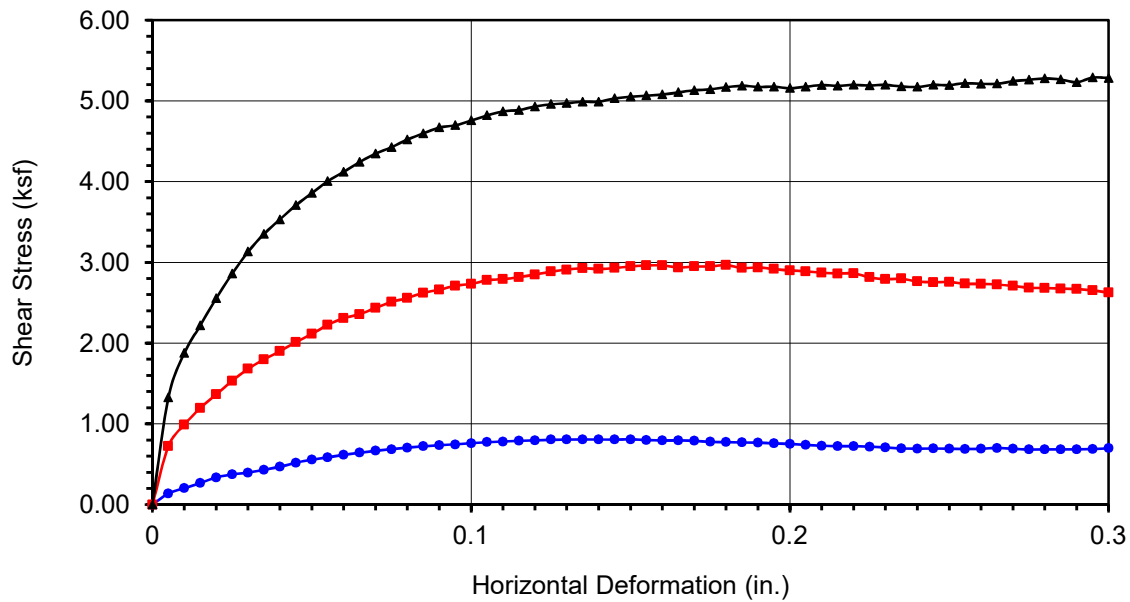
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	160.66	168.24	171.45
Weight of Ring(gm):	40.90	45.03	45.52

Before Shearing

Weight of Wet Sample+Cont.(gm):	206.48	206.48	206.48
Weight of Dry Sample+Cont.(gm):	205.70	205.70	205.70
Weight of Container(gm):	60.22	60.22	60.22
Vertical Rdg.(in): Initial	0.0000	0.2433	0.2578
Vertical Rdg.(in): Final	-0.0171	0.2950	0.3160

After Shearing

Weight of Wet Sample+Cont.(gm):	190.29	189.06	196.01
Weight of Dry Sample+Cont.(gm):	171.40	171.32	175.21
Weight of Container(gm):	61.67	61.74	56.90
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	LB-3
Sample No.	R-2
Depth (ft)	7.5
<u>Sample Type:</u>	
Ring	
<u>Soil Identification:</u>	
Light olive poorly-graded sand with silt (SP-SM)	

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 0.808	■ 2.965	▲ 5.288
Shear Stress @ End of Test (ksf)	○ 0.698	□ 2.625	△ 5.282
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	0.54	0.54	0.54
Dry Density (pcf)	99.1	101.9	104.2
Saturation (%)	2.1	2.2	2.3
Soil Height Before Shearing (in.)	0.9829	0.9483	0.9418
Final Moisture Content (%)	17.2	16.2	17.6



