GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
13925 BENSON AVENUE
CHINO, CALIFORNIA

Prepared For

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Project No. 13807.001

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Rexford Industrial Realty & Management, Inc.
11620 Wilshire Boulevard, Suite 1000
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Attention: Mr. Corey Guerrero

Subject: Geotechnical Exploration Report
Proposed Industrial Building
13925 Benson Avenue
Chino, California

In response to your request and authorization, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. Based on review of the *Conceptual Site Plan* (A1-0), dated July 19, 2022, we understand the proposed development will include a new one-story industrial building constructed at-grade with a total building area of 150,000 square feet. The proposed building is planned to include dock-high truck loading on the southern side of the building with associated surface parking on the west, south, and east sides of the building. San Bernardino County Fire Department access is also planned around the entire building. Ancillary improvements will likely consist of utility infrastructure, pavement, flatwork, and landscaping.

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

Based on the results of our study, the project is considered feasible from a geotechnical standpoint. This report presents the results of our exploration, conclusions and geotechnical design recommendations for the proposed development.

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

<u>Tables</u>

<u>Attachments</u>

1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 13925 Benson Avenue in the city of Chino, San Bernardino County, California. The site location (latitude 34.0003°, longitude -117.6792°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

The project site is rectangular in shape and covers approximately 6.6 acres. The site is bordered by Benson Avenue to the west, existing industrial properties to the north and east, and a Southern California Edison (SCE) substation to the south. Access to the site is via Benson Avenue from the west. The site is currently occupied by one (1) existing industrial building with several mobile trailers, storage containers, various equipment storage areas, and associated paved surface parking and access.

The project site is relatively flat with sheet flow generally directed to the south over paved surfaces to curbs and gutters. Review of the United States Geological Survey (USGS) 7.5-Minute Ontario Quadrangle (USGS, 1981) indicates the ground surface at the site is between approximately Elevation (El.) +680 to +690 feet mean sea level (msl).

Based on review of historic aerial photographs (NETR, 2023), the site was vacant, undeveloped land until at least 1966. Between approximately 1966 and 1972, the existing building located in the central portion of the site was constructed to its current configuration.

Based on *Conceptual Site Plan* (Sheet A1-0), dated July 19, 2022, and discussions with you, we understand the proposed development concept for the site consists of a new one-story industrial building constructed at-grade with a total building area of 150,000 square feet. The proposed building is planned to include dock-high truck loading on the southern side of the building with associated surface parking on the west, south, and east sides of the building. San Bernardino County Fire Department access is also planned around the entire building. Ancillary improvements will likely consist of utility infrastructure, pavement, flatwork, and landscaping. Preliminary structural loading information was not yet available at the time this report was prepared.

1.2 Purpose and Scope

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

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

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

- *Field Exploration* – Our subsurface exploration was performed on February 7, 2023 and included drilling, logging, and sampling of five (5) hollow-stem auger borings (designated LB-1 through LB-5) to approximate depths between 31½ and 51½ feet below the existing ground surface (bgs). Two (2) additional borings (designated LP-1 and LP-2) were drilled to an approximate depth of 10 feet bgs, respectively, 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 hollow-stem auger borings, bulk and drive samples were obtained from the borings 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 2½ and 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, *Exploration Logs*.

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. After completion of drilling, the borings were backfilled with soil cuttings and patched with cold-mix asphalt concrete at the surface.

- *Percolation Testing* – Borings LP-1 and LP-2 were converted to temporary percolation test wells upon completion of drilling and sampling. The test wells consisted of 2-inch slotted (0.020”) PVC well casing surrounded by #3 Monterey Sand placed in the annulus of the well within the test zone. In-situ percolation testing was performed on February 8, 2023 in general accordance with *San Bernardino County Technical Guidance Document (TGD) for Water Quality Management Plans* (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 soil. Tests performed during this investigation include:

  - In- situ Moisture Content and Dry Density (ASTM D2216 and ASTM D2937);
  - Maximum Dry Density (ASTM D 1557);
  - Expansion Index (ASTM D 4829);
  - Atterberg Limits (ASTM D 4318);
  - Direct Shear (ASTM D 3080);
  - Consolidation (ASTM D 2435);
  - 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 in the southwestern portion of the Chino Basin in the northern portion of the Peninsular Ranges geomorphic province of California. Major structural features surround this region, including the Cucamonga fault and the San Gabriel Mountains to the north, the Chino fault and Puente/Chino Hills to the west, and the San Jacinto fault to the east. This is an area of large-scale crustal disturbance as the relatively northwestward moving Peninsular Ranges province collides with the Transverse Ranges province (San Gabriel Mountains) to the north. Several active and potentially active faults have been mapped in the region and are believed to accommodate compression associated with this collision. The Sierra Madre fault, a major structural feature along the southern flank of the San Gabriel Mountains, is located approximately 9.5 miles northwest of the site. This fault, as well as other faults in the region, has the potential for generating strong ground motions at the project site. Further discussion of faulting relative to the site is provided in Section 3.1, Faulting and Seismicity, of this report.

The site is located in an area underlain by thick accumulations of alluvial sand, silt, gravel, cobbles, and boulders eroded from the mountains and deposited in the site vicinity by the Santa Ana River and smaller tributaries such as the San Antonio Creek and Cucamonga Creek (Morton and Miller, 2006; Dibblee, 2002).

2.2 Surficial Geology

The Quaternary-age deposits that cover the floor and margins of the Chino Basin at the surface are mapped to be composed primarily of recent (middle Holocene) young alluvial fan deposits. These young sediments consist predominately of slightly to moderately consolidated silt, sand and coarse-grained sand to boulder alluvial deposits derived from the surrounding mountains and hills with finer clays and silts deposited into the basin over the broad floodplain. The surficial geologic units mapped in the vicinity of the project site are shown on Figure 3, *Regional Geology Map*.

2.3 Subsurface Soil Conditions

Based on our subsurface explorations, the site is underlain by a layer of undocumented artificial fill materials (Afu) overlying Quaternary-aged (middle Holocene) young alluvial fan deposits (Qyf). A layer of undocumented artificial fill

(Afu) on the order of approximately 2 to 3 feet in thickness was encountered at the explored locations overlying natural alluvial soils. The fill materials encountered generally consist of dark brown silty sand and are likely associated with the existing and previous site improvements. Localized thicker accumulations of the fill materials should be anticipated between explored locations during future earthwork construction, particularly below the existing structures. Since there is no documentation for the placement, compaction and testing of the existing fill onsite, the artificial fill is considered undocumented and unsuitable for structural support in its current condition.

Below the artificial fill materials, Quaternary-aged young alluvial fan deposits (Qyf) were encountered in the borings to the maximum depth explored (51½ feet bgs). The alluvial materials encountered generally consist of light gray to grayish brown, moist, soft to hard clay, silt, and silty clay to light gray to yellowish brown, moist, medium dense to very dense silty sand and sand with variable amounts of gravel.

Detailed descriptions of the subsurface soils 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.

2.3.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result.

One (1) near-surface bulk soil sample obtained during our subsurface exploration was tested for expansion potential. The test results indicate an Expansion Index (EI) value of 6 (“very low” potential for expansion). The Expansion Index laboratory test results are included in Appendix C of this report.

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

2.3.2 Soil Corrosivity

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

The test results indicate a soluble sulfate concentration of 156 parts per million (ppm), chloride content of 40 ppm, pH value of 7.80, and minimum resistivity value of 5,600 ohm-cm.

The results of the resistivity tests indicate the underlying soil is mildly corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil samples, concrete in contact with the soil is expected to have moderate exposure to sulfate attack (S1) per ACI 318 (ACI, 2014). The samples tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. Concrete below grade is assigned a chloride exposure class C1 due to its likely exposure to moisture.

