GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
2411 N. GLASSELL STREET
CITY OF ORANGE, CALIFORNIA

Prepared For **REXFORD INDUSTRIAL REALTY & MANAGEMENT, INC.**
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Project No. 13333.001

December 20, 2021
(Revised January 12, 2022)

Leighton Consulting, Inc.
A Leighton Group Company

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Rexford Industrial Realty & Management, Inc.
11620 Wilshire Boulevard, 10th Floor
Los Angeles, California 90025

Attention: Mr. Michael Ramirez

Subject: Geotechnical Exploration Report
Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

Per our July 12, 2021 proposal, authorized on October 27, 2021, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. We understand the proposed development concept includes demolition of the existing buildings and site improvements to allow construction of a new one-story industrial building. Based on review of the *Conceptual Site Plan (Scheme A1-0) by Rexford Industrial Realty*, dated August 9, 2021 (delta 3 revision), and discussions with you, we understand the proposed development consists of a new one-story industrial building with a total building area of 277,000 square feet. The proposed concrete tilt-up building will be constructed at grade with associated truck parking, truck loading and 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.

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

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

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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TABLE OF CONTENTS

Tables

ATTACHMENTS

Appendices

1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 2411 N. Glassell Street in the city of Orange, Orange County, California. The site location (latitude 33.82838°, longitude -117.85110°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

The project site is a 12.5-acre, irregularly shaped parcel of land currently occupied by four (4) existing commercial buildings and associated paved access and surface parking. The site is bordered by N. Glassell Street to the west, Fletcher Avenue to the north, the Burlington Northern Santa Fe (BNSF) railroad to the east, and a former railroad spur followed by existing commercial properties to the south. Access to the site is via two (2) drive approaches that connect the property to N. Glassell Street.

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 Orange Quadrangle (USGS, 1981) indicates the ground surface at the site is between approximately Elevation (El.) +200 and +210 feet mean sea level (msl).

Based on review of historical aerial photographs (NETR, 2021), the site was an orchard until at least 1963. Between approximately 1966 and 1980, the site remained vacant and undeveloped. The existing buildings were constructed between 1980 and 1992. The site appears to have the same configuration since 1992.

Based on discussions with you, we understand the proposed development concept includes demolition of the existing buildings and site improvements to allow construction of a new one-story industrial building. Based on our review of the *Conceptual Site Plan (Sheet A1-0)* by Rexford Industrial Realty, dated August 9, 2021 (delta 3 revision), and discussions with you, we understand the proposed development consists of a new one-story industrial building with a total building area of 277,000 square feet. The proposed concrete tilt-up building will be constructed at grade with associated truck loading, surface parking, and Orange County Fire Department access. Ancillary improvements likely consist of utility infrastructure, pavement, flatwork, and landscaping. Preliminary structural data 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 November 4 and November 12, 2021, and included drilling, logging, and sampling a total of five (5) hollow-stem auger borings (designated LB-1 through LB-5) to approximate depths between 22 and 41 feet below the existing ground surface (bgs). Refusal was encountered at LB-2, LB-4, and LB-5 at 25.5, 41, and 22 feet bgs, respectively, prior to reaching target depth. 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 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);
  - Atterberg Limits (ASTM D 4318);
  - Direct Shear (ASTM D 3080);
  - Consolidation (ASTM D 2435);
  - Maximum Dry Density (ASTM D 1557);

- Expansion Index (ASTM D 4829);
- R-value; 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 site is located approximately 0.7 miles east of the Santa Ana River. The surficial deposits at the site are mapped to primarily consist of Quaternary-aged old alluvial fan deposits (Bedrossian and Roffers, 2010; Morton and Miller, 2006). A small area in the northwestern portion of the site is mapped to consist of Quaternary-aged young alluvial deposits deposited by the Santa Ana River and tributaries. The older alluvial deposits consist of slightly to moderately consolidated alluvial sediments are generally comprised of relatively flat-lying, non-marine deposits of boulder, cobble, gravel, sand, and silt. The Quaternary-age young alluvial fan deposits mapped in the northwestern portion of the site consist of unconsolidated alluvial sediments that are generally comprised of sand, silt, and clay. 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 old alluvial fan deposits (Qof). The artificial fill encountered in our borings at the explored locations is generally between 3 and 7 feet in thickness across the site, likely associated with the existing and previous site improvements. The fill soils consist primarily of locally derived clayey sand, clayey silt, silty clay, and clay with variable amounts of gravel. 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, older alluvial fan deposits (Qof) were encountered in the borings to the maximum depth explored (41 feet bgs). The alluvial sediments generally consist of light brown to brown, moist, medium dense to very dense, clayey sand with gravel, gravelly sand, silty sand and sandy gravel. A layer of brown, medium stiff, lean clay to sandy clay was encountered in the eastern portion of the site between approximately 15 to 20 feet bgs in boring LB-3 and between approximately 18 to 20 and 25 to 40 feet bgs in boring LB-4.

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.

Two (2) near-surface bulk soil samples obtained during our subsurface exploration were tested for expansion potential. Test results indicate Expansion Index (EI) values of 24 and 26 (“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, low expansion potential of site materials may be considered to support design and verified upon completion of earthwork grading.

2.3.2 Soil Corrosivity

Two (2) near-surface bulk soil samples 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 33 and 99 parts per million (ppm), chloride content of 40 ppm, pH values of 8.18 and 8.49, and minimum resistivity value of 2,197 and 2,498 ohm-cm.

The results of the resistivity test indicate the underlying soil is moderately to 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

Two (2) 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 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 (ø) and cohesion (c) for use in geotechnical analysis.

2.3.5 Excavation Characteristics

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

2.4 Groundwater Conditions

Groundwater was not encountered at the site during our subsurface exploration performed at the site to the maximum depth of 41 feet bgs. Based on review of the *Seismic Hazard Zone Report for the Orange Quadrangle* (CGS, 1997), the historically shallowest groundwater depth at the site is greater than 40 feet bgs. Based on review of available groundwater information from the California Department of Water Resources for a nearby groundwater monitoring well located approximately 0.9 miles northwest of the project site (State Well # 04S09W07N001S), the shallowest groundwater level measured for a monitoring period between June 1988 and February 2021 was approximately 96.6 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 and southern portions 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	12.5
LP-2	5 to 10	7.6

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 12.5 inches per hour (LP-1) and 7.6 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* (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 6.3 miles from the site. The San Andreas fault, which is the largest active fault in California, is approximately 37.7 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	33.82838°
Site Longitude	-117.85110°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.495 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.529 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}	1.495 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}	0.997 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	null[1]
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.694 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 Orange Quadrangle (CGS, 1998), the project site is **not** located within a seismically-induced landslide hazard zone identified by the State of California as being potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*). Based on this consideration, the potential for seismically-induced landslide hazards at the site is not considered a hazard at the site.

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 less than 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, earthquake-induced lateral spreading is also not considered a hazard at the site.

2.10 Earthquake-Induced Landsliding

As shown on the *Seismic Hazard Zones* map for the Orange 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), the project site is located within a flood hazard area identified as “Zone X”. A majority of the site is defined as an area of minimal flood hazard, and the western portion of the site is located in an area that is defined as an area with reduced flood risk due to levee. As shown on Figure 6, *Flood Hazard Zone Map*, the western portion of the site **is** located within a 500-year flood hazard zone. Regionally, storm runoff flow is generally directed to the southwest.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. 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, 2021). 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 structures may be supported on shallow spread-type foundations established in engineered fill or undisturbed natural 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.

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 Orange, 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

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. Earthwork for the project is expected to include overexcavation and recompaction of existing fill soils below new improvement footprints. Leighton should review the final grading plan and landscape plan when it becomes available to verify the recommendations in this report.

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 3 to 7 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. 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.

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 90 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 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

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

3.1.6 Shrinkage

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience.

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

3.1.7 Reuse of Concrete and Asphalt Rubble

If encountered during site clearing and/or during preparation activities, construction rubble (i.e., Portland cement concrete and asphalt concrete) may be incorporated in the proposed development. For use as structural fill, the processed material should be crushed to develop a relatively well-graded mixture with a maximum particle size of 3-inch nominal diameter. Concrete rubble should be free of rebar and processed asphalt pavement rubble may be used if mixed with the existing base course (where present). Processed material may be used as structural fill if uniformly mixed with onsite soils in proportion of 1 part processed asphalt 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 18 inches for continuous footings and 24 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 ½ 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 to very 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)	35
At-Rest (braced)	60
Passive Resistance (compacted fill)	300
Seismic Increment (add to active pressure)	20

Walls that are free to rotate or deflect may be designed using active earth pressure. For basement walls or walls that are fixed against rotation, the at-rest pressure should be used. For seismic condition, the pressure should be distributed as an inverted triangular distribution and the dynamic thrust should be applied at a height of 0.6H above the base of the wall. 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), 2018 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

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

3.6 Paving

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

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

3.6.1 Asphalt Concrete

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of engineered fill with an R-value greater than 15, 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 two (2) near surface samples of existing onsite soils indicate values of 16 and 18.

Table 4 – Asphalt Concrete Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5	4	6½
6	4	9½
7	5	11
8	6	13
9	6	16½

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 15. 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	6	4
6	6½	4
7	7	4
8	7½	4
9	8½	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 ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Near-surface onsite soils are to be considered Type B soils.

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

3.9 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”), 2018 Edition. Utility trenches can be backfilled with onsite sandy material free of rubble, debris, organic and oversized material up to (≤) 3-inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), water densified in place, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* (“Greenbook”), 2018 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 sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> 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 <u>Not</u> Covered

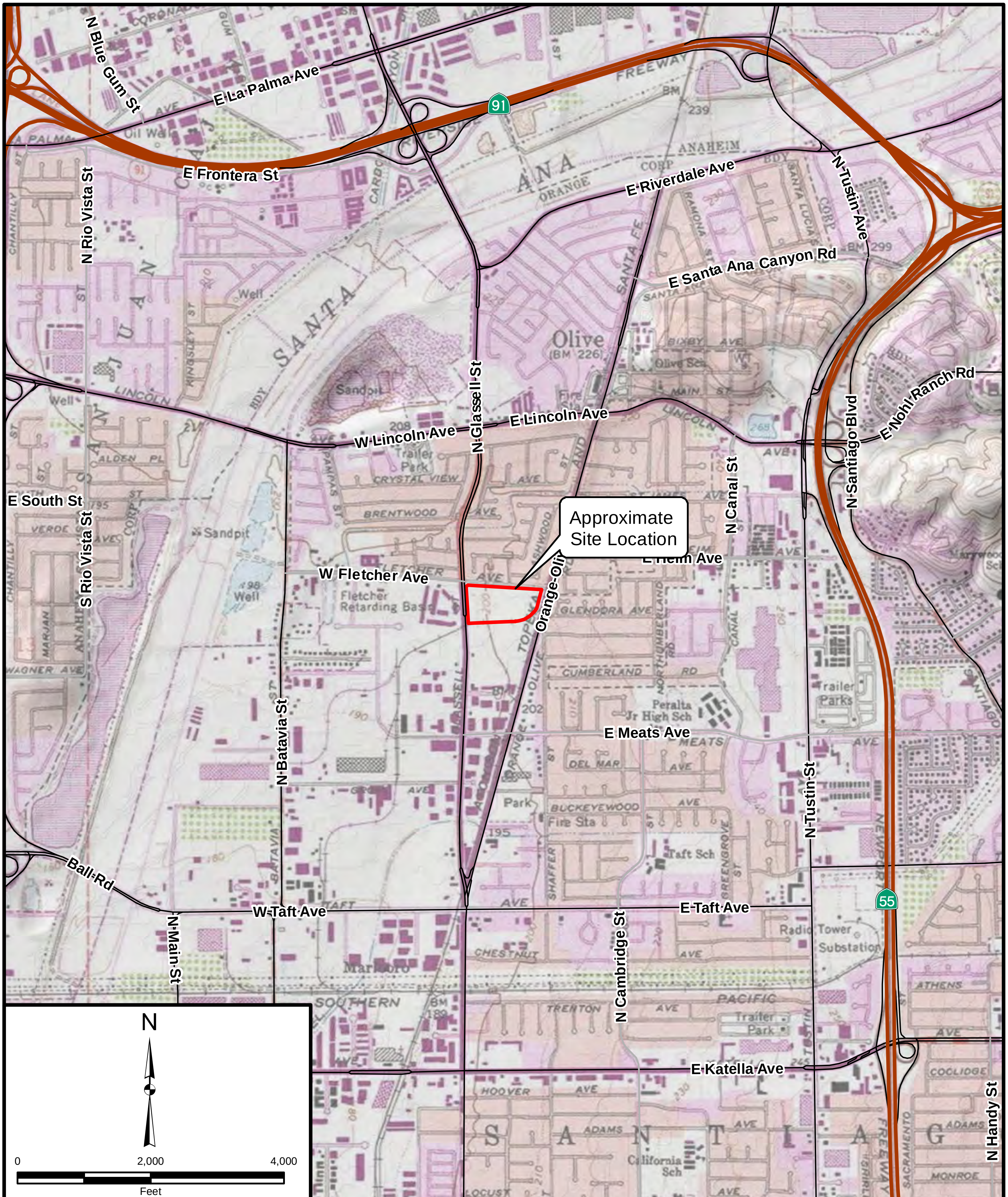
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: December 2021