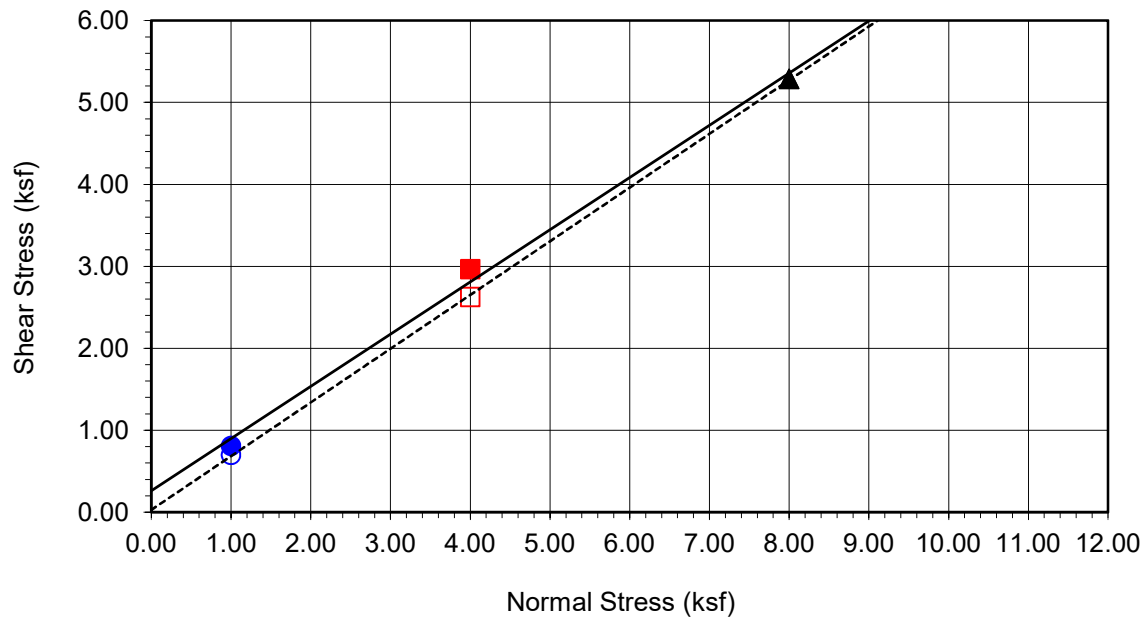
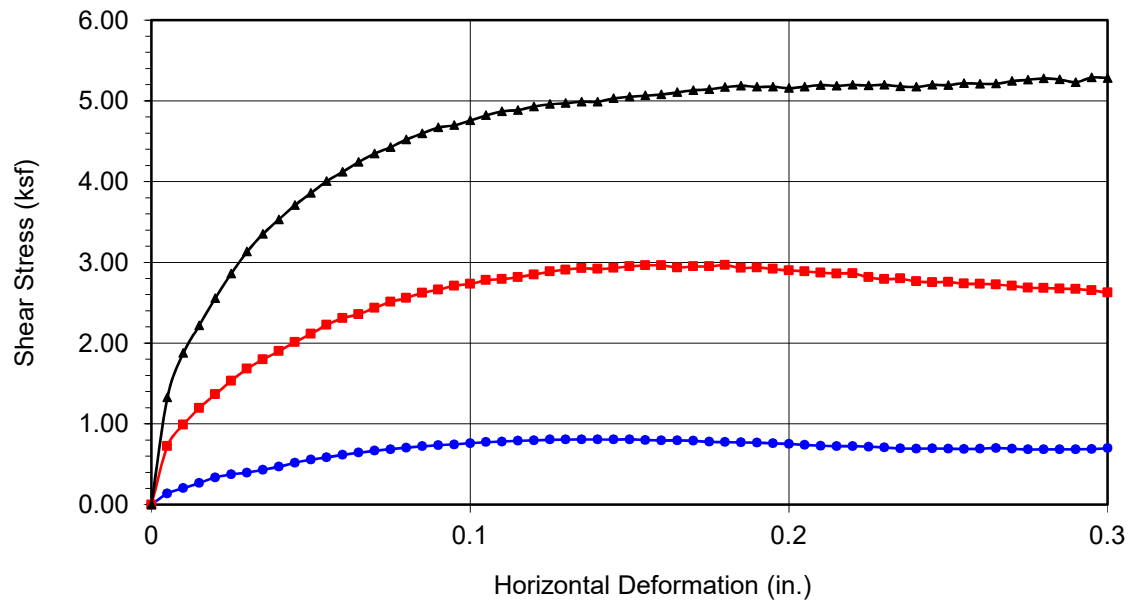
DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



Boring No.	LB-3	
Sample No.	R-2	
Depth (ft)	7.5	
<u>Sample Type:</u> Ring		
<u>Soil Identification:</u> Light olive poorly-graded sand with silt (SP-SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ ($^{\circ}$)
Peak	261	32
Ultimate	28	33

Normal Stress (kip/ft ²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft ²)	● 0.808	■ 2.965	▲ 5.288
Shear Stress @ End of Test (ksf)	○ 0.698	□ 2.625	△ 5.282
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	0.54	0.54	0.54
Dry Density (pcf)	99.1	101.9	104.2
Saturation (%)	2.1	2.2	2.3
Soil Height Before Shearing (in.)	0.9829	0.9483	0.9418
Final Moisture Content (%)	17.2	16.2	17.6