2.3.3 Soil Compressibility

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

2.3.4 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing on three (3) samples of the onsite soils recovered from the borings. 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 alluvial materials can generally be excavated using conventional excavation equipment in good operating condition.

The soils within the planned excavation depths are variable, and locally consist of layers that contain granular, unconsolidated soils with little or no cementation and few fines. These materials are prone to cave in or collapse in unshored excavations. See Section 3.7, *Temporary Excavations* for additional information on soil type and excavation characteristics.

2.4 Groundwater Conditions

Groundwater was not encountered during our subsurface exploration performed at the site to the maximum depth of 51½ feet bgs. Based on review of information available from the Chino Basin Watermaster (2016), groundwater depth at the site is anticipated to be greater than 100 feet bgs. In addition, based on review of available information from the California Department of Water Resources (DWR, 2023) for a nearby groundwater monitoring well located approximately 2.1 miles east of the project site (Local Well Name CHINO-1208672), the shallowest groundwater level measured for a monitoring period between September 2011 and April 2022 was approximately 135 feet bgs in April 2020.

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 southwestern and southeastern portions of the site to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with the *San*

Bernardino County Technical Guidance Document (TGD) for Water Quality Management Plans (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 falling-head test method was employed for test wells LP-1 and LP-2 in which the volume of discharge was calculated by adding the total volume of water that dropped within the PVC pipe and within the annulus, and incorporating a porosity reduction factor to account for the porosity of the annulus material. The flow area was based on the average water height within the slotted pipe section of the test well. The infiltration rate was calculated by dividing the rate of discharge by the infiltration surface area, or flow area.

Detailed results of the field testing and measured infiltration rate for the tests performed 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	0.09
LP-2	5 to 10	0.05

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 0.09 inch per hour (LP-1) and 0.05 inch per hour (LP-2), respectively. According to the *San Bernardino County Technical Guidance Document (TGD) for Water Quality Management Plans* (2013), the measured infiltration rate at both test well locations do **not** meet the minimum feasibility criteria of 0.3 inch per hour. Therefore, due to the unfavorable infiltration characteristics of the subsurface at the tested locations and depths, stormwater infiltration is not feasible at the subject site.

2.5 Surface Fault Rupture

Our review of available literature indicates that no known active faults have been mapped across the site, and the site is **not** located within a currently established *Alquist-Priolo Earthquake Fault Zone* (CGS, 2018; Bryant and Hart, 2007). Therefore, a surface fault rupture hazard evaluation is not mandated for this site and the potential for surface fault rupture at the site is expected to be low.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008b). The closest active faults to the site with the potential for surface fault rupture are the Chino fault, San Jose fault and Elsinore Fault Zone located approximately 2.4 miles, 6.5 miles and 8.7 miles from the site, respectively. The San Andreas fault, which is the largest active fault in California, is approximately 22 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.6 Strong Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California (Figure 4). The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

Accordingly, design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). The 2022 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 2022 CBC:

Table 2 – 2022 CBC Based Ground Motion Parameters (Mapped Values)

Categorization/Coefficient	Value
Site Latitude	34.0003°
Site Longitude	-117.6792°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.682 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.6 g
Short Period (0.2 sec) Site Coefficient, F_a	1.0
Long Period (1 sec) Site Coefficient, F_v	1.7[1]
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.682 g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	1.02 g[1]
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.122 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.68 g[1]
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.757 g

[1]See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters SM1 and SD1 in the table are increased by 50%.

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.

The project site is in an area that has not been evaluated by the CGS for liquefaction hazard or earthquake-induced landslide hazard. However, based on review of the *San Bernardino County Land Use Plan – Geologic Hazard Overlays* map (San Bernardino County, 2010), the project site is **not** located within a liquefaction susceptibility zone. In addition, groundwater was not encountered during our subsurface investigation to the maximum depth explored of approximately 51½ feet bgs and current depth to groundwater is anticipated to be greater than 100 feet bgs. Based on these findings, liquefaction 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 onsite natural soils below the anticipated bearing grade of the proposed structure, the potential 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 constrained laterally, earthquake-induced lateral spreading is also not considered a hazard at the site.

2.10 Earthquake-Induced Landsliding

The project site is located in an area that has not been evaluated by the CGS for earthquake-induced landslide hazard. However, based on review of the *San Bernardino County Land Use Plan – Geologic Hazard Overlays* map (San Bernardino County, 2010), the project site is **not** located within a landslide susceptibility zone. Based on this consideration, the potential for seismically-induced landslide hazard at the site is not considered a hazard at the site.

2.11 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site is located within an area identified as having a 0.2 percent annual chance flood hazard. Accordingly, the project site **is** located within a 500-year flood hazard zone as shown on Figure 5, *Flood Hazard Zone Map*. Regionally, storm runoff flow is generally directed to the south.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. As shown on Figure 6, the site is **not** mapped within a dam inundation zone. Therefore, the risk of seismically-induced flooding due to dam failure is considered low.

2.12 Seiches and Tsunamis

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

2.13 Methane

Based on review of State of California Geologic Energy Management Division (CalGEM) records, the project site is **not** located within an oil field boundary and there are no documented oil wells onsite (CalGEM, 2023). The nearest documented oil well to the site is located approximately 0.5 miles northwest of the site (API# 0407100142; Lease by C.S. Summar, Well No. 1) and is reported as idle (CalGEM, 2023). Based on these findings, the potential for methane hazard at the site is low.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

The proposed structure may be supported on 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 the proposed building pad and other structural improvement areas prior to placement of engineered fill. We estimate removal and recompaction of existing undocumented fill materials will generally be on the order of approximately 2 to 3 feet below existing grades across the site. Localized areas anticipated to require deeper removals, in particular beneath existing building. The lateral extent of overexcavation beyond foundations should be equal to the depth of overexcavation below the proposed foundations.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. Additional exploration and/or evaluation may be required in the future once more detailed development plans become available. 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 Chino, the County of San Bernardino and other governing agencies.

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

3.1 Site Grading

Earthwork for the project is expected to consist of surcharging and removal of unsuitable soil materials, excavation, and placement of compacted fill. We recommend that earthwork on the site be performed in accordance with the recommendations presented in this report and the project specifications as prepared by others. The *Earthwork and Grading Guide Specifications* included in Appendix D may be used for guidance in developing the project specifications. If conflict arises, the recommendations in Appendix D shall be superseded by the project specifications, recommendations contained in this report and/or the County of San Bernardino Guidelines, whichever is more stringent. All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, 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. All undocumented fill or man-made debris, unsuitable native soils and former foundation remnants should be excavated and removed from the proposed building/structure footprint prior to placement of engineered fill.

3.1.2 Removals and Overexcavations

To provide uniform foundation support and reduce the potential for excessive static settlement, all existing undocumented fill and any unsuitable soil, as deemed by the geotechnical engineer, should be removed to expose suitable native soils and replaced as engineered fill below the proposed building and other structural improvements. Based on our field explorations, we estimate removals of existing undocumented fill will be approximately 2 to 3 feet below existing grades across the site. Localized areas may require deeper removals as determined during grading by a representative of the geotechnical engineer depending on observed subsurface conditions. Unexplored portions of the site, including areas beneath existing building, in areas of existing utilities, and areas disturbed during demolition of existing buildings and improvements may also require deeper removals. The lateral extent of removals and overexcavation beyond foundations should be equal to the depth of overexcavation below the proposed foundations.