Base Map: ESRI ArcGIS Online 2021

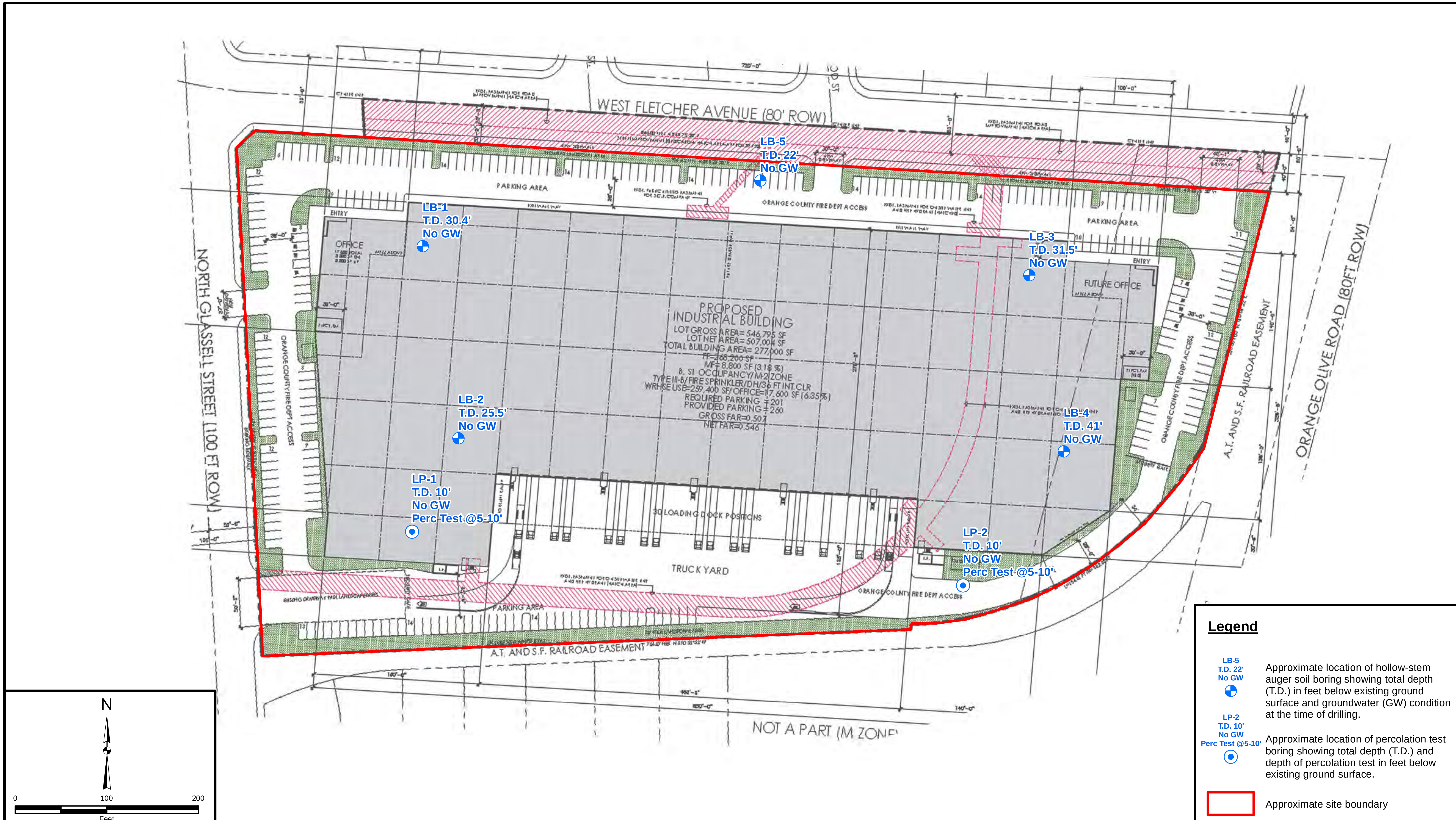
SITE LOCATION MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F01_SLM_2021-11-16.mxd on 12/20/2021 10:59:59 AM Author: KVM (kmanchikanti)



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 100 '	Date: December 2021
Base Map: ESRI ArcGIS Online 2021	

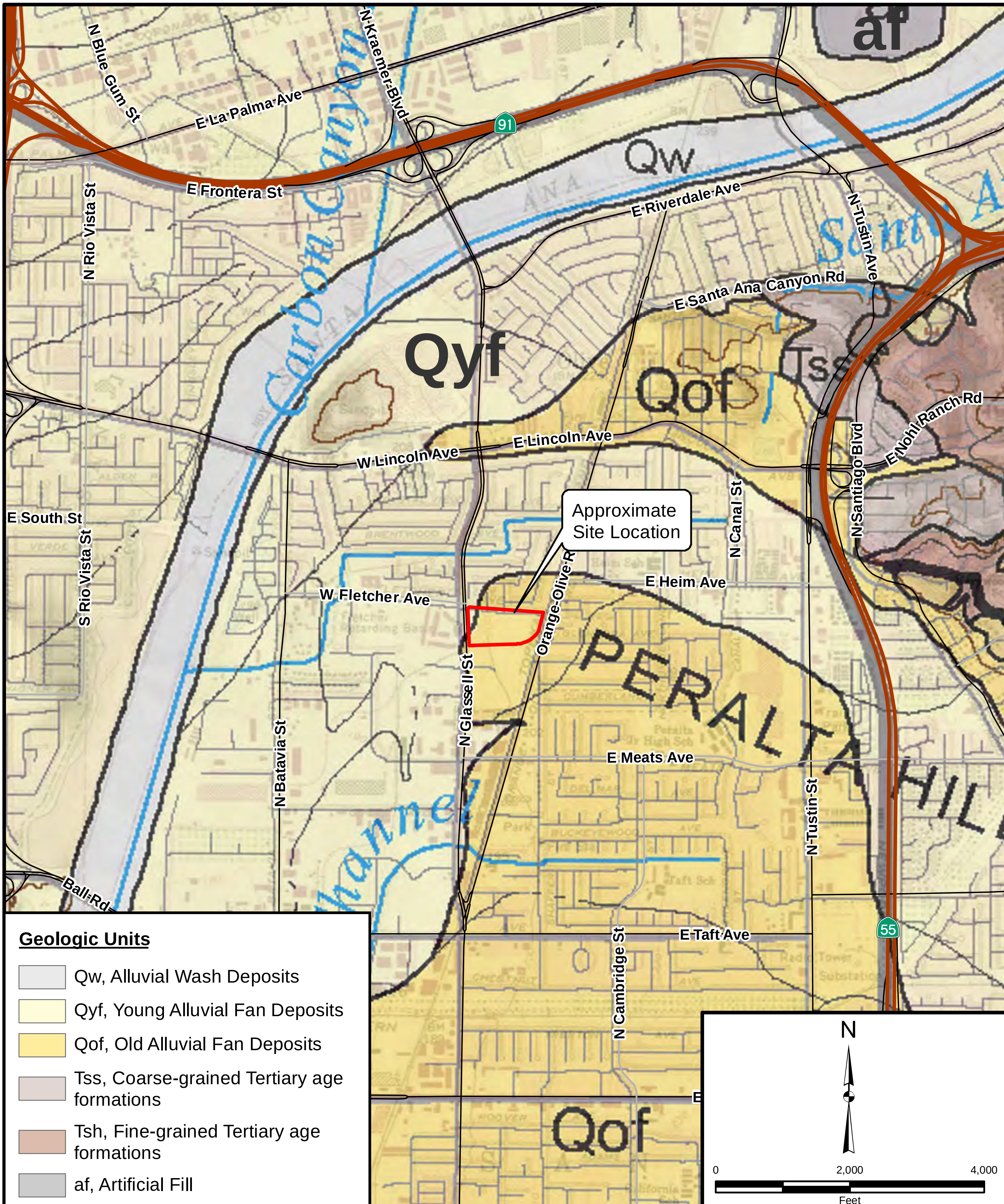
EXPLORATION LOCATION MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F02_ELM_2021-11-16.mxd on 12/20/2021 11:13:19 AM Author: KVM (kmanchikanti)



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: December 2021

Basemap: USGS Topo Map Service from Esri, 2021
Reference: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, Bedrossian et. al., 2010

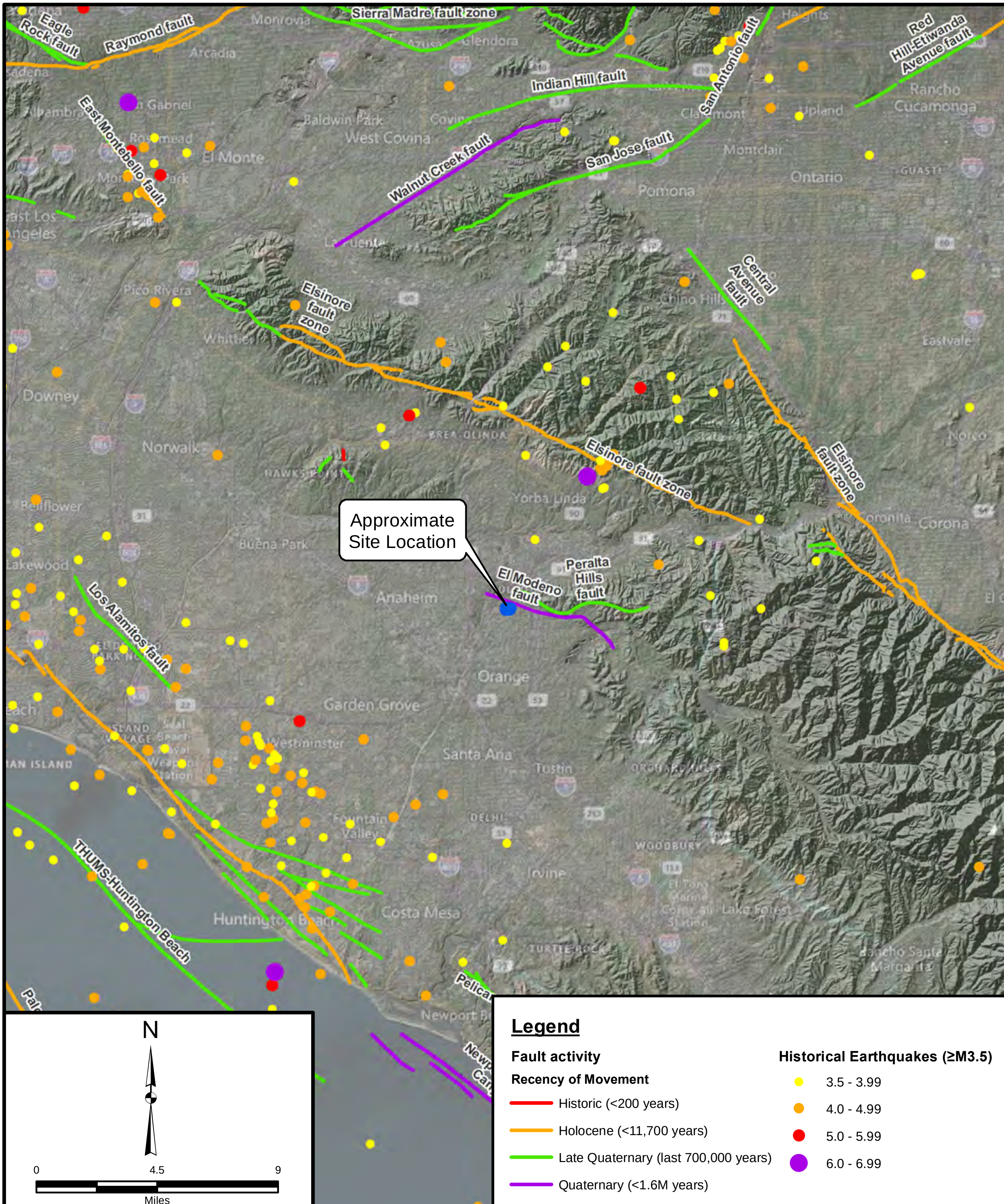
REGIONAL GEOLOGY MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 3

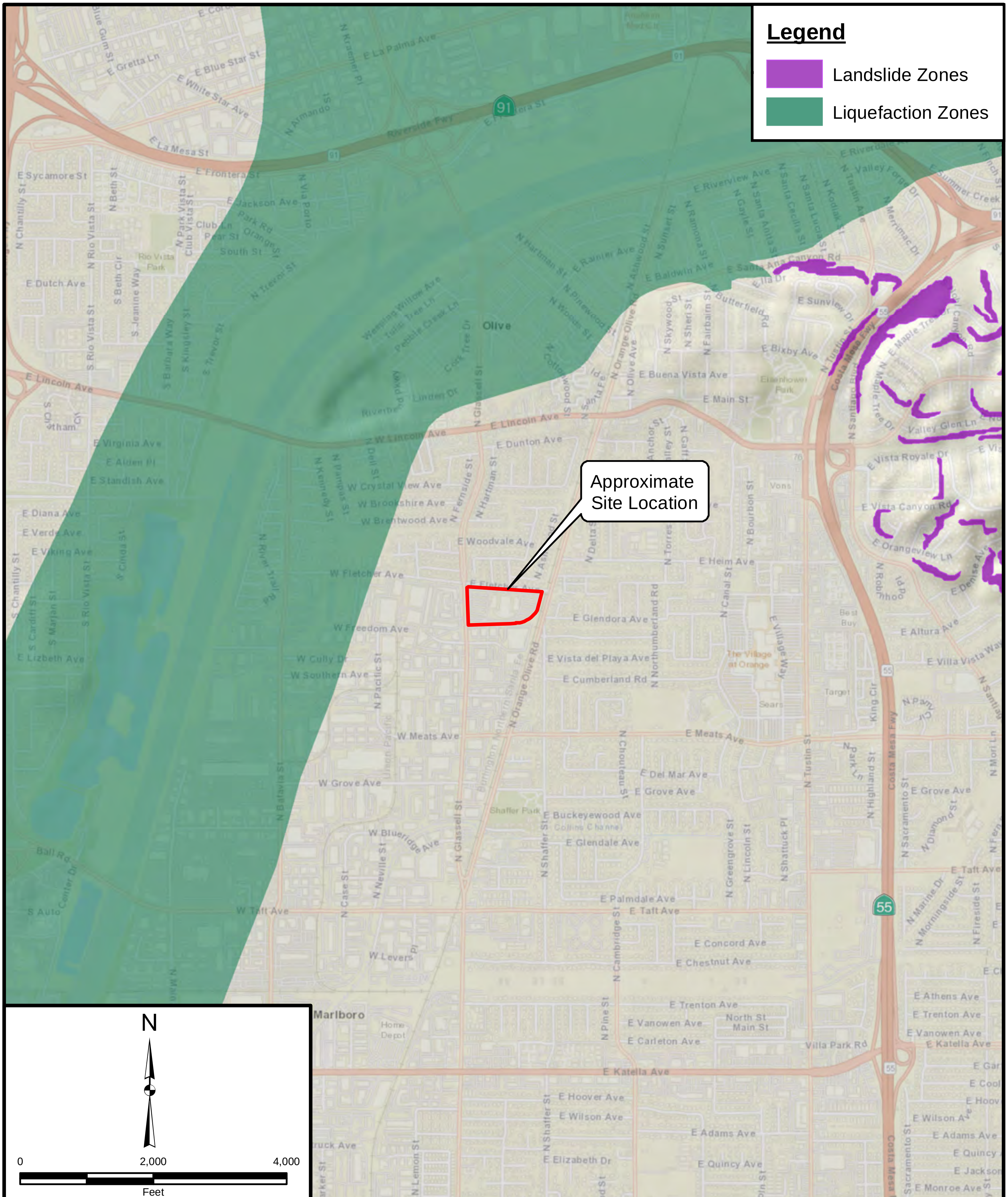
Leighton

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F03_RGM_2021-11-16.mxd on 12/20/2021 11:14:18 AM Author: KVM (kmanchikanti)