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13477.001

Rexford Raymond, Fullerton

03-22



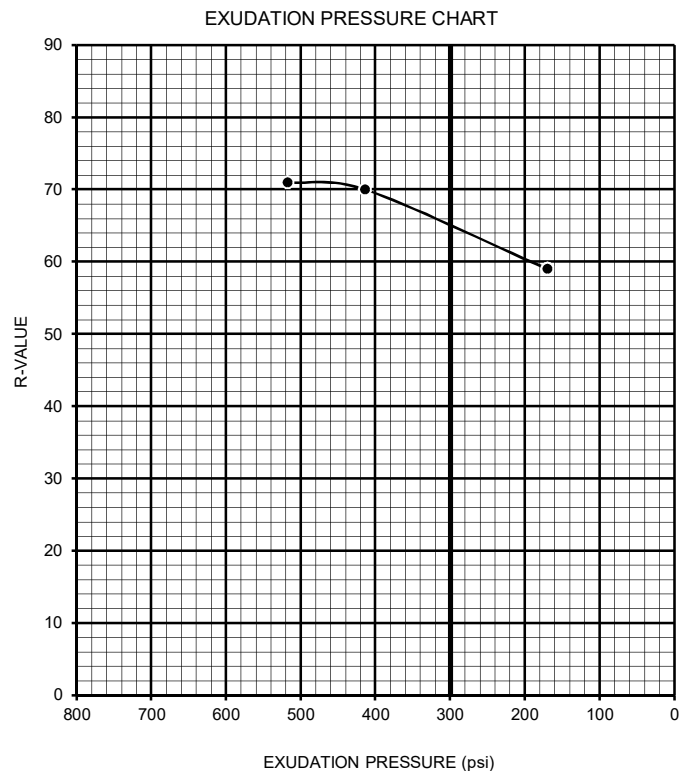
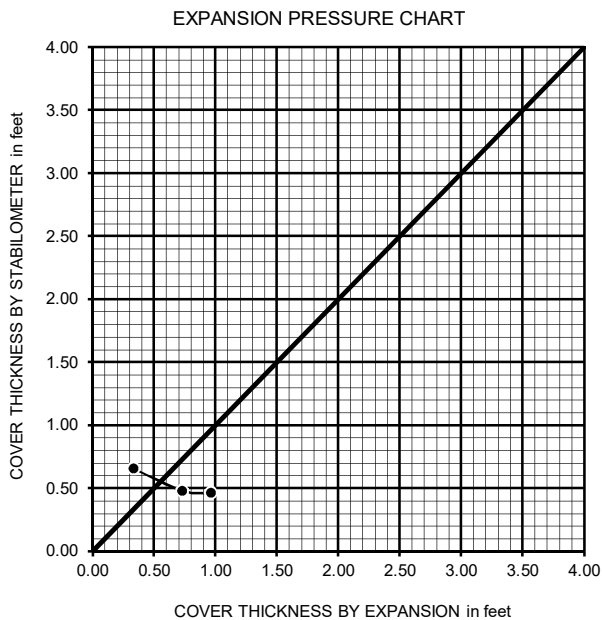
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME: Rexford Raymond, Fullerton PROJECT NUMBER: 13477.001
BORING NUMBER: LB-3 DEPTH (FT.): 0-5
SAMPLE NUMBER: B-1 TECHNICIAN: O. Figueroa
SAMPLE DESCRIPTION: Dark brown silty clayey sand (SC-SM) DATE COMPLETED: 3/24/2022

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.4	10.9	12.0
HEIGHT OF SAMPLE, Inches	2.46	2.48	2.49
DRY DENSITY, pcf	122.3	120.8	120.7
COMPACTOR PRESSURE, psi	300	230	150
EXUDATION PRESSURE, psi	517	414	169
EXPANSION, Inches x 10exp-4	29	22	10
STABILITY Ph 2,000 lbs (160 psi)	27	28	41
TURNS DISPLACEMENT	4.95	5.00	5.12
R-VALUE UNCORRECTED	71	70	59
R-VALUE CORRECTED	71	70	59

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.	0.46	0.48	0.66
EXPANSION PRESSURE THICKNESS, ft.	0.97	0.73	0.33



R-VALUE BY EXPANSION: 66
R-VALUE BY EXUDATION: 65
EQUILIBRIUM R-VALUE: 65



**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: Rexford Raymond, Fullerton Tested By : J. Domingo Date: 03/26/22
Project No. : 13477.001 Checked By: A. Santos Date: 04/12/22