The depth of overexcavation in non-structural areas planned for new pavement construction is recommended to be 2 feet below the current grade or planned subgrade elevation to develop a suitable bearing subgrade for pavement support. Deeper overexcavations in localized areas may be recommended during grading by a representative of the geotechnical engineer depending on observed subsurface conditions. Preparation limited to 2 feet of overexcavation below subgrade may result in the need for increased pavement maintenance and periodic repairs where existing undocumented fill is left in place below the recommended overexcavation depth of 2 feet. Alternatively, removals can be performed such that all undocumented fill is removed to expose suitable natural soils (alluvium) and replaced as engineered fill.

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

Onsite soil that is free of construction debris, organics, cobbles, boulders, rubble, or rock larger than 6 inches in largest dimension is suitable to be used as fill for support of structures. Oversized materials larger than 6 inches in diameter encountered during site grading may require special handling, and may be placed in non-structural areas or areas of deep fill at depth below anticipated excavations such as for any footings, utilities, future developments, etc. 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 thin, loose lifts, moisture-conditioned to within 2 percent of optimum moisture content and compacted using appropriate equipment and methods to achieve 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.

3.1.6 Shrinkage

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

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

3.1.7 Reuse of Concrete and Asphalt Rubble

If encountered during site clearing and/or during preparation activities, construction rubble (i.e., Portland cement concrete and asphalt concrete) may be incorporated in the proposed development. For use as structural fill, the processed material should be crushed to develop a relatively well-graded mixture with a maximum particle size of 3-inch nominal diameter. Concrete rubble should be free of rebar and processed asphalt pavement rubble may be used if mixed with the existing base course (where present). Processed material may be used as structural fill if uniformly mixed with onsite soils in proportion of 1 part processed material to 3 parts soil. For use as pavement base course, crushed material should satisfy gradation requirements of Section 200-2.4 of the *Standard Specifications for Public Works Construction* (Greenbook), 2021 Edition. 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 the proposed building. Footings should be embedded a minimum 12 inches below the lowest adjacent grade. An average allowable soil bearing pressure of 3,000 pounds per square foot (psf) may be used for footings with a minimum width of 12 inches for continuous footings and 18 inches for isolated footings.

A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used. The ultimate bearing capacity can be taken as 9,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.5 should be used for initial bearing capacity evaluation with factored loads.

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

The allowable bearing capacity for shallow footings is based on a total static 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 250 psf per foot of depth to a maximum of 2,500 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 moderate exposure to water-soluble sulfates in the soil (Exposure Class S1). Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC 2022 requirements. However, concrete exposed to recycled water should be designed using Type V cement.

Based on our laboratory testing, the onsite soil is considered mildly 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, field and laboratory verification are recommended before use. However, site soils can be variable in composition, clast size and expansive characteristics. 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 7, *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)	250
Seismic Increment (add to active pressure)	25

*Only for level and drained properly compacted backfill

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

3.5.1 Sliding and Overturning

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

3.5.2 Drainage

Adequate drainage may be provided by a subdrain system positioned behind the walls. Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.6 of the *Standard Specifications for Public Works Construction* (Greenbook), 2021 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the *Standard Specifications for Public Works Construction* (Greenbook), 2021 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

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

3.6 Paving

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

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 40, compacted to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557 as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on a near surface samples of existing onsite soils indicate a value of 46.

Table 4 – Asphalt Concrete Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5	3	4
6	4	4½
7	4	7
8	4	9½
9	6	8½

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 onsite soils will have an R-value of at least 45, which will need to be verified after the completion of site grading.

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	5½	4
6	6½	4
7	7	4
8	7½	4
9	8	4

The paving should be provided with control joints or 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* (Greenbook), 2021 Edition. The base course should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D 1557.

3.7 Temporary Excavations

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

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

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

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

3.8 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the *Standard Specifications for Public Works Construction* (Greenbook), 2021 Edition. Utility trenches can be backfilled with onsite sandy material free of rubble, debris, organic and oversized material up to (≤) 3-inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

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

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* (Greenbook), 2021 Edition. CLSM should not be jetted.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.9 Drainage and Landscaping

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

3.10 Additional Geotechnical Services

Leighton should review the grading plans, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated.

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 ASFE'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 San Bernardino County. We do not make any warranty, either expressed or implied.

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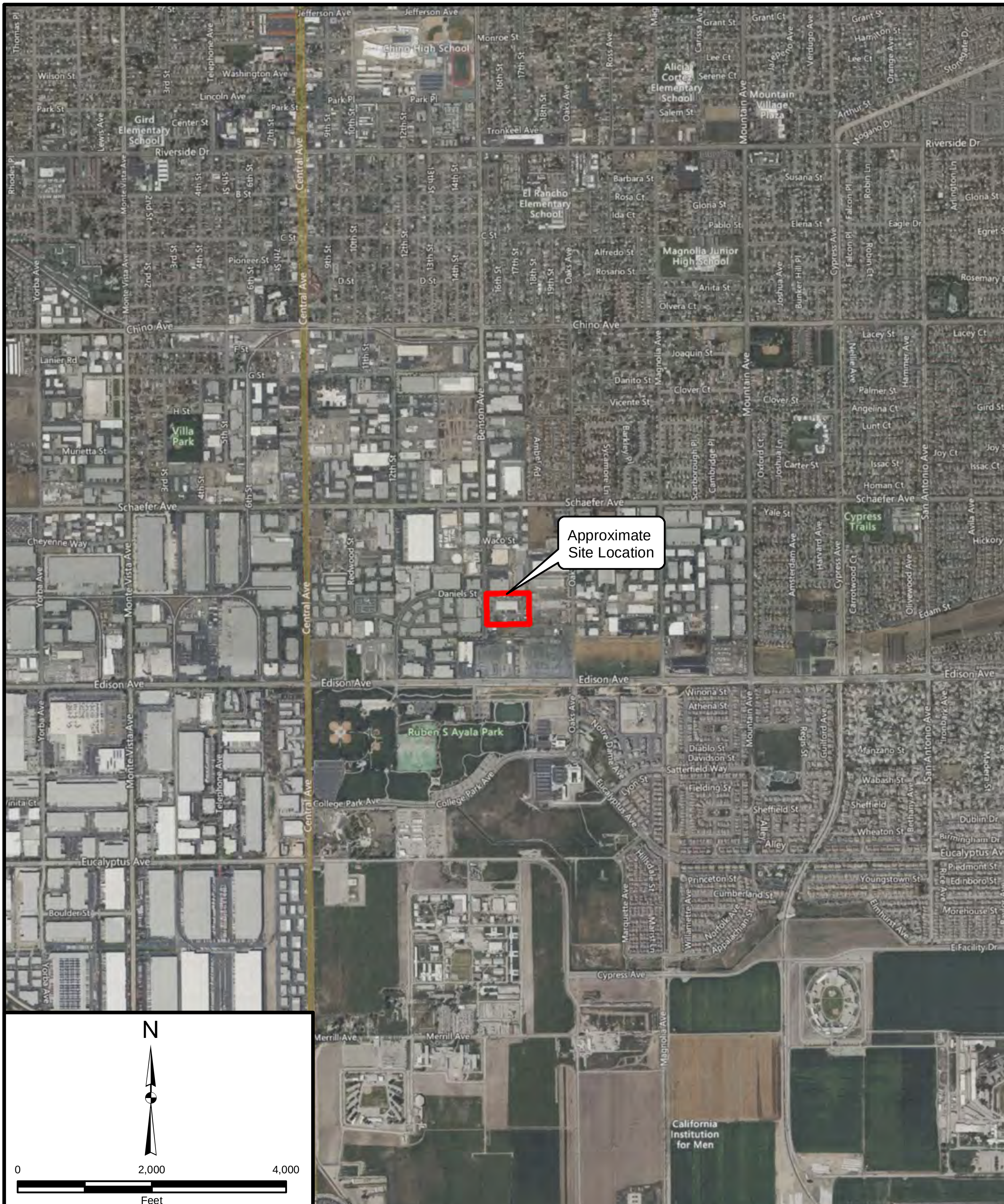