Project: 13333.001	Eng/Geol: CCK/JMP	REGIONAL FAULTS AND HISTORIC SEISMICITY MAP	FIGURE 4
Scale: 1 " = 5 miles	Date: December 2021	Proposed Industrial Building 2411 N. Glassell Street City of Orange, California	Leighton
Base Map: ESRI ArcGIS Online 2021 Reference: maps.conservation.ca.gov			

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F04_RFHSM_2021-11-16.mxd on 12/1/2021 3:31:42 PM Author: KVM (kmanchikanti)



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: December 2021

Base Map: ESRI ArcGIS Online 2021
Reference: maps.conservation.ca.gov

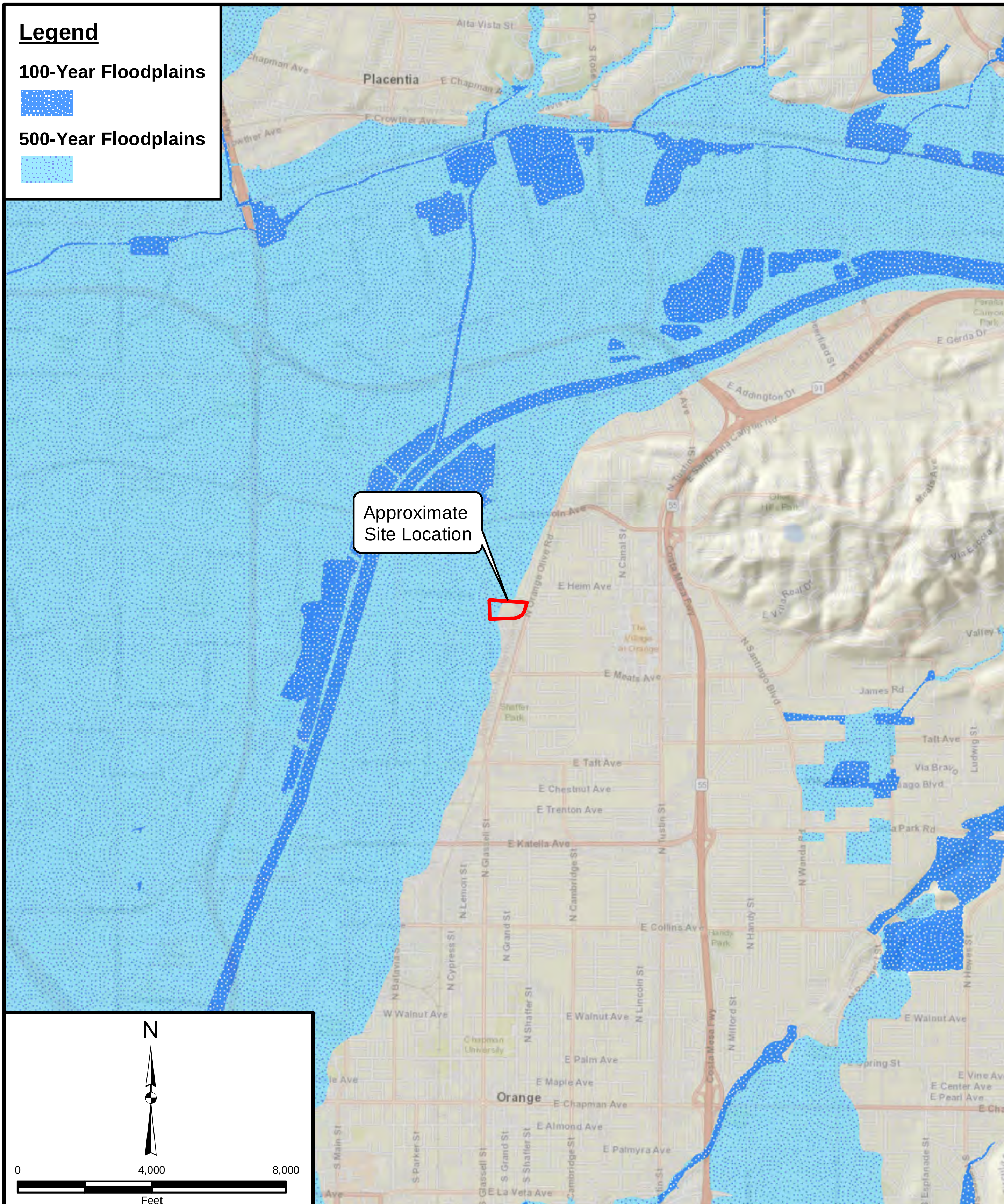
SEISMIC HAZARD MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 5

Leighton

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F05_SHM_2021-11-16.mxd on 12/20/2021 11:18:24 AM Author: KVM (kmanchikanti)



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: December 2021

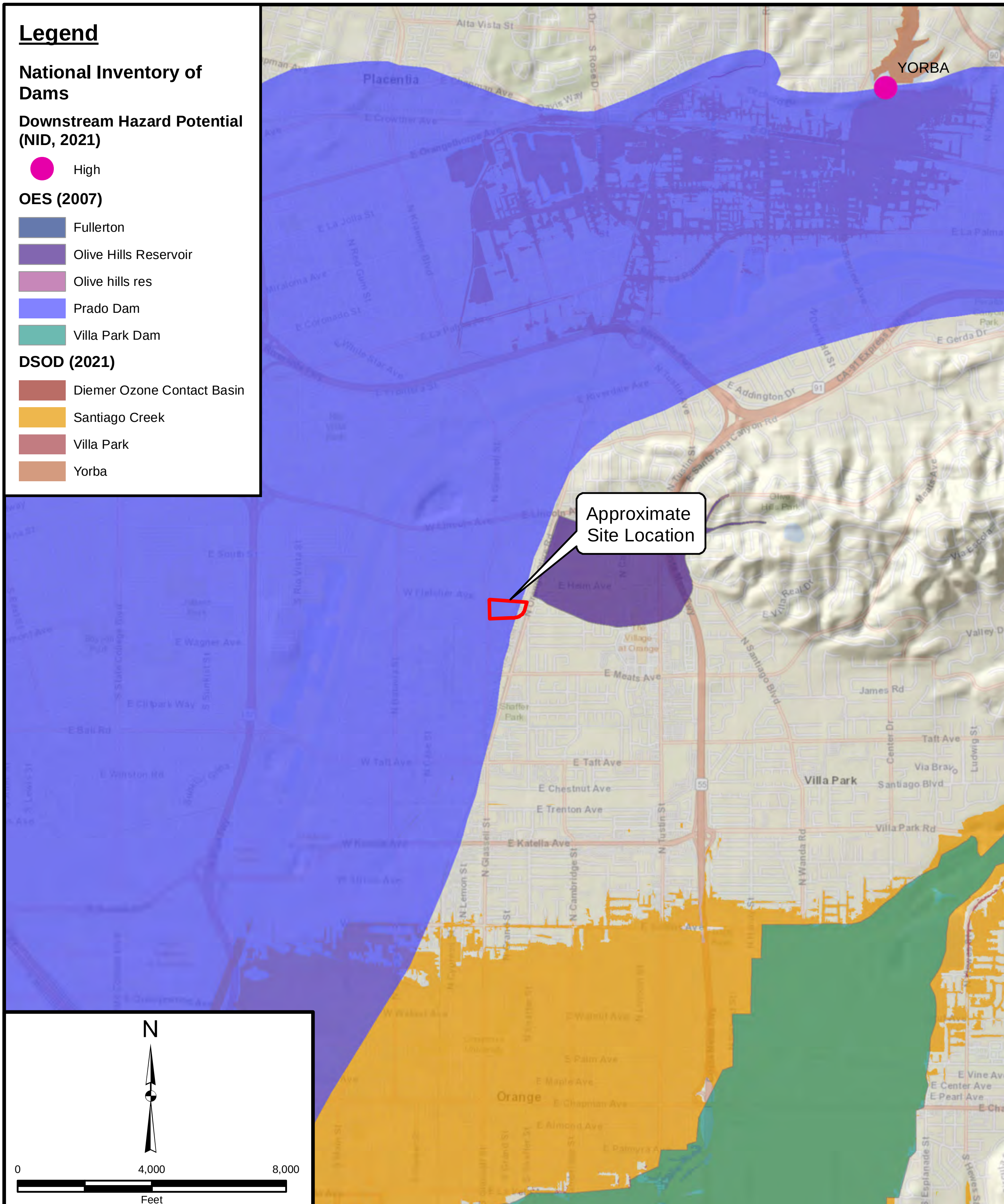
Base Map: ESRI ArcGIS Online 2021
Reference: FEMA, DWR

FLOOD HAZARD ZONE MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 6

Leighton



Project: 13333.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: December 2021

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

DAM INUNDATION MAP

Proposed Industrial Building
2411 N. Glassell Street
City of Orange, California

FIGURE 7

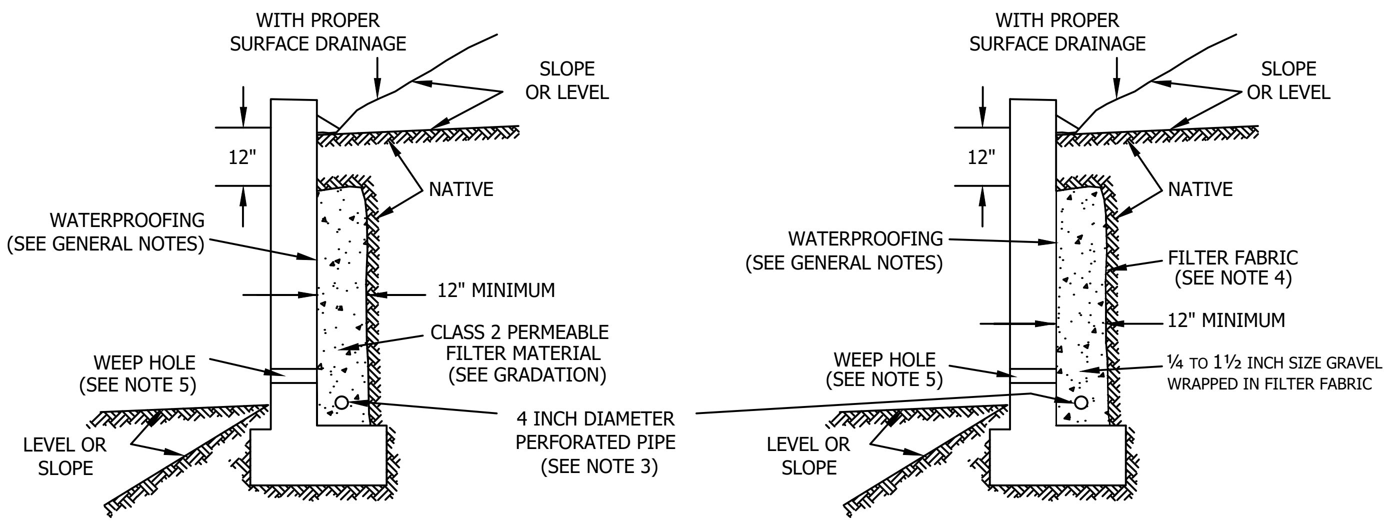
Leighton

Map Saved as V:\Drafting\13333\001\Maps\13333-001_F07_DIM_2021-11-16.mxd on 12/20/2021 11:17:50 AM Author: KVM (kmanchikanti)

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

Leighton

FIGURE 8

V:\DRAFTING\TEMPLATES\STANDARD-FIGURES\ALL-STANDARD-FIGURES.DWG (08-02-21 9:07:35AM) Plotted by: btran

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~203'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 3.5 inches of asphalt over 4 inches of base	
				B-1				CL-ML	**Artificial Fill, Undocumented (Afu):** @0.625': Sandy Silty CLAY, dark brown, moist, low plasticity	
	5			S-1	5 4 3		7	SP	@5': Poor recovery. Poorly-graded SAND, loose to medium dense, brown, moist, fine to medium sand, fine gravel, little to some silt	
									Quaternary Old Alluvium (Qof): @7': Gravelly SAND, brown, moist	
	10			R-2	23 50/5"			SC	@10': Gravelly Clayey SAND, very dense, brown, moist, fine to coarse sand, fine to coarse gravel, subangular	
	15			S-3	32 43 50/5"		3		@15': Gravelly Clayey SAND, very dense, brown, moist, fine to coarse sand, fine to coarse gravel, subangular gravel, mechanically fractured gravel	
	20			R-4	6 12 12			SC	@20': Clayey SAND with Gravel, medium dense, brown, moist, fine to coarse sand, fine gravel, clay layer in shoe	
	25			S-5	14 24 50/4"		3	SP	@25': Gravelly SAND, very dense, brown, moist, fine to coarse sand, fine to coarse gravel @26': Difficult drilling due to gravel	
	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-1

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~203'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30	. . . .		R-6	50/5"	132	4		@30': Gravelly SAND, very dense, fine cobble, rounded **Total Depth: 30.4 feet** **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
	35									
	40									
	45									
	50									
	55									
	60									

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

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

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~202'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 3.5 inches of asphalt over 5 inches of base	
				B-1				ML-CL	**Artificial Fill, Undocumented (Afu):** @0.71': Silty CLAY, dark brown, moist, low plasticity	
	5			R-1	4 8 8	123	12	CL	@5': CLAY, stiff, dark brown, moist, medium plasticity, fine to coarse gravel, trace asphalt pieces	
								SC	**Quaternary Old Alluvium (Qof):** @7': Clayey SAND with Gravel, brown, moist	
	10			S-2	14 34 32		4		@10': Clayey SAND with Gravel, very dense, moist, brown, fine sand, fine to coarse gravel, FeO staining on gravel @11': Encountered gravel	
	15			R-3	50/3"				@15': No recovery. Very dense, possible fine cobbles @16': Gravelly soil cuttings	
	20			S-4	50/5"		6		@20': Gravelly Clayey SAND, very dense, brown, moist	
	25			R-5	50/0"				@25': No recovery. Very dense **Total Depth: 25.5 feet** **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
	30									

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.