Boring No.	LB-3			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Dark brown (SC-SM)			
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)	100.85			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	7			
Crucible No.	5			
Furnace Temperature (°C)	860			
Time In / Time Out	11:00/11:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.5037			
Wt. of Crucible (g)	18.4992			
Wt. of Residue (g) (A)	0.0045			
PPM of Sulfate (A) x 41150	185.18			
PPM of Sulfate, Dry Weight Basis	185			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO ₃ Soln. Used in Titration (C)	1.0			
PPM of Chloride (C -0.2) * 100 * 30 / B	80			
PPM of Chloride, Dry Wt. Basis	80			

pH TEST, DOT California Test 643

pH Value	8.11			
Temperature °C	21.5			



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Rexford Raymond, Fullerton
Project No. : 13477.001
Boring No.: LB-3
Sample No. : B-1

Tested By : J. Domingo Date: 04/07/22
Checked By: A. Santos Date: 04/12/22
Depth (ft.) : 0-5

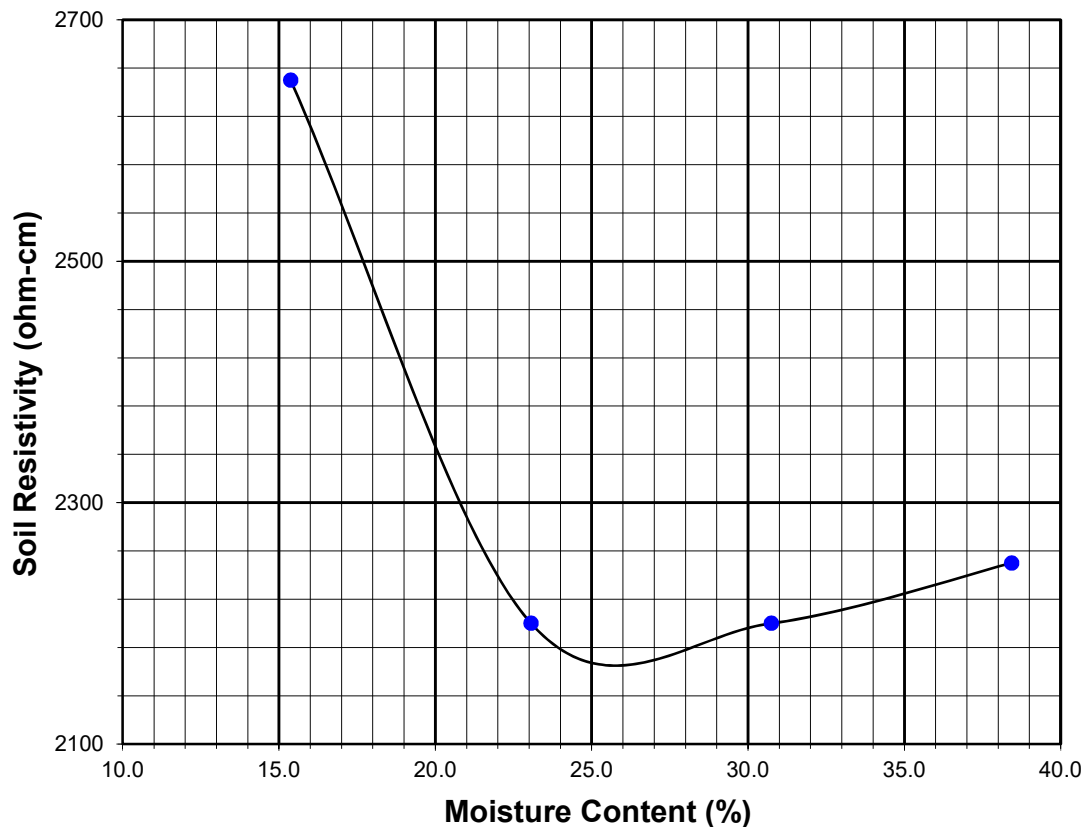
Soil Identification:* Dark brown (SC-SM)

*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	20	15.37	2650	2650
2	30	23.06	2200	2200
3	40	30.75	2200	2200
4	50	38.43	2250	2250
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)	130.10
Box Constant	1.000
$MC = (((1 + MC_i / 100) \times (W_a / W_t + 1)) - 1) \times 100$	

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	
2170	26.0	185	80	8.11	21.5



APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D

LEIGHTON CONSULTING, INC. EARTHWORK AND GRADING GUIDE SPECIFICATIONS

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D - 1 . 0 G E N E R A L

D-1.1 Intent

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

D-1.2 Role of Leighton Consulting, Inc.

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

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

D-1.3 The Earthwork Contractor

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

responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

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

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

D - 2 . 0 P R E P A R A T I O N O F A R E A S T O B E F I L L E D

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 F I L L M A T E R I A L

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 ($\leq$) 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 F I L L P L A C E M E N T A N D C O M P A C T I O N

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 ($\geq$) 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 ($\geq$) 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 E X C A V A T I O N

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 T R E N C H B A C K F I L L S

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.