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: March 2023

Reference: © 2023 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

SITE LOCATION MAP

Proposed Industrial Building
13925 Benson Avenue
Chino, California

FIGURE 1

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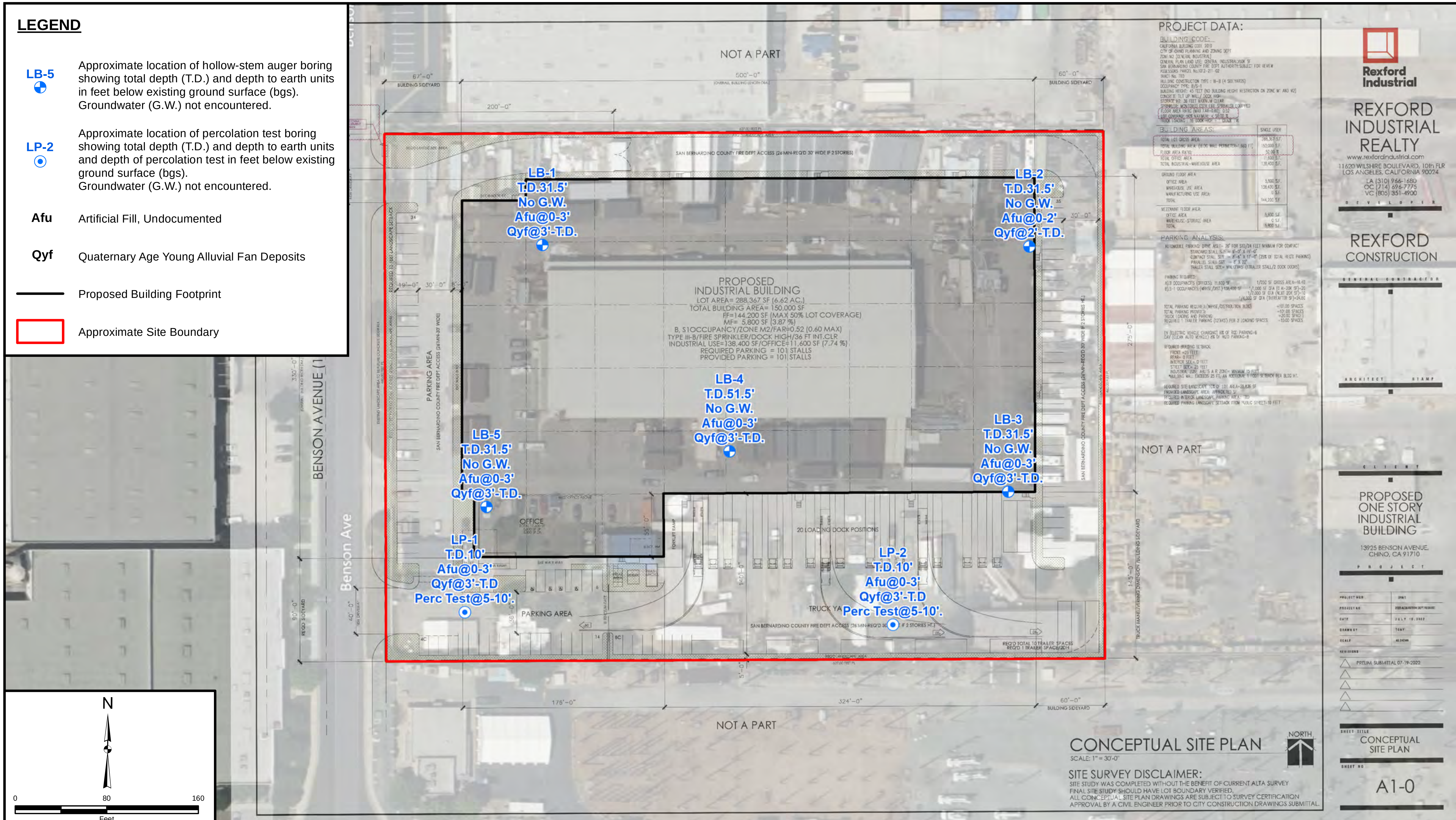

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 80 '	Date: March 2023
Reference: As Shown	

EXPLORATION LOCATION MAP

Proposed Industrial Building
13925 Benson Avenue
Chino, California

FIGURE 2

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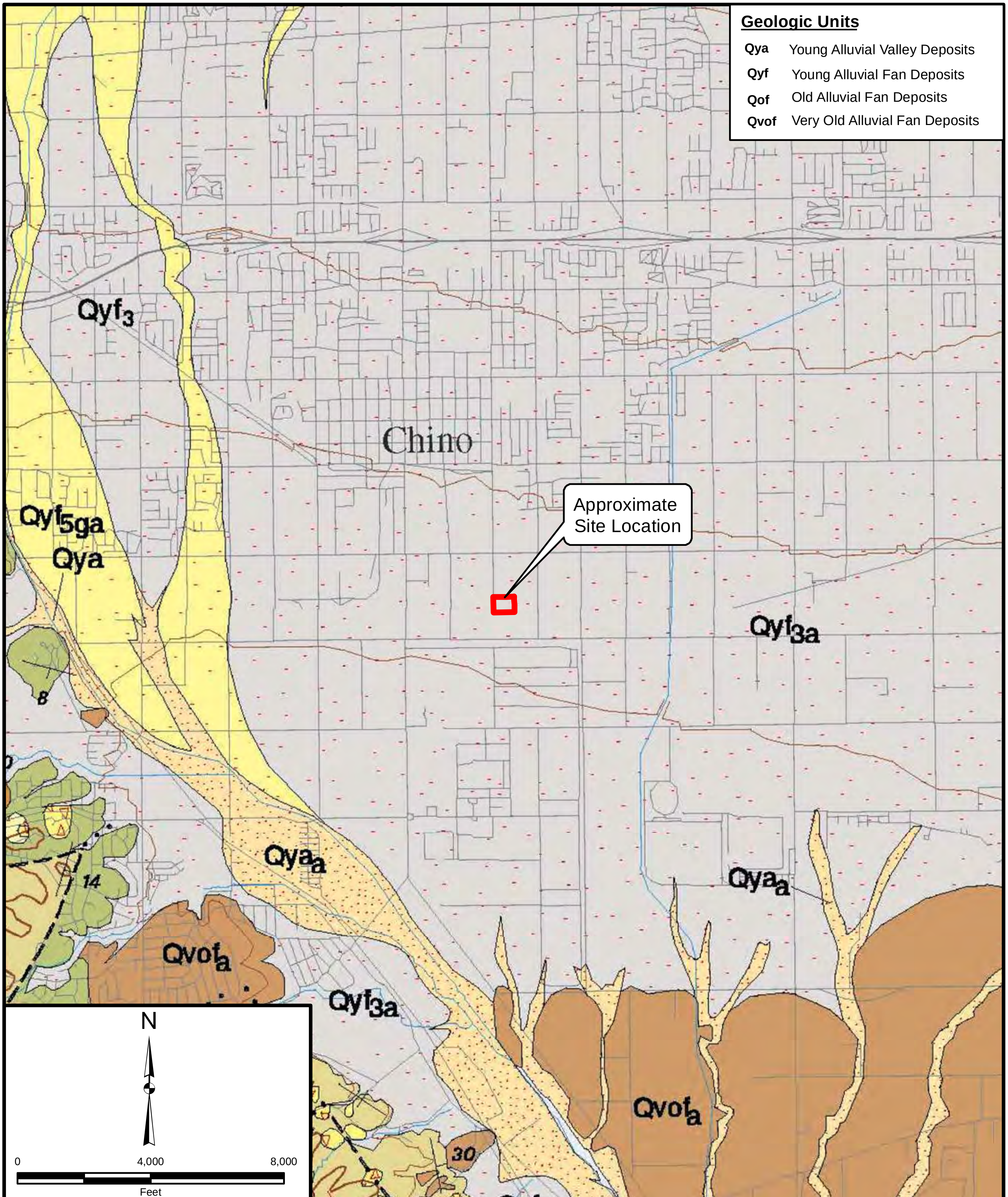