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.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~205'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								@Surface: 5 inches of asphalt over 4 inches of base	
				B-1				ML	**Artificial Fill, Undocumented (Afu):** @0.75': Clayey SILT with Gravel, brown, moist, fine to coarse gravel, trace cobbles	
								SC	**Quaternary Old Alluvium (Qof):** @3': Gravelly Clayey SAND, brown, moist, fine to coarse sand and gravel, some silt	
	5			S-1	5 23 44		5		@5': Gravelly Clayey SAND, very dense, brown, moist, fine to coarse sand and gravel, subangular, angular from mechanical fracturing. Difficult drilling from 5' to 15'	
	10			R-2	29 50/5"	122	5		@10': Gravelly SAND with Clay, very dense, brown, moist, fine to coarse sand & gravel, subangular, clayey at ~11'	
	15			S-3	6 4 2		19	CL	@15': CLAY, medium stiff, brown, moist, medium plasticity	
								GP	@18': Refusal. Stepped over and redrilled to 20'	
	20								@20': No recovery. Encountered difficult drilling	
	25			R-4	50/4"	90	3		@25': Sandy GRAVEL, very dense, brown, moist, fine sand, mostly cobble fragments in sampler, fine to coarse gravel, some cobbles, fine to coarse cobble	
	30									

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

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

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

GEOTECHNICAL BORING LOG LB-3

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~205'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S-5	17 14 8		14	ML-CL	@30': Silty CLAY, hard, moist, brown, low to medium plasticity, few fine to coarse gravel fragments, angular to subangular	
									Total Depth: 31.5 feet **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
	35									
	40									
	45									
	50									
	55									
	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.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~204'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								@Surface: 4 inches of asphalt over 4 inches of base	
				B-1				SC	**Artificial Fill, Undocumented (Afu):** @0.67': Clayey SAND with gravel, brown, moist, low to medium plasticity	
	5			R-1	50/4"	95	6	SC	**Quaternary Old Alluvium (Qof):** @5': Gravelly Clayey SAND, very dense, moist, brown, fine to coarse sand, fine to coarse gravel, subangular gravel, some silt. Difficult drilling	
	10			S-2	50/4"		5		@10': Low recovery. Very dense	
									@11': Increased gravel and cobbles	
	15			R-3	10 8 8				@15': No recovery. Gravelly SAND, medium dense, brown, moist, fine to coarse sand and gravel	
								CL	@18': Cuttings become CLAY, brown, moist, medium plasticity	
	20			S-4-1	31 34		19		@20': CLAY, hard, brown, moist, trace fine gravel, medium plasticity	
				S-4-2	50/6"		5	SP	@21': Becomes Gravelly SAND with Clay, brown, moist, fine to coarse sand and gravel, subangular gravel, decrease in fines with depth	
	25			R-5	50/4"	112	16	CL	@25': CLAY, hard, brown, moist	
								SP-CL	@26': Becomes Sandy CLAY, hard, moist, brown, fine to coarse sand, fine gravel, coarse gravel in shoe	
	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-4

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~204'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S-6	3 5 6		19		@30': Sandy CLAY, very stiff, brown, moist, fine to coarse sand, fine gravel, medium plasticity	
	35			R-7	50/5"	100	16	CL	@35': CLAY, hard, moist, brown, medium plasticity, fine gravel, subangular	
	40			S-8	50/0"				@40': No recovery	
								COBBLES	@41: Grinding on cobble	
									Total Depth: 41 feet **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
	45									
	50									
	55									
	60									

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

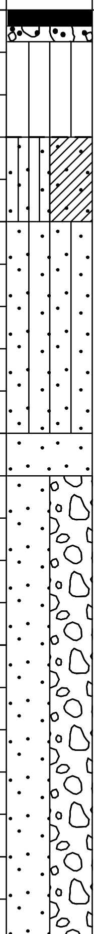
TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY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

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

GEOTECHNICAL BORING LOG LB-5

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~200'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Graphic Log (N / S)



Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							@Surface: 4.5 inches of asphalt over 4.5 inches of base	
			B-1				ML	**Artificial Fill, Undocumented (Afu):** @0.75': SILT with Gravel and Sand, brown, moist, fine gravel, little fine sand	
							SC-SM	**Quaternary Old Alluvium (Qof):** @3': Gravelly Clayey SAND to Silty SAND, light brown	
	5		R-1	50/6"	114	5	SM	@5': Silty SAND, very dense, brown, moist, fine sand, fine to coarse gravel @6': Encountered increase in gravel	
	10		S-2	14 26 28		3	SP SP-GP	@10': Poorly-graded SAND with Gravel, very dense, moist, fine to coarse sand, mostly fine sand, fine to coarse gravel, angular from mechanical fracturing @11: Cuttings appear to be Sandy GRAVEL	
	15		R-3	43 50/3"	122	4		@15': Gravelly SAND to Sandy GRAVEL, very dense, light brown, moist, fine to coarse gravel, fine cobble, subangular gravel	
	20		S-4	25 50/5"		4		@20': Very dense. Difficult drilling from ~20' to 22'	
								Total Depth: 22 feet **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
	25								
	30								

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.

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

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

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

GEOTECHNICAL BORING LOG LP-1

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~202'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 3.5 inches of asphalt over 5 inches of base	
								SC	**Artificial Fill, Undocumented (Afu):** @0.71': Clayey SAND, light brown, moist, fine sand	
								SC	**Quaternary Old Alluvium (Qof):** @3': Gravelly Clayey SAND, brown, moist, fine sand	
	5			S-1	10 22 25		6	SM	@5': Silty SAND, very dense, brown, moist, fine to medium sand, fine to coarse gravel, little to some silt	
				R-2	29 42 50/6"	119	4		@8.5': Silty SAND, very dense, fine to coarse sand, mostly fine to medium sand, FeO staining on gravel surfaces	
	10								**Total Depth: 10 feet** **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.** **Upon completion of percolation test, well casing removed, boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
	15									
	20									
	25									
	30									

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

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- 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 LP-2

Project No.	13333.001	**Date Drilled**	11-4-21
Project	Rexford Glassell St. Orange	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~204'
Location	See Figure 2- Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0							SM	@Surface: 3.5 inches of asphalt over 5.5 inches of base **Artificial Fill, Undocumented (Afu):** @2.5': Gravelly SILT with Sand to Silty SAND	
								CL	**Quaternary Old Alluvium (Qof):** @3': Gravelly SILT with Sand, brown, moist	
	5			S-1	15 12 29		11		@5': Silty CLAY, hard, slightly moist, brown, little fine sand, fine subangular gravel, porous @7': Refusal. Stepped over and drilled down to 8.5' and sampled	
				R-2	21 50/4"	120	6	ML	@8.5': Sandy SILT, hard, brown, slightly moist, fine sand, subangular, fine to coarse gravel	
	10								**Total Depth: 10 feet** **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.** **Upon completion of percolation test, well casing removed, boring backfilled with soil cuttings and patched with cold-mix asphalt.**	
	15									
	20									
	25									
	30									

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

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- 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:	13333.001	**Test Hole Number:**	LP-1
Project Name:	Rexford Glassell	**Date Excavated:**	11/4/2021
Earth Description:	Alluvium	**Date Tested:**	11/12/2021
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	MM	**Radius of boring, r (in):**	4
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	7
		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	10:40	-	-	-	0.0
2	10:50	10	6.53	41.6	10.0
3	11:00	10	6.49	42.1	19.9
4	11:10	10	6.47	42.4	29.9
5	11:20	10	6.50	42.0	39.9
6	11:30	10	6.51	41.9	49.8
7	11:40	10	6.51	41.9	59.8
8	11:50	10	6.49	42.1	69.8

Total Volume of Water Delivered (gallons)	69.8
Total Volume of Water Delivered (cubic inches)	16116.639
Average Water Height (inches)	42.0
Average Percolation Surface Area (cubic Inches)	1105.8
Duration of Test (minutes)	70
Duration of Test (hours)	1.17

$$\text{Measured Infiltration Rate} = (\text{Total Volume})/(\text{Test Duration})/(\text{Surface Area})$$

Measured Infiltration Rate (inches per hour) = 12.5

Boring Percolation Test Data Sheet

Project Number:	13333.001	**Test Hole Number:**	LP-2
Project Name:	Rexford Glassell	**Date Excavated:**	11/4/2021
Earth Description:	Alluvium	**Date Tested:**	11/12/2021
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	MM	**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	7:50	-	-	-	0.0
2	8:00	10	6.02	47.8	7.0
3	8:10	10	5.97	48.4	13.9
4	8:20	10	5.93	48.8	20.9
5	8:30	10	5.93	48.8	27.8
6	8:40	10	5.94	48.7	34.8
7	8:50	10	5.95	48.6	41.7
8	9:00	10	5.96	48.5	48.7

Total Volume of Water Delivered (gallons)	48.7
Total Volume of Water Delivered (cubic inches)	11238.15
Average Water Height (inches)	48.5
Average Percolation Surface Area (cubic Inches)	1269.6
Duration of Test (minutes)	70
Duration of Test (hours)	1.17

$$\text{Measured Infiltration Rate} = (\text{Total Volume})/(\text{Test Duration})/(\text{Surface Area})$$

Measured Infiltration Rate (inches per hour) = 7.6

APPENDIX C

LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Rexford Glassell	Tested By:	J. Gonzalez	Date:	11/09/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Yellowish brown sandy silty clay s(CL-ML)				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist		
		Dry		
Compaction Method	X	Mechanical Ram		
		Manual Ram		

Scalp Fraction (%)	
#3/4	
#3/8	
#4	11.4

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3865	3989	3890			
Weight of Mold (g)	1850	1850	1850			
Net Weight of Soil (g)	2015	2139	2040			
Wet Weight of Soil + Cont. (g)	546.0	527.0	463.5			
Dry Weight of Soil + Cont. (g)	514.4	485.6	415.9			
Weight of Container (g)	40.1	39.0	40.1			
Moisture Content (%)	6.66	9.27	12.67			
Wet Density (pcf)	133.4	141.6	135.1			
Dry Density (pcf)	125.1	129.6	119.9			

Maximum Dry Density (pcf)	**129.6**	**Optimum Moisture Content (%)**	**9.2**
Corrected Dry Density (pcf)	**133.1**	**Corrected Moisture Content (%)**	**8.3**

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

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

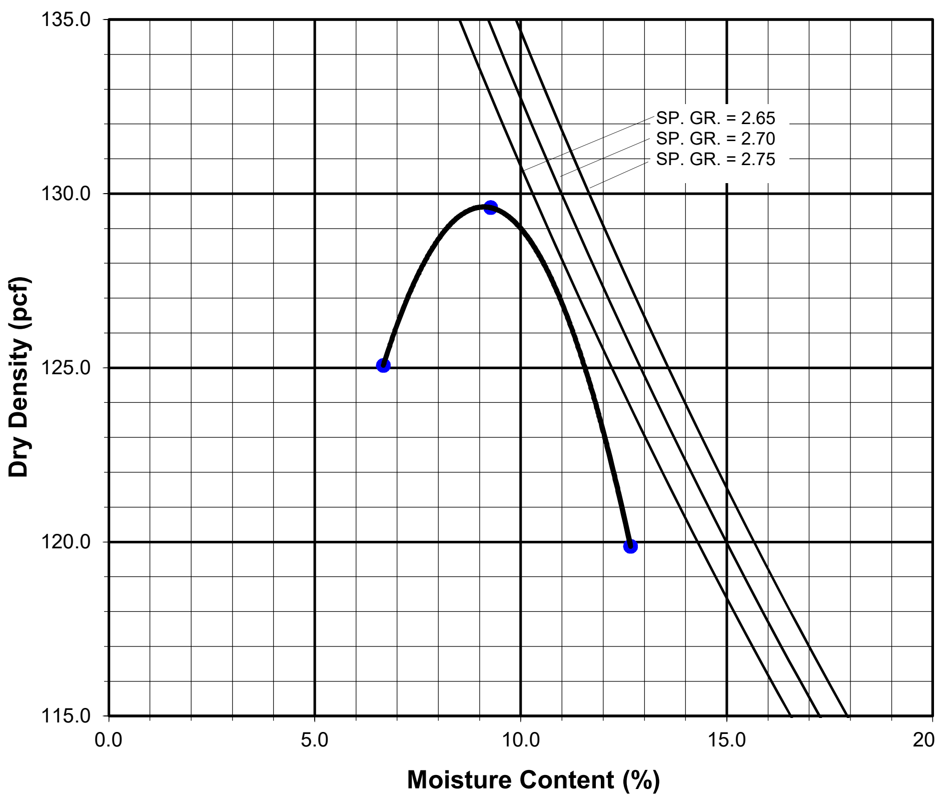
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Rexford Glassell	Tested By:	J. Gonzalez	Date:	11/08/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Yellowish brown clayey sand with gravel (SC)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	
#4	20.7

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) = 0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3774	3941	3951			
Weight of Mold (g)	1850	1850	1850			
Net Weight of Soil (g)	1924	2091	2101			
Wet Weight of Soil + Cont. (g)	537.4	495.6	446.3			
Dry Weight of Soil + Cont. (g)	519.2	467.0	413.3			
Weight of Container (g)	39.0	39.8	38.4			
Moisture Content (%)	3.79	6.69	8.80			
Wet Density (pcf)	127.4	138.4	139.1			
Dry Density (pcf)	122.7	129.7	127.8			

Maximum Dry Density (pcf)	**129.8**	**Optimum Moisture Content (%)**	**7.1**
Corrected Dry Density (pcf)	**136.3**	**Corrected Moisture Content (%)**	**5.8**

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

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

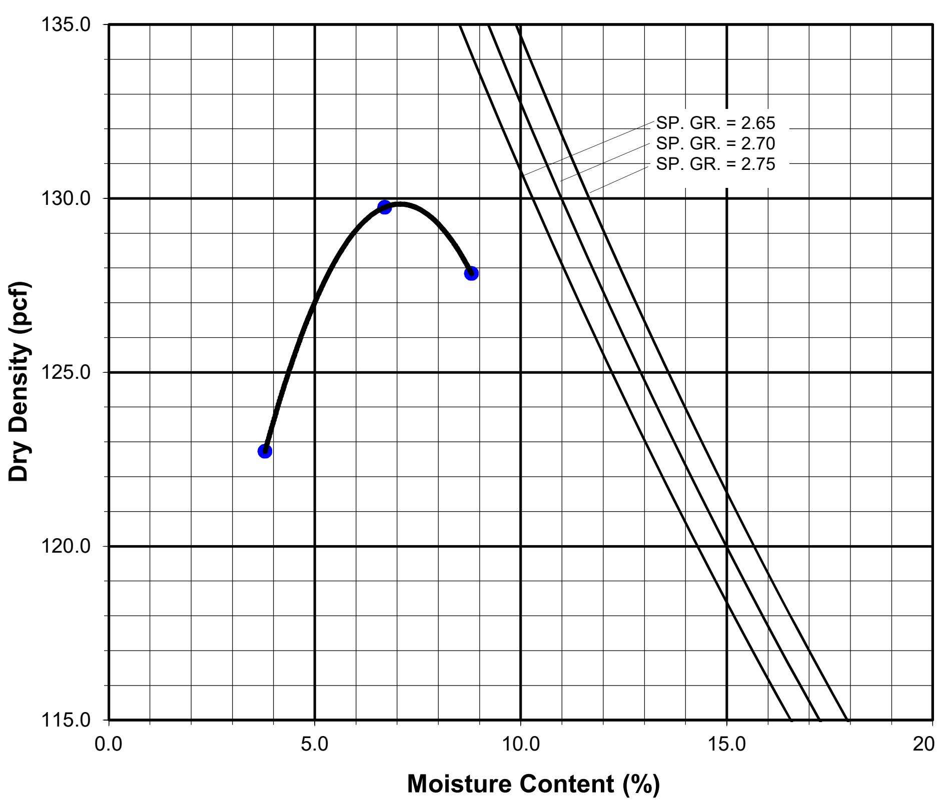
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Rexford Glassell Street	Tested By:	S. Felter	Date:	11/10/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Yellowish brown sandy silty clay s(CL-ML)				