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: March 2023

Reference:
Geologic Map of the San Bernardino and
Santa Ana Quadrangles, California
Douglas M. Morton and Fred K. Miller, 2006

REGIONAL GEOLOGY MAP

Proposed Industrial Building
13925 Benson Avenue
Chino, California

FIGURE 3

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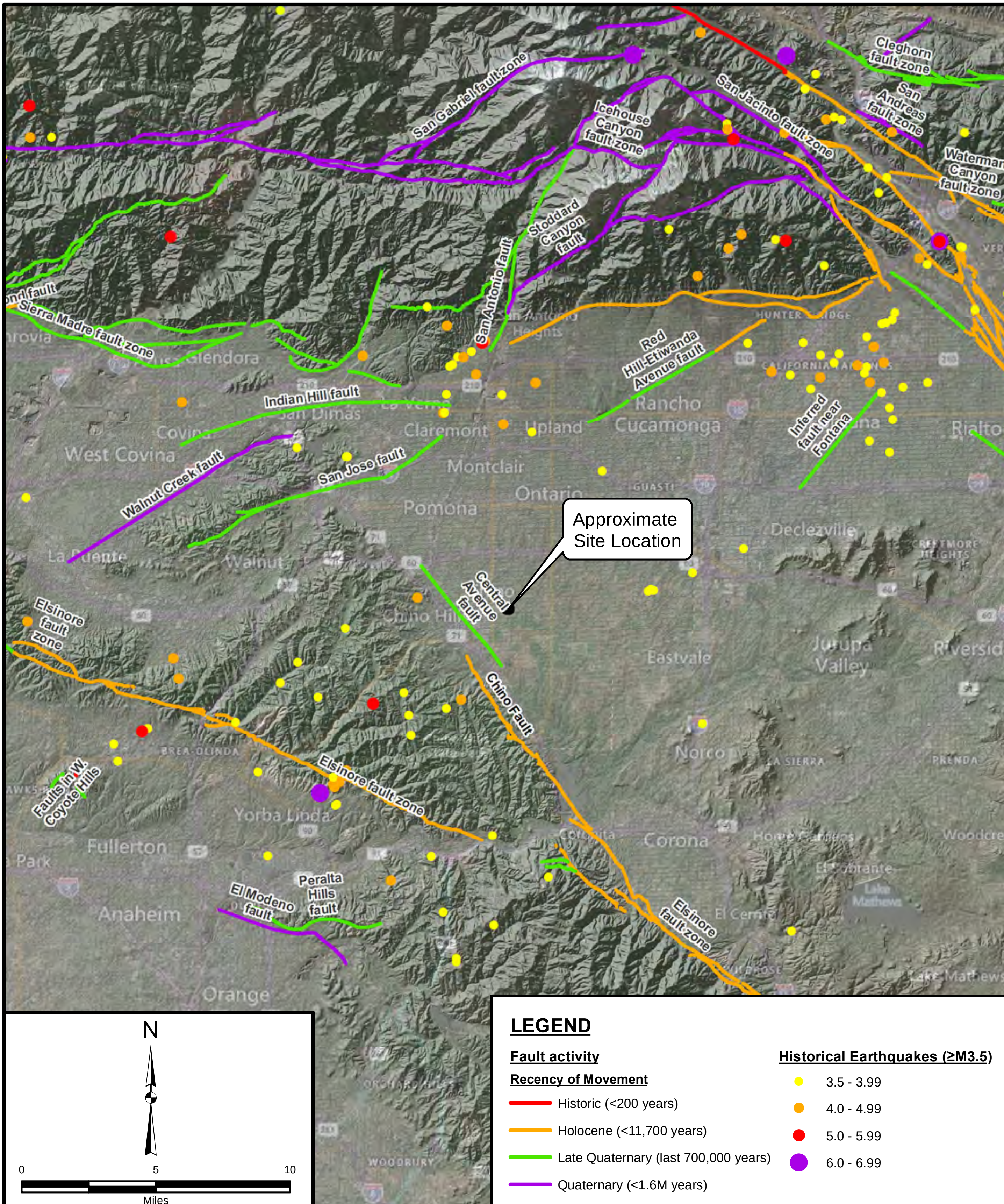

LEGEND

Fault activity

Recency of Movement

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

Historical Earthquakes (≥M3.5)

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

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 5 miles	Date: March 2023

Basemap Reference: © 2023 Microsoft Corporation Earthstar Geographics SIO © 2022 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Proposed Industrial Building
13925 Benson Avenue
Chino, California

FIGURE 4

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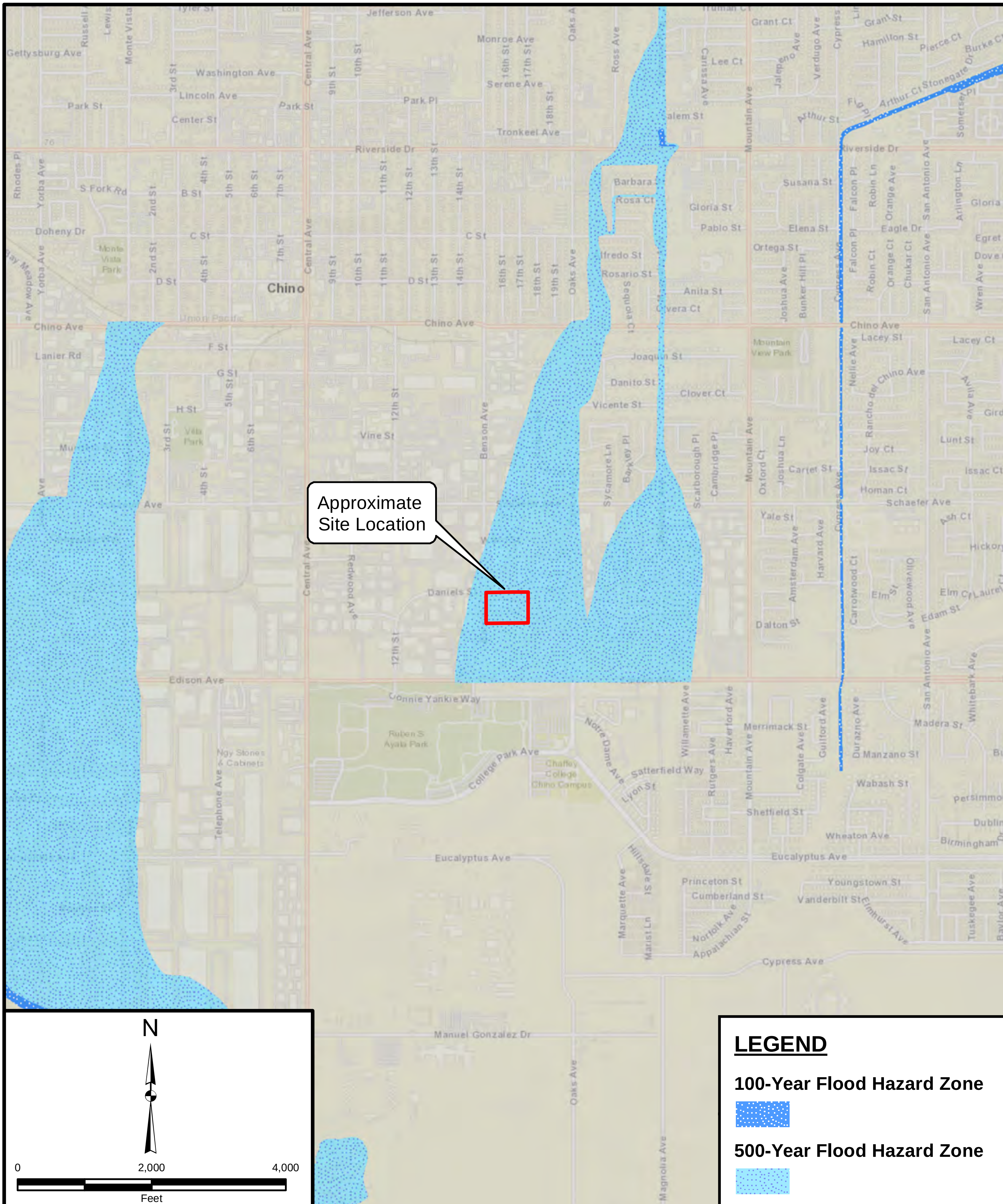

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 2,000 '	Date: March 2023

Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
FEMA (http://www.fema.gov/index.shtm), DWR (http://www.dwr.ca.gov)

FLOOD HAZARD ZONE MAP

Proposed Industrial Building
13925 Benson Avenue
Chino, California

FIGURE 5

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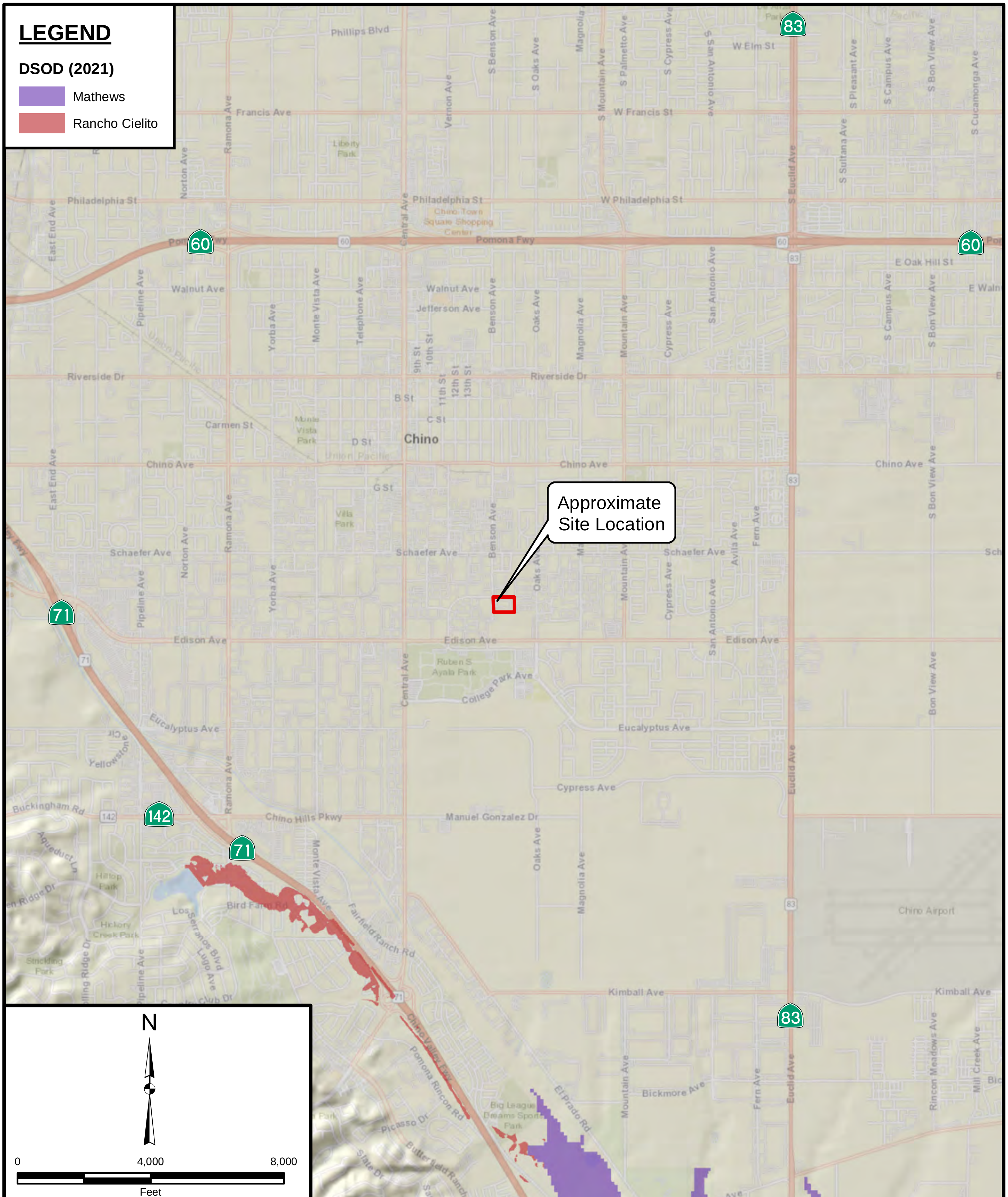

Project: 13807.001	Eng/Geol: CCK/JMP
Scale: 1 " = 4,000 '	Date: March 2023

Base Map: ESRI ArcGIS Online 2023
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
13925 Benson Avenue
Chino, California

FIGURE 6

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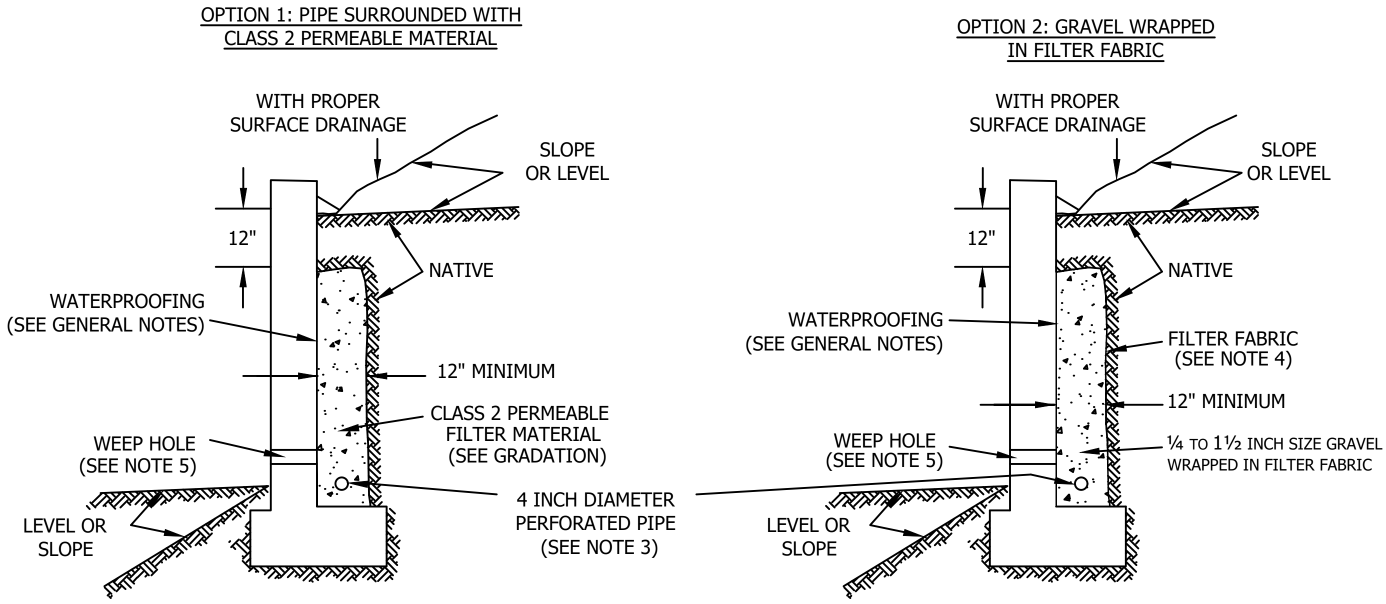
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