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

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0235
Wt. Comp. Soil + Mold	(g)	605.90	448.60
Wt. of Mold	(g)	184.40	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	842.70	633.00
Dry Wt. of Soil + Cont.	(g)	780.30	574.68
Wt. of Container	(g)	0.00	184.40
Moisture Content	(%)	8.00	14.94
Wet Density	(pcf)	127.1	132.2
Dry Density	(pcf)	117.7	115.0
Void Ratio		0.432	0.466
Total Porosity		0.302	0.318
Pore Volume	(cc)	62.4	67.3
Degree of Saturation (%) [S meas]		**50.0**	86.6

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
11/10/21	8:51	1.0	0	0.5740
11/10/21	9:01	1.0	10	0.5735
Add Distilled Water to the Specimen				
11/10/21	12:25	1.0	204	0.5960
11/11/21	6:41	1.0	1300	0.5975
11/11/21	7:45	1.0	1364	0.5975

Expansion Index (EI meas) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**24**

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Rexford Glassell Street	Tested By:	S. Felter	Date:	11/11/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Yellowish brown clayey sand (SC)				

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	1.0260
Wt. Comp. Soil + Mold	(g)	607.80	450.50
Wt. of Mold	(g)	190.10	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	842.70	640.60
Dry Wt. of Soil + Cont.	(g)	780.30	576.86
Wt. of Container	(g)	0.00	190.10
Moisture Content	(%)	8.00	16.48
Wet Density	(pcf)	126.0	132.4
Dry Density	(pcf)	116.7	113.7
Void Ratio		0.445	0.483
Total Porosity		0.308	0.326
Pore Volume	(cc)	63.7	69.1
Degree of Saturation (%) [S_{meas}]		**48.5**	92.2

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.)
11/11/21	8:40	1.0	0	0.5880
11/11/21	8:50	1.0	10	0.5880
Add Distilled Water to the Specimen				
11/11/21	10:35	1.0	105	0.6115
11/12/21	6:50	1.0	1320	0.6140
11/12/21	7:50	1.0	1380	0.6140

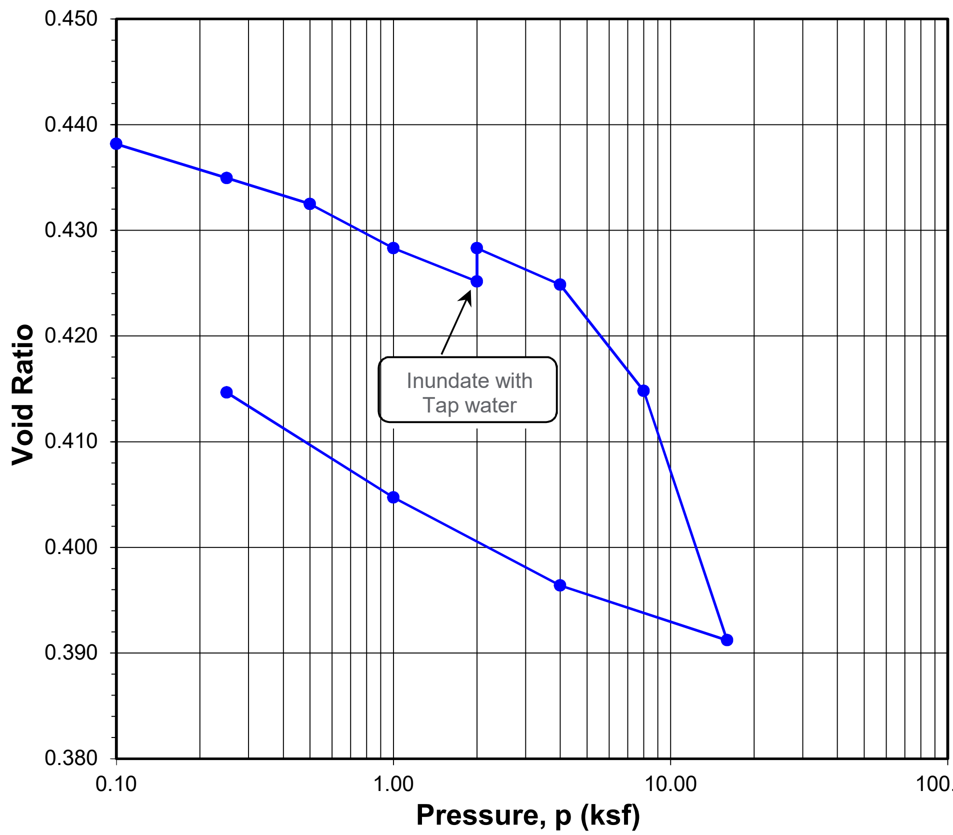
Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**26**

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford Glassell Street	Tested By:	GB/YN	Date:	11/10/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1	Sample Type:	90% Remold		
Soil Identification:	Yellowish brown sandy silty clay s(CL-ML)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	198.00
Weight of Ring (g)	44.25
Height after consol. (in.)	0.9833
Before Test	
Wt.Wet Sample+Cont. (g)	158.54
Wt.of Dry Sample+Cont. (g)	148.33
Weight of Container (g)	36.57
Initial Moisture Content (%)	9.1
Initial Dry Density (pcf)	117.2
Initial Saturation (%)	56
Initial Vertical Reading (in.)	0.2815
After Test	
Wt.of Wet Sample+Cont. (g)	263.08
Wt. of Dry Sample+Cont. (g)	244.02
Weight of Container (g)	58.97
Final Moisture Content (%)	13.54
Final Dry Density (pcf)	119.1
Final Saturation (%)	88
Final Vertical Reading (in.)	0.2611
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

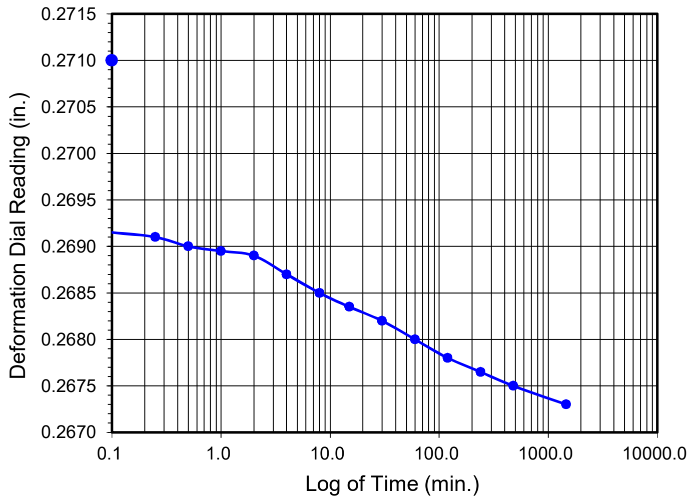


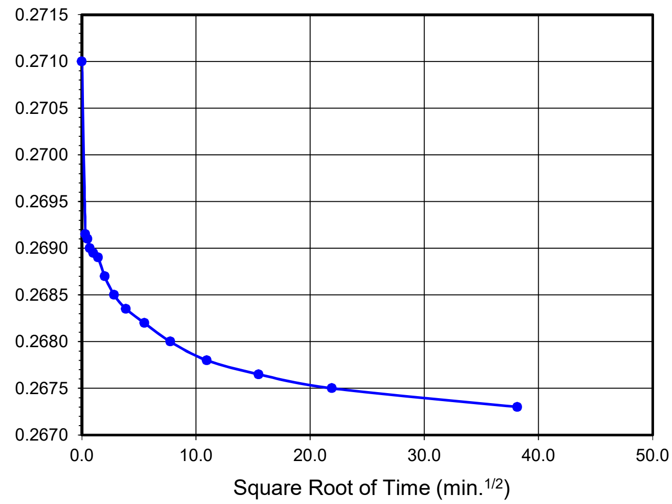
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2812	0.9997	0.00	0.03	0.438	0.03
0.25	0.2782	0.9967	0.07	0.33	0.435	0.26
0.50	0.2759	0.9944	0.13	0.56	0.432	0.43
1.00	0.2722	0.9907	0.21	0.93	0.428	0.72
2.00	0.2688	0.9873	0.33	1.27	0.425	0.94
2.00	0.2710	0.9895	0.33	1.05	0.428	0.72
4.00	0.2673	0.9858	0.46	1.42	0.425	0.96
8.00	0.2585	0.9770	0.64	2.30	0.415	1.66
16.00	0.2399	0.9584	0.86	4.16	0.391	3.30
4.00	0.2453	0.9638	0.68	3.62	0.396	2.94
1.00	0.2529	0.9714	0.50	2.86	0.405	2.36
0.25	0.2611	0.9796	0.37	2.04	0.415	1.67

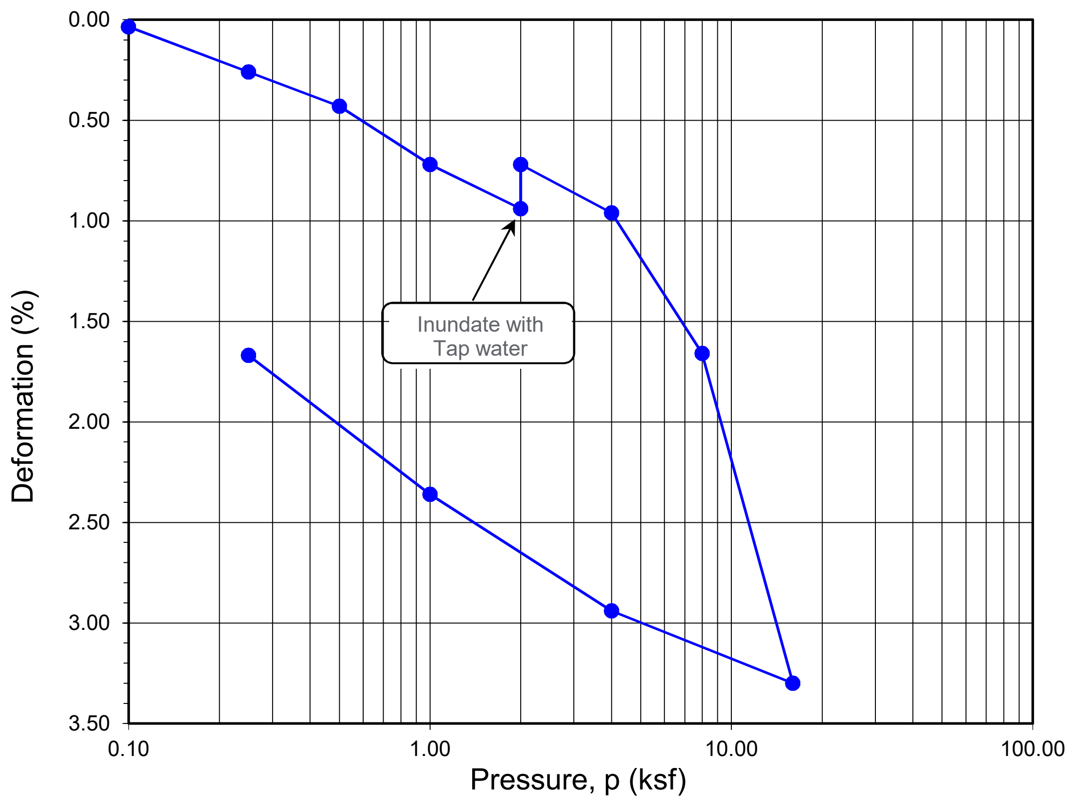
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
11/15/21	8:20:00	0.0	0.0	0.2710
11/15/21	8:20:06	0.1	0.3	0.2692
11/15/21	8:20:15	0.2	0.5	0.2691
11/15/21	8:20:30	0.5	0.7	0.2690
11/15/21	8:21:00	1.0	1.0	0.2690
11/15/21	8:22:00	2.0	1.4	0.2689
11/15/21	8:24:00	4.0	2.0	0.2687
11/15/21	8:28:00	8.0	2.8	0.2685
11/15/21	8:35:00	15.0	3.9	0.2684
11/15/21	8:50:00	30.0	5.5	0.2682
11/15/21	9:20:00	60.0	7.7	0.2680
11/15/21	10:20:00	120.0	11.0	0.2678
11/15/21	12:20:00	240.0	15.5	0.2677
11/15/21	16:20:00	480.0	21.9	0.2675
11/16/21	8:36:00	1456.0	38.2	0.2673

Time Readings @ 4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**B-1**	**0-5**	**9.1**	**13.5**	**117.2**	**119.1**	**0.439**	**0.415**	**56**	**88**

Soil Identification: Yellowish brown sandy silty clay s(CL-ML)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13333.001

Rexford Glassell Street

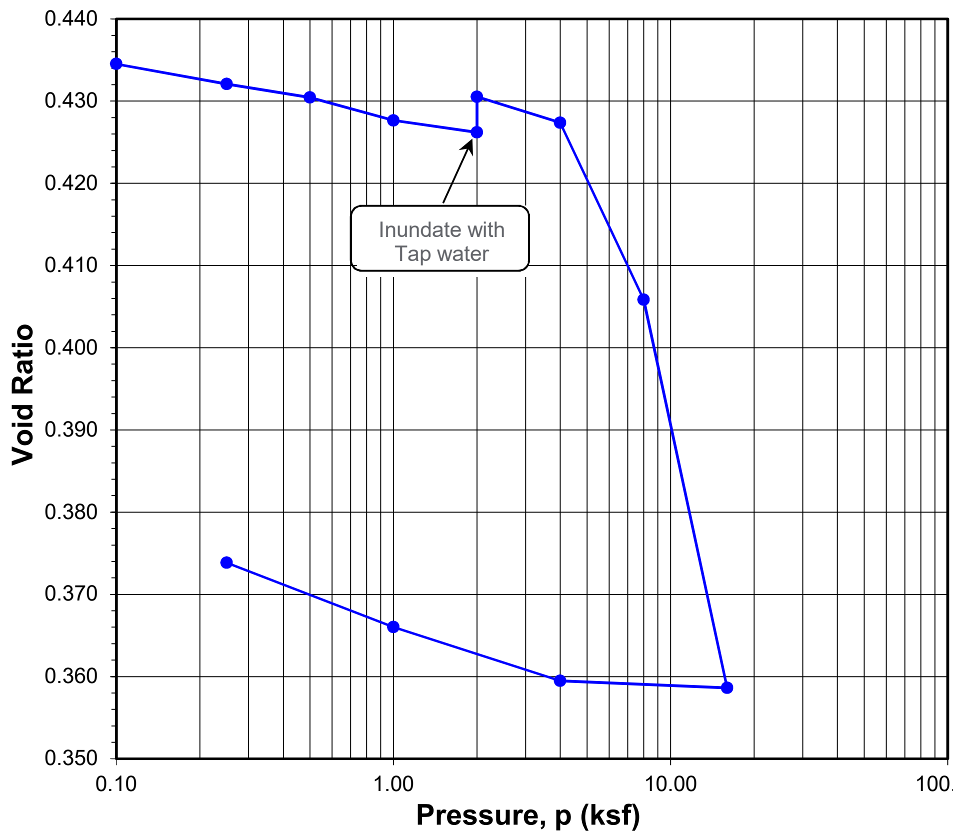
12-21

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford Glassell Street	Tested By:	GB/YN	Date:	11/10/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-1	Sample Type:	90% Remold		
Soil Identification:	Yellowish brown clayey sand (SC)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	194.07
Weight of Ring (g)	42.87
Height after consol. (in.)	0.9575
Before Test	
Wt.Wet Sample+Cont. (g)	169.50
Wt.of Dry Sample+Cont. (g)	162.36
Weight of Container (g)	60.90
Initial Moisture Content (%)	7.0
Initial Dry Density (pcf)	117.5
Initial Saturation (%)	44
Initial Vertical Reading (in.)	0.3143
After Test	
Wt.of Wet Sample+Cont. (g)	266.28
Wt. of Dry Sample+Cont. (g)	248.38
Weight of Container (g)	64.60
Final Moisture Content (%)	12.70
Final Dry Density (pcf)	122.4
Final Saturation (%)	91
Final Vertical Reading (in.)	0.2657
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

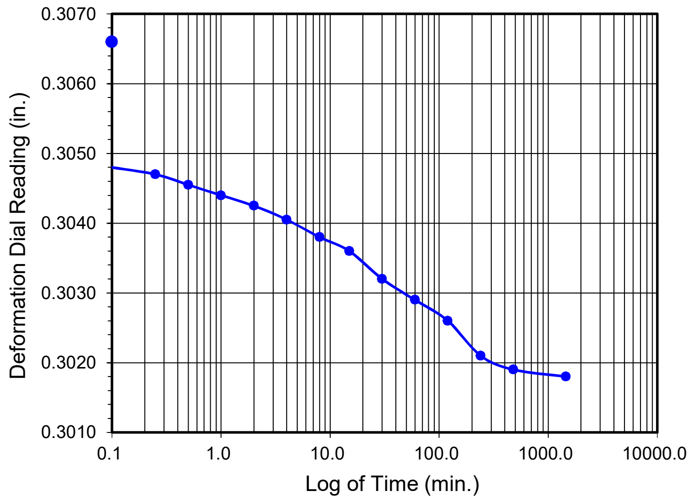


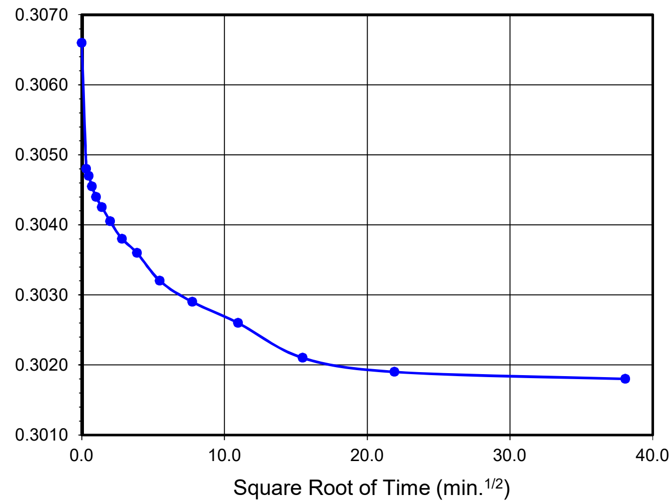
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3141	0.9998	0.00	0.02	0.435	0.02
0.25	0.3117	0.9974	0.07	0.26	0.432	0.19
0.50	0.3097	0.9954	0.16	0.47	0.430	0.31
1.00	0.3066	0.9923	0.27	0.77	0.428	0.50
2.00	0.3036	0.9893	0.47	1.07	0.426	0.60
2.00	0.3066	0.9923	0.47	0.77	0.431	0.30
4.00	0.3018	0.9875	0.73	1.25	0.427	0.52
8.00	0.2841	0.9698	1.00	3.02	0.406	2.02
16.00	0.2477	0.9334	1.35	6.66	0.359	5.31
4.00	0.2517	0.9374	1.01	6.26	0.359	5.25
1.00	0.2588	0.9445	0.76	5.56	0.366	4.80
0.25	0.2657	0.9514	0.61	4.86	0.374	4.25

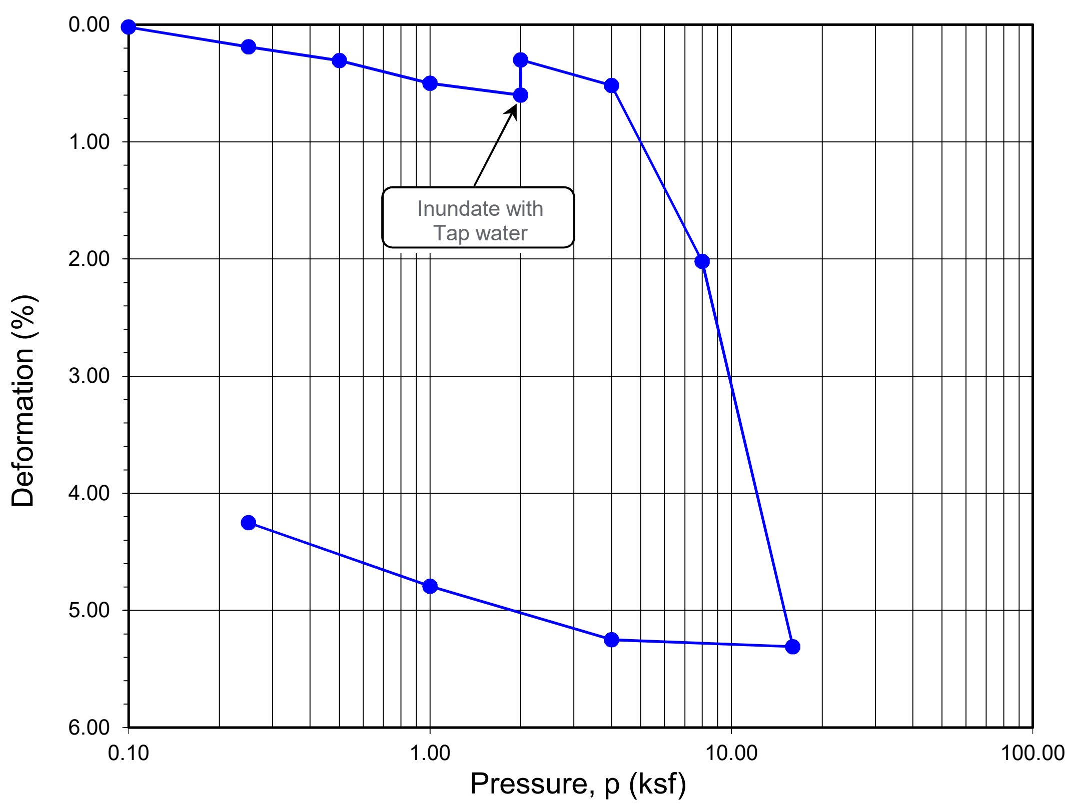
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
11/15/21	8:25:00	0.0	0.0	0.3066
11/15/21	8:25:06	0.1	0.3	0.3048
11/15/21	8:25:15	0.2	0.5	0.3047
11/15/21	8:25:30	0.5	0.7	0.3046
11/15/21	8:26:00	1.0	1.0	0.3044
11/15/21	8:27:00	2.0	1.4	0.3043
11/15/21	8:29:00	4.0	2.0	0.3041
11/15/21	8:33:00	8.0	2.8	0.3038
11/15/21	8:40:00	15.0	3.9	0.3036
11/15/21	8:55:00	30.0	5.5	0.3032
11/15/21	9:25:00	60.0	7.7	0.3029
11/15/21	10:25:00	120.0	11.0	0.3026
11/15/21	12:25:00	240.0	15.5	0.3021
11/15/21	16:25:00	480.0	21.9	0.3019
11/16/21	8:35:00	1450.0	38.1	0.3018

Time Readings @ 4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-4	**B-1**	**0-5**	**7.0**	**12.7**	**117.5**	**122.4**	**0.435**	**0.374**	**44**	**91**

Soil Identification: Yellowish brown clayey sand (SC)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13333.001

Rexford Glassell Street

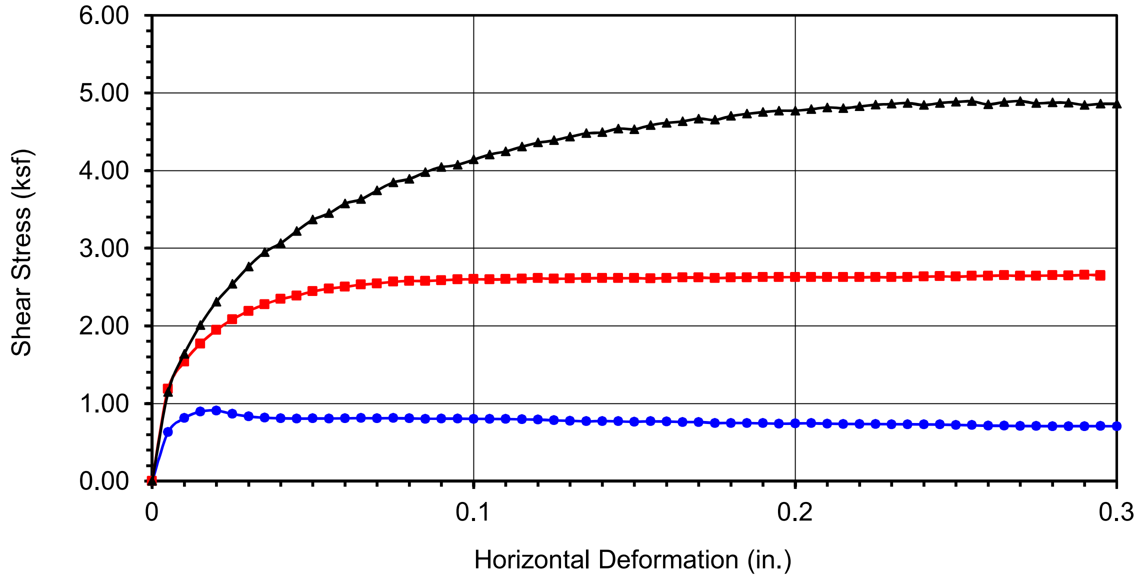
12-21

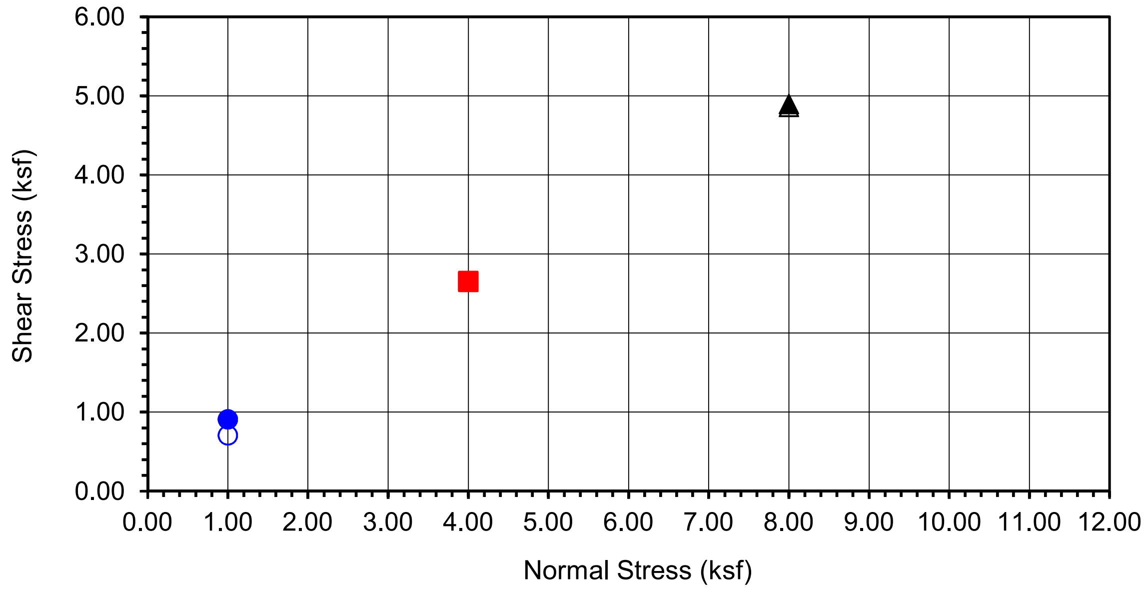
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Rexford Glassell Street	Tested By:	G. Bathala	Date:	11/18/21
Project No.:	13333.001	Checked By:	A. Santos	Date:	12/12/21
Boring No.:	LB-1	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		
Soil Identification:	Yellowish brown sandy silty clay s(CL-ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	199.03	198.93	198.80
Weight of Ring(gm):	45.58	45.35	44.88
Before Shearing			
Weight of Wet Sample+Cont.(gm):	158.54	158.54	158.54
Weight of Dry Sample+Cont.(gm):	148.33	148.33	148.33
Weight of Container(gm):	36.57	36.57	36.57
Vertical Rdg.(in): Initial	0.0000	0.2397	0.2420
Vertical Rdg.(in): Final	-0.0024	0.2583	0.2666
After Shearing			
Weight of Wet Sample+Cont.(gm):	211.45	219.85	214.57
Weight of Dry Sample+Cont.(gm):	190.86	201.05	197.00
Weight of Container(gm):	51.14	61.75	58.97
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type: 90% Remold	
Soil Identification: Yellowish brown sandy silty clay s(CL-ML)	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.909	■ 2.656	▲ 4.895
Shear Stress @ End of Test (ksf)	○ 0.707	□ 2.644	△ 4.860
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 (%)	9.14	9.14	9.14
Dry Density (pcf)	116.9	117.0	117.3
Saturation (%)	55.9	56.0	56.4
Soil Height Before Shearing (in.)	0.9976	0.9814	0.9754
Final Moisture Content (%)	14.7	13.5	12.7