FIGURE 7

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No. 13807.001
Project Rexford Chino Benson Avenue
Drilling Co. Martini Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Exploration Location Map

Date Drilled 2-7-23
Logged By MM
Hole Diameter 8"
Ground Elevation 682'
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" asphalt concrete, no base	
								SM	**Undocumented Artificial Fill (Afu)** @3": Silty SAND, dark brown, fine sand, moist	
680										
								SM	**Quaternary Age Young Alluvial Fan Deposits (Qyf)**	
	5			S-1	3 6 7		12		@5': Silty SAND, light gray, medium dense, fine sand, moist	
675										
				R-2	4 5 5	100	22	CL	@7.5': Lean CLAY, light gray, medium stiff, medium plasticity, moist, FeO staining, CaCO3 nodules	
	10			S-3	1 1 1		19		@10': Lean CLAY, olive brown, soft, medium plasticity, moist, FeO staining, CaCO3 precipitate and nodules	AL
670										
	15			R-4	3 9 13	117	16		@15': Lean CLAY, brown, very stiff, medium plasticity, moist, abundant MnO spots	
665										
	20			S-5	2 6 6		17	SC	@20': Clayey SAND, yellow brown, medium dense, fine sand, moist, FeO staining, few MnO spots, Lean CLAY in shoe	
660										
	25			R-6	3 7 14	104	19	ML	@25': Sandy SILT, yellow brown, very stiff, fine sand, FeO staining, micaceous	
655										
	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

Leighton

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

GEOTECHNICAL BORING LOG LB-1

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	682'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S-7	6 10 12		13	SM	@30': Silty SAND, yellow brown, dense, fine sand, moist, oxidation throughout	
650									**T.D. 31.5' bgs** **No groundwater encountered during drilling** **Backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
	35									
645										
	40									
640										
	45									
635										
	50									
630										
	55									
625										
	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

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	684'
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			B-1				SM	@Surface: 2.5" asphalt concrete, no base **Undocumented Artificial Fill (Afu)** @2.5": Silty SAND, dark brown, fine sand, moist	
								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @2': SILT w/ Sand, gray brown, moist	
680	5			R-1	6 14 16	117	10	SM	@5': Silty SAND, gray brown, medium dense, fine sand, trace fine subangular gravel, moist	
				S-2	2 3 4		18	ML	@7.5': SILT, gray brown, stiff, moist, micaceous, trace MnO spots, few FeO stains	
675	10			R-3	4 4 6	104	20	CL	@10': Lean CLAY, gray brown, medium stiff, moist, micaceous, few FeO veins	
670	15			S-4	2 4 7		18	SC	@15': Clayey SAND, brown, medium dense, medium plasticity, moist, few FeO veins	
665	20			R-5	3 6 9	94	25	CL	@20': Lean CLAY, gray brown, very stiff, medium plasticity, moist, laminated, FeO staining along laminations, trace MnO spots	
660	25			S-6	2 4 8		27	SC	@25': Clayey SAND, gray brown, medium dense, medium plasticity, moist, FeO staining throughout, trace MnO spots	
655	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

Leighton

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GEOTECHNICAL BORING LOG LB-2

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	684'
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-7	3 8 16	112	20	CL	@30': Lean CLAY, gray brown, very stiff, few fine to coarse subrounded gravel, medium plasticity, moist, FeO staining	
									T.D. 31.5' bgs **No groundwater encountered during drilling** **Backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
650	35									
645	40									
640	45									
635	50									
630	55									
625	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

Leighton

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GEOTECHNICAL BORING LOG LB-3

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	683'
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: 4" asphalt concrete, no base **Undocumented Artificial Fill (Afu)** @4": Silty SAND, dark brown, fine sand, moist	
680								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @3': SILT, gray brown, moist	
	5			S-1	7 7 7		18		@5': SILT, light gray, trace light orange oxidation, very stiff, slightly moist, trace pores	
675				R-2	5 9 10	101	13		@7.5': Sandy SILT, light gray, very stiff, slightly moist, dark brown silty sand pocket, transitions to light brown Silty SAND w/ depth	
	10			S-3	2 2 3		17		@10': SILT, gray brown, medium stiff, moist, FeO staining, micaceous	
670										
	15			R-4	3 5 7	108	18	ML-CL CL	@15': Silty CLAY, brown, stiff, moist, FeO staining, micaceous @16': Lean CLAY, brown, stiff, low-medium plasticity, moist, FeO staining	
665										
	20			S-5	1 3 6		25		@20': Lean CLAY, brown, stiff, low plasticity, moist, FeO staining, micaceous	
660										
	25			R-6	3 6 12	95	27		@25': Lean CLAY, gray brown, stiff, low plasticity, moist, FeO veins and spots, micaceous	
655										
	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

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GEOTECHNICAL BORING LOG LB-3

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	683'
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
	30			S-7	1 3 5		19		@30': Lean CLAY, brown, stiff, low plasticity, moist, trace MnO spots	
650									**T.D. 31.5' bgs** **No groundwater encountered during drilling** **Backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
	35									
645										
	40									
640										
	45									
635										
	50									
630										
	55									
625										
	60									

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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	681'
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" asphalt concrete, no base	
680				B-1				SM	**Undocumented Artificial Fill (Afu)** @3.5": Silty SAND, very dark yellowish brown, mostly fine sand, moist	MD,EI, DS,CN, RV,CR
								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @3': Sandy SILT, gray brown, slightly moist to moist	
675	5			R-1	12 13 13	108	17		@5': Sandy SILT, brown, very stiff, fine sand, trace fine subrounded gravel and coarse sand, slightly moist	DS,CN
				S-2	4 3 2		16	CL	@7.5': Lean CLAY, gray brown, medium stiff, fine sand, slightly moist, trace oxide veins	
								ML-CL	@8.5': Clayey SILT, gray brown, medium stiff, no plasticity, moist, abundant MnO spotting	
670	10			R-3	3 5 8	101	20	ML	@10': Sandy SILT, brown, stiff, trace coarse subrounded gravel, no plasticity, moist, micaceous, faint FeO spotting throughout	DS
665	15			S-4	2 3 3			CL	@15': Lean CLAY, olive brown, medium stiff, low plasticity, moist, FeO spotting, trace MnO spots	AL
660	20			R-5	3 7 12	95	26		@20': Lean CLAY, gray brown, very stiff, low plasticity, moist, FeO spotting/pinpoints	
655	25			S-6	7 9 9		9	SM	@25': Silty SAND, light brown, medium dense, mostly fine sand, trace medium subangular gravel, moist, FeO veining throughout	
	30									

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

TYPE OF TESTS:

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

Leighton

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GEOTECHNICAL BORING LOG LB-4