Leighton

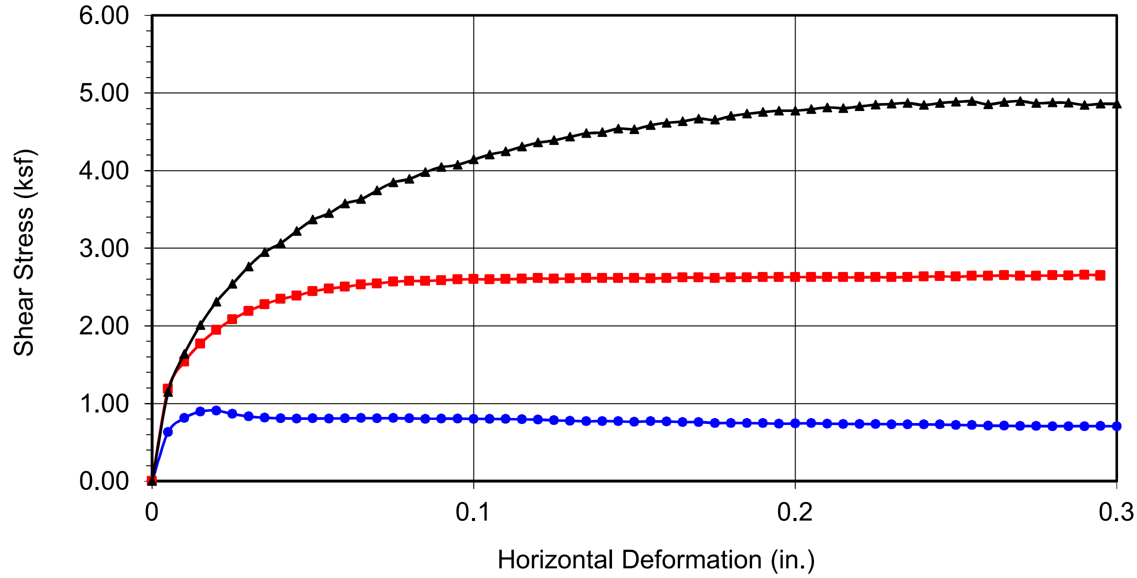
DIRECT SHEAR TEST RESULTS

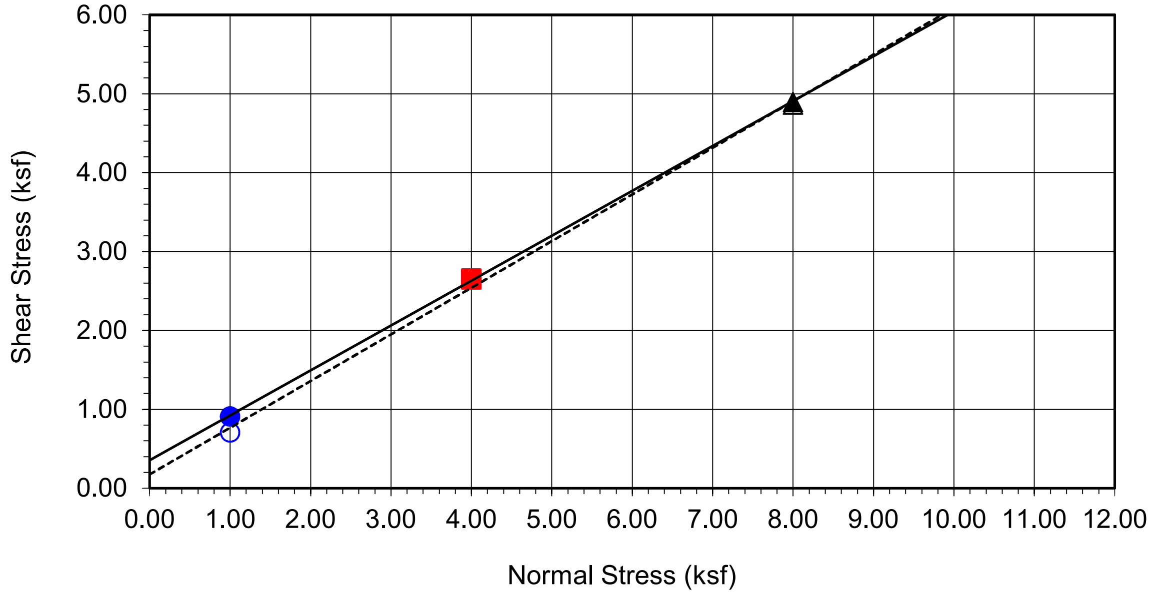
Consolidated Drained - ASTM D 3080

Project No.: 13333.001

Rexford Glassell Street

11-21





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold
Soil Identification: Yellowish brown sandy silty clay s(CL-ML)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	355	30
Ultimate	175	31

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.909	■ 2.656	▲ 4.895
Shear Stress @ End of Test (ksf)	○ 0.707	□ 2.644	△ 4.860
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 (%)	9.14	9.14	9.14
Dry Density (pcf)	116.9	117.0	117.3
Saturation (%)	55.9	56.0	56.4
Soil Height Before Shearing (in.)	0.9976	0.9814	0.9754
Final Moisture Content (%)	14.7	13.5	12.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13333.001

Rexford Glassell Street

11-21

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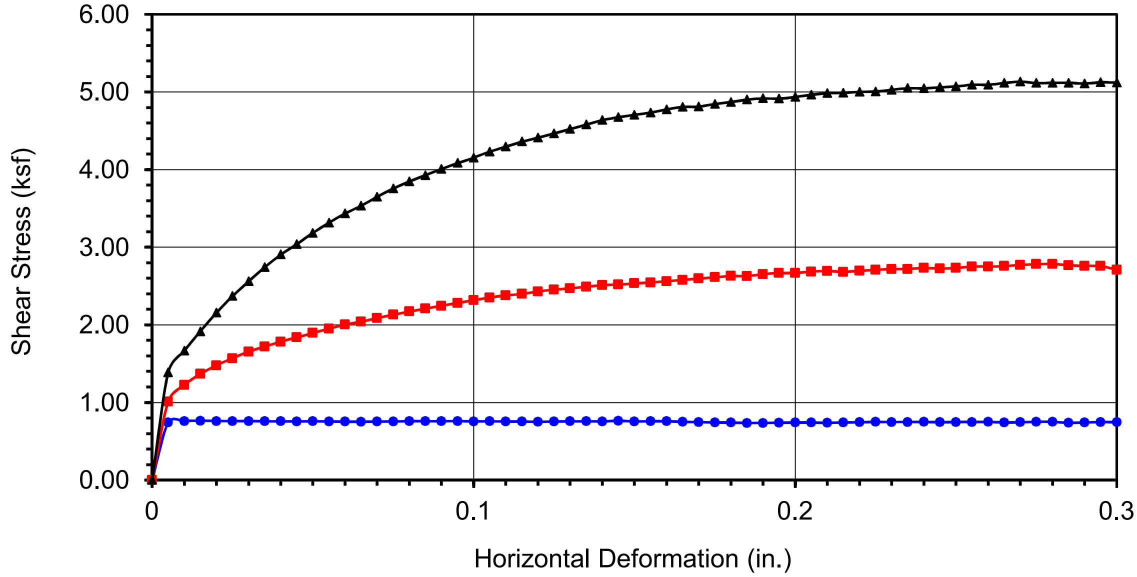
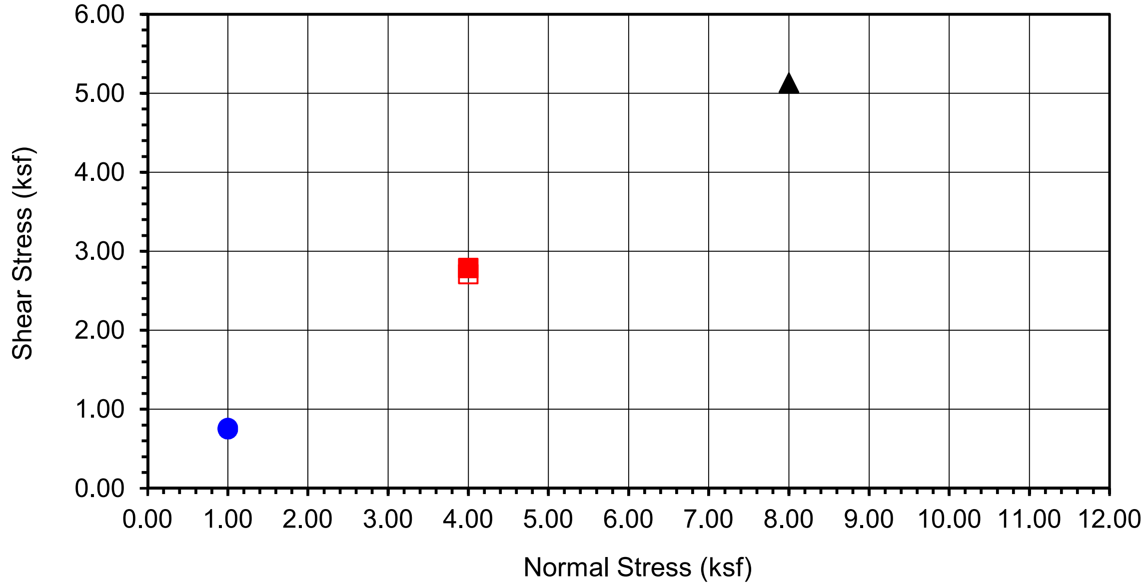
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: Rexford Glassell Street
Tested By: G. Bathala
Date: 11/18/21
Project No.: 13333.001
Checked By: A. Santos
Date: 12/06/21
Boring No.: LB-4
Sample Type: 90% Remold
Sample No.: B-1
Depth (ft.): 0-5
Soil Identification: Yellowish brown clayey sand (SC)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	195.53	196.47	196.70
Weight of Ring(gm):	44.88	45.59	45.34
Before Shearing			
Weight of Wet Sample+Cont.(gm):	169.50	169.50	169.50
Weight of Dry Sample+Cont.(gm):	162.36	162.36	162.36
Weight of Container(gm):	60.90	60.90	60.90
Vertical Rdg.(in): Initial	0.2739	0.2689	0.0000
Vertical Rdg.(in): Final	0.2708	0.2841	-0.0320
After Shearing			
Weight of Wet Sample+Cont.(gm):	218.37	194.86	215.13
Weight of Dry Sample+Cont.(gm):	198.00	177.02	198.75
Weight of Container(gm):	59.15	38.30	59.16
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:
Yellowish brown clayey sand (SC)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.764	■ 2.785	▲ 5.134
Shear Stress @ End of Test (ksf)	○ 0.745	□ 2.710	△ 5.121
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	7.04	7.04	7.04
Dry Density (pcf)	117.1	117.2	117.6
Saturation (%)	43.2	43.4	43.9
Soil Height Before Shearing (in.)	1.0031	0.9848	0.9680
Final Moisture Content (%)	14.7	12.9	11.7

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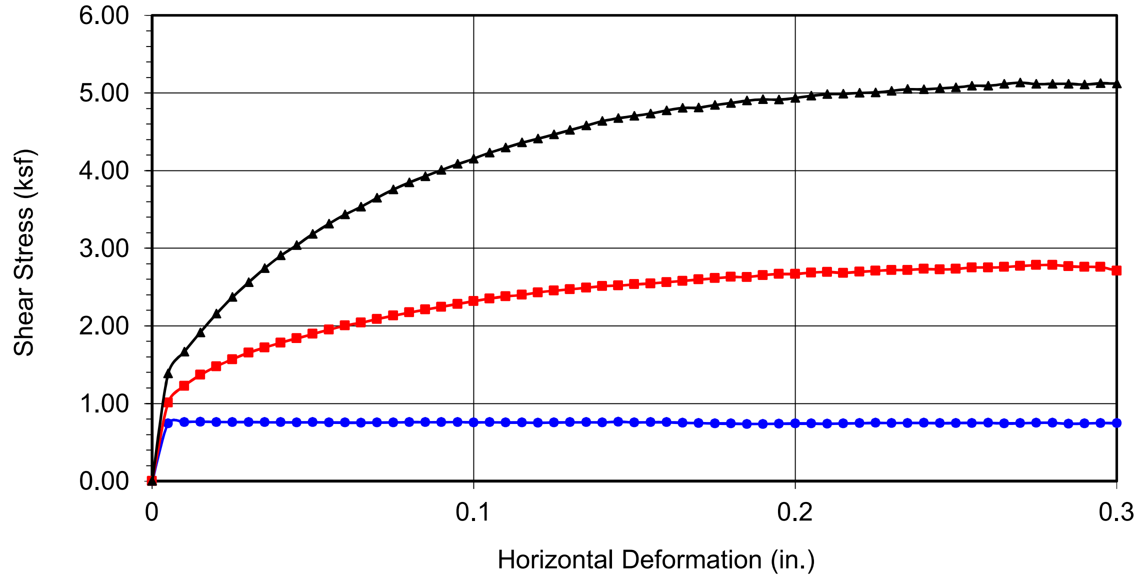
DIRECT SHEAR TEST RESULTS

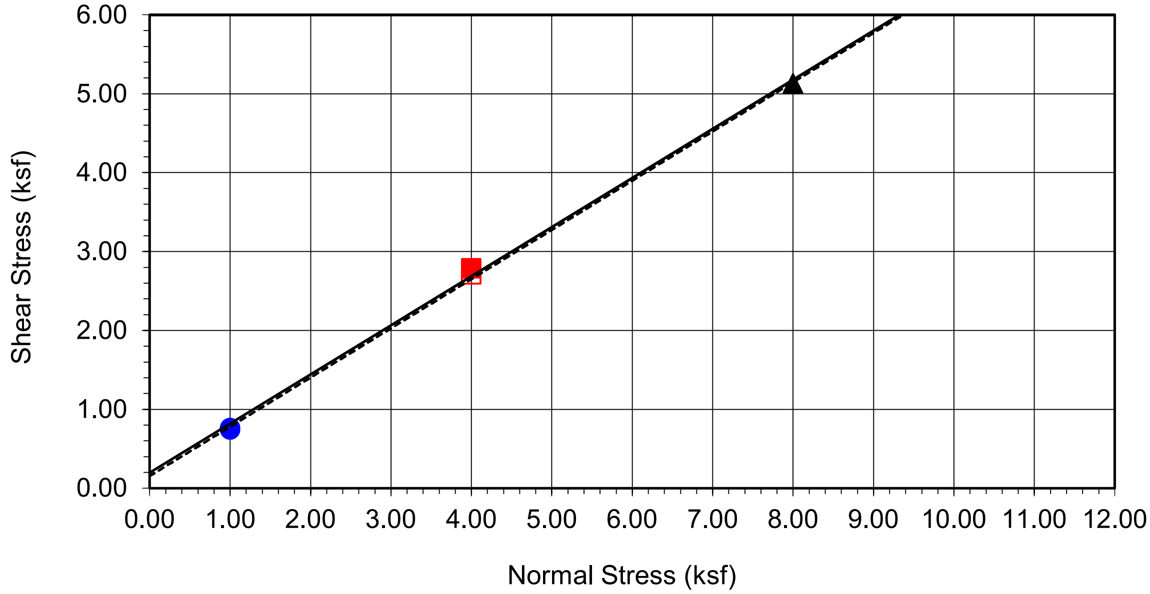
Consolidated Drained - ASTM D 3080

Project No.: 13333.001

Rexford Glassell Street

11-21





Boring No.	**LB-4**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold
Soil Identification: Yellowish brown clayey sand (SC)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	198	32
Ultimate	155	32

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.764	■ 2.785	▲ 5.134
Shear Stress @ End of Test (ksf)	○ 0.745	□ 2.710	△ 5.121
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	7.04	7.04	7.04
Dry Density (pcf)	117.1	117.2	117.6
Saturation (%)	43.2	43.4	43.9
Soil Height Before Shearing (in.)	1.0031	0.9848	0.9680
Final Moisture Content (%)	14.7	12.9	11.7

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13333.001

Rexford Glassell Street

11-21

R-VALUE TEST RESULTS

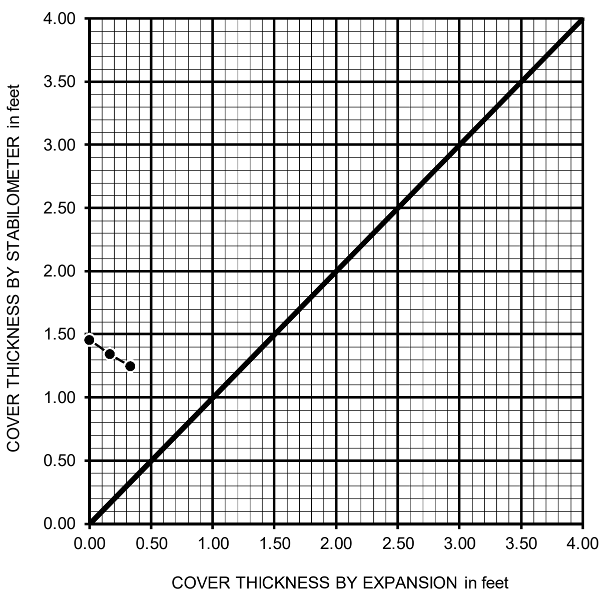
DOT CA Test 301

PROJECT NAME:	Rexford Glassell Street	PROJECT NUMBER:	13333.001
BORING NUMBER:	LB-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Yellowish brown sandy silty clay s(CL-ML)	DATE COMPLETED:	11/18/2021

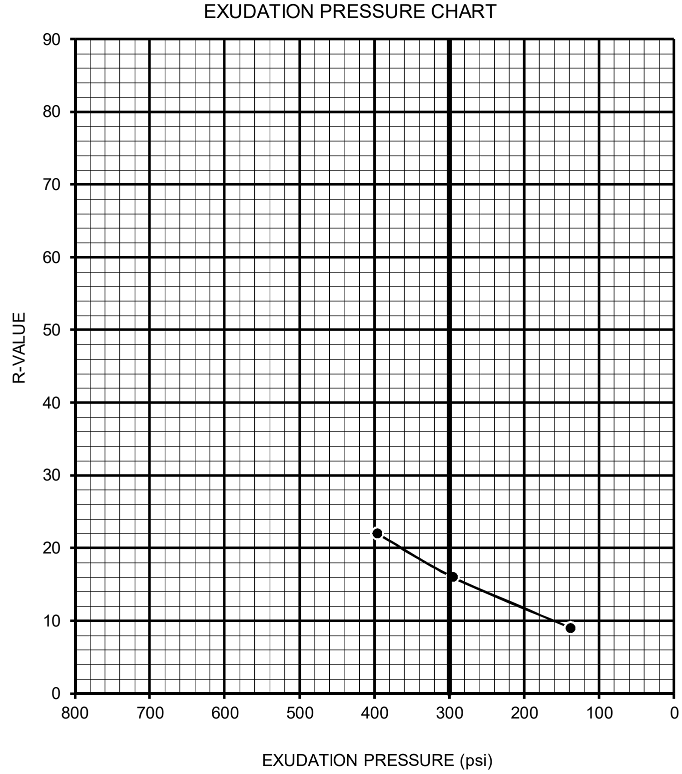
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	12.0	12.9	14.3
HEIGHT OF SAMPLE, Inches	2.47	2.51	2.55
DRY DENSITY, pcf	124.5	123.9	119.7
COMPACTOR PRESSURE, psi	110	70	65
EXUDATION PRESSURE, psi	396	295	138
EXPANSION, Inches x 10exp-4	10	5	0
STABILITY Ph 2,000 lbs (160 psi)	108	120	136
TURNS DISPLACEMENT	4.32	4.45	4.52
R-VALUE UNCORRECTED	22	16	9
R-VALUE CORRECTED	22	16	9

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.	1.25	1.34	1.46
EXPANSION PRESSURE THICKNESS, ft.	0.33	0.17	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	34
R-VALUE BY EXUDATION:	16
EQUILIBRIUM R-VALUE:	16

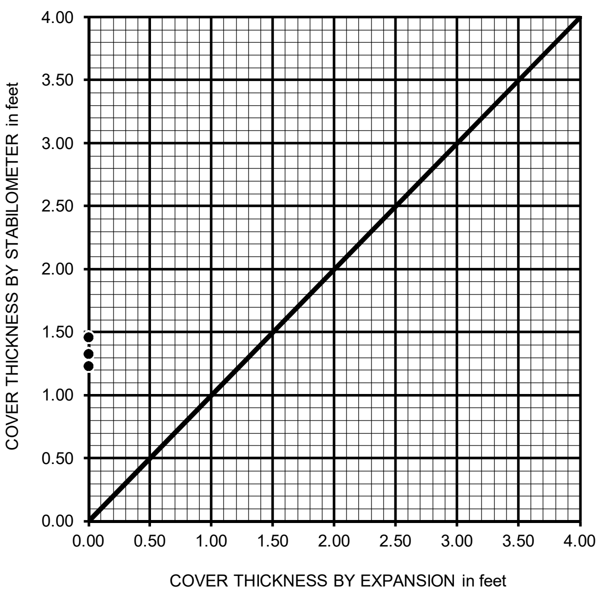
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Rexford Glassell Street	PROJECT NUMBER:	13333.001
BORING NUMBER:	LB-4	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Yellowish brn clayey sand with gravel (SC)g	DATE COMPLETED:	11/18/2021

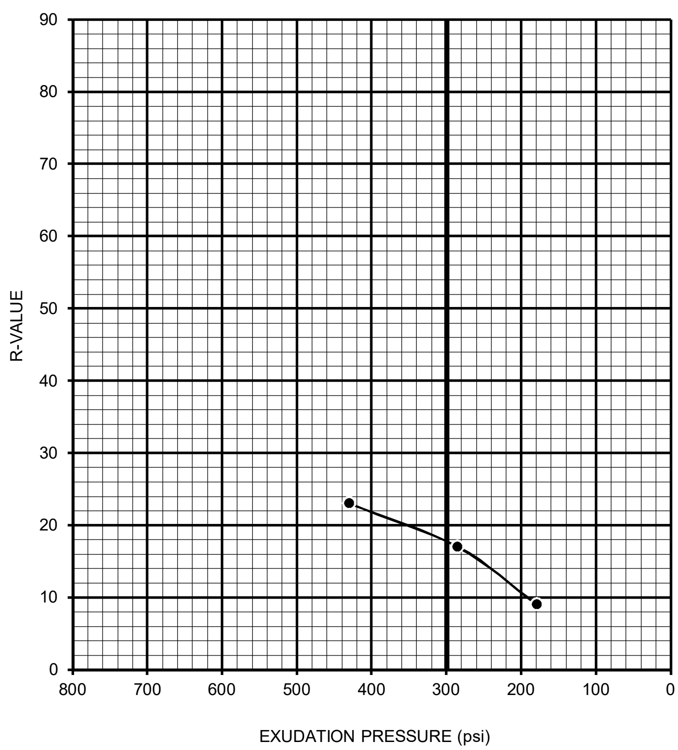
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.6	11.5	12.7
HEIGHT OF SAMPLE, Inches	2.49	2.53	2.50
DRY DENSITY, pcf	128.9	127.2	123.6
COMPACTOR PRESSURE, psi	100	70	50
EXUDATION PRESSURE, psi	430	285	179
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	110	120	136
TURNS DISPLACEMENT	3.80	4.20	4.32
R-VALUE UNCORRECTED	23	17	9
R-VALUE CORRECTED	23	17	9

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.	1.23	1.33	1.46
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	18
EQUILIBRIUM R-VALUE:	18

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Rexford Glassell Street | Tested By : JD/GB | Date: 11/10/21

Project No. : 13333.001 | Checked By: A. Santos | Date: 12/12/21

Boring No.	LB-1	LB-4		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Yellowish brown s(CL-ML)	Yellowish brown (SC)		
Wet Weight of Soil + Container (g)	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00		
Weight of Container (g)	1.00	1.00		
Moisture Content (%)	0.00	0.00		
Weight of Soaked Soil (g)	100.47	100.67		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	95	200A		
Crucible No.	16	14		
Furnace Temperature (°C)	860	860		
Time In / Time Out	7:00/7:45	6:35/7:20		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	18.4737	19.6848		
Wt. of Crucible (g)	18.4713	19.6840		
Wt. of Residue (g) (A)	0.0024	0.0008		
PPM of Sulfate (A) x 41150	98.76	32.92		
PPM of Sulfate, Dry Weight Basis	**99**	**33**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	15		
ml of AgNO3 Soln. Used in Titration (C)	0.6	0.4		
PPM of Chloride (C -0.2) * 100 * 30 / B	40	40		
PPM of Chloride, Dry Wt. Basis	**40**	**40**		

pH TEST, DOT California Test 643

pH Value	**8.49**	**8.18**		
Temperature °C	21.4	22.3		

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Rexford Glassell Street	Tested By :	A. Santos Date: 11/22/21
Project No. :	13333.001	Checked By:	G. Bathala Date: 12/07/21
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	B-1		

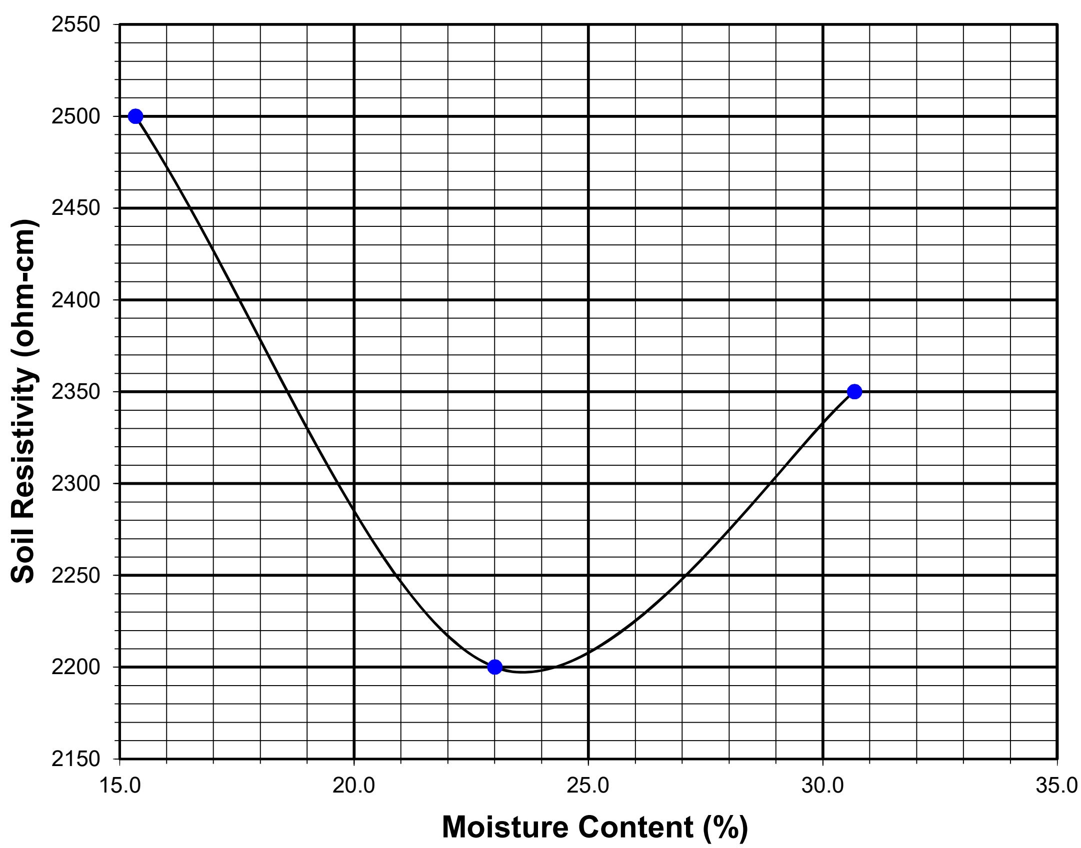
Soil Identification:* Yellowish brown s(CL-ML)

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.34	2500	2500
2	30	23.01	2200	2200
3	40	30.67	2350	2350
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.40
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2197	**23.6**	**99**	**40**	**8.49**	**21.4**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Rexford Glassell Street	Tested By :	A. Santos Date: 11/22/21
Project No. :	13333.001	Checked By:	G. Bathala Date: 12/12/21
Boring No.:	LB-4	Depth (ft.) :	0-5
Sample No. :	B-1		

Soil Identification:* Yellowish brown (SC)

*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	15.30	2700	2700
2	30	22.95	2500	2500
3	40	30.60	2800	2800
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.70
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2498	**22.5**	**33**	**40**	**8.18**	**22.3**