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	681'
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
650	30			R-7	6 6 12	111	18	SC	@30': Clayey Sand, light gray brown, medium dense, mostly fine sand, trace coarse sand, moist, FeO veins, trace MnO spots	
645	35			S-8	13 25 30		5	SM	@35': Silty SAND, light brown, very dense, fine to coarse sand, fine to coarse subangular gravel, moist, FeO staining throughout	
								CL	Transitions to Lean CLAY, light gray brown, laminated, FeO-staining along laminations, moist	
640	40			R-9	8 14 18	111	10	SM	@40': Silty SAND, gray brown, medium dense, fine to coarse sand, few fine to coarse gravel, FeO staining, moist, mostly fine grained sand in show w/ laminated Lean CLAY interbedded	
635	45			S-10	4 8 18		32	CL	@45': Lean CLAY, brown, hard, few silt, trace to few fine sand, low plasticity, moist to very moist, micaceous, trace FeO staining	
630	50			R-11	6 11 12	102	23		@50': Lean CLAY, light gray, very stiff, low plasticity, moist, trace CaCO3 deposits, trace to few FeO spots/veins, trace burrows	
625	55								**T.D. 51.5' bgs** **No groundwater encountered during drilling** **Backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
	60									

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

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- 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

Leighton

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GEOTECHNICAL BORING LOG LB-5

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	681'
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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	@Surface: 3" asphalt concrete, no base **Undocumented Artificial Fill (Afu)** @3": Silty SAND, dark brown, fine sand, moist	
680								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @3': Sandy SILT, light gray, fine sand, slightly moist	
675	5			R-1	6 13 16	113	14		@5': SILT, light gray, medium dense, little fine sand, slightly moist, CaCO3 nodules	
				S-2	2 3 5		20	CL	@7.5': Lean CLAY, light gray, stiff, slightly moist	
670	10			R-3	2 3 7	100	22		@10': Lean CLAY, olive brown, medium stiff, low to medium plasticity, moist, FeO veins	AL,CN
665	15			S-4	push 1 2		22		@15': Lean CLAY, olive brown, soft, medium plasticity, moist, trace FeO veins	AL
660	20			R-5	2 3 7	94	27		@20': Lean CLAY, yellow brown, medium stiff, medium plasticity, FeO veins	
655	25			S-6	4 6 5		12	SM	@25': Silty SAND, yellow brown, medium dense, mostly fine sand, FeO staining throughout	
	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

Leighton

GEOTECHNICAL BORING LOG LB-5

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	681'
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.	
650	30			R-7	5 9 17	118	13	SC	@30': Clayey SAND, brown, medium dense, fine sand, trace medium to coarse sand, moist	
									T.D. 31.5' bgs **No groundwater encountered during drilling** **Backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
645	35									
640	40									
635	45									
630	50									
625	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

Leighton

GEOTECHNICAL BORING LOG LP-1

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	682'
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							SM	@Surface: 3" asphalt concrete, no base **Undocumented Artificial Fill (Afu)** @3": Silty SAND, dark brown, fine sand, moist	
680										
								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @3': SILT with sand, light gray, slightly moist	
	5			S-1	5 6 7		13		@5': SILT, light gray, very stiff, moist, few fine sand, trace fine gravel	
675										
				S-2	2 2 3		17		@8': SILT, light gray, medium stiff, moist, few fine sand, abundant FeO veins	
	10									
670									**T.D. 10' bgs** **No groundwater encountered during drilling** **Installed temporary test well using 2-inch diameter pipe** **Solid pipe from 0-5 feet bgs and 0.020-inch slotted pipe from 5-10 feet bgs** **No. 3 Monterey SAND placed in annulus from 4-10 feet bgs** **Upon completion of percolation test, removed pipe and backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
	15									
665										
	20									
660										
	25									
655										
	30									

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

TYPE OF TESTS:

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

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

GEOTECHNICAL BORING LOG LP-2

Project No.	13807.001	**Date Drilled**	2-7-23
Project	Rexford Chino Benson Avenue	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	682'
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							SM	@Surface: 3" asphalt concrete over 2" base **Undocumented Artificial Fill (Afu)** @3": Silty SAND, dark brown, fine sand	
680								ML	**Quaternary Age Young Alluvial Fan Deposits (Qyf)** @3': SILT with sand, light gray, moist	
	5			S-1	2 4 3		18	CL	@5': Lean CLAY, light gray, stiff, moist, no to low plasticity, trace FeO staining	
675				S-2	2 2 3		19		@8': medium stiff	
	10								**T.D. 10' bgs** **No groundwater encountered during drilling** **Installed temporary test well using 2-inch diameter pipe** **Solid pipe from 0-5 feet bgs and 0.020-inch slotted pipe from 5-10 feet bgs** **No. 3 Monterey SAND placed in annulus from 4-10 feet bgs** **Upon completion of percolation test, removed pipe and backfilled with soil cuttings and patched with cold-mix asphalt concrete**	
670										
	15									
665										
	20									
660										
	25									
655										
	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

APPENDIX B

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	13807.001	**Test Hole Number:**	LP-1
Project Name:	Rexford Benson Chino	**Date Excavated:**	2/7/2023
Earth Description:	Alluvium	**Date Tested:**	2/8/2023
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	MM	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	2/7/2023 14:00	**Length of slotted of casing (ft):**	5
Start Time for Standard:	7:37	**Depth to Initial Water Depth (ft):**	4
Standard Time Interval Between Readings, mins:	30	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data - Falling Head Test

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
1	7:37	30	4.32	68.2	2.5	0.06
	8:07		4.53	65.6		
2	8:07	30	4.53	65.6	3.1	0.07
	8:37		4.79	62.5		
3	8:37	30	4.79	62.5	2.5	0.06
	9:07		5.00	60.0		
4	9:07	30	5.00	60.0	2.8	0.07
	9:37		5.23	57.2		
5	9:38	30	5.00	60.0	3.5	0.09
	10:08		5.29	56.5		
6	10:09	30	5.00	60.0	3.4	0.09
	10:39		5.28	56.6		
7	10:40	30	5.00	60.0	3.5	0.09
	11:20		5.29	56.5		
8	11:22	30	5.00	60.0	3.4	0.09
	11:52		5.28	56.6		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 Readings) = 0.09 in./hr.

Boring Percolation Test Data Sheet

Project Number:	13807.001	**Test Hole Number:**	LP-2
Project Name:	Rexford Benson Chino	**Date Excavated:**	2/7/2023
Earth Description:	Alluvium	**Date Tested:**	2/8/2023
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	MM	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	2/7/2023 14:37	**Length of slotted of casing (ft):**	5
Start Time for Standard:	7:12	**Depth to Initial Water Depth (ft):**	5
Standard Time Interval Between Readings, mins:	30	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data - Falling Head Test

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
1	7:12	30	5.00	60.0	1.8	0.05
	7:42		5.15	58.2		
2	7:45	30	5.00	60.0	1.8	0.05
	8:15		5.15	58.2		
3	8:17	30	5.00	60.0	1.7	0.04
	8:47		5.14	58.3		
4	8:48	31	5.00	60.0	1.9	0.05
	9:19		5.16	58.1		
5	9:20	30	5.00	60.0	1.8	0.05
	9:50		5.15	58.2		
6	9:51	30	5.00	60.0	1.8	0.05
	10:21		5.15	58.2		
7	10:24	30	5.00	60.0	1.9	0.05
	10:54		5.16	58.1		
8	10:56	30	5.00	60.0	1.8	0.05
	11:26		5.15	58.2		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 Readings) = 0.05 in./hr.

APPENDIX C

LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Rexford/13925 Benson Ave/Geo — Tested By: MRV/FLM — Date: 02/16/23

Project No.: 13807.001 — Checked By: M. Vinet — Date: 03/03/23

Boring No.: LB-4 — Depth (ft.): 0 - 5.0

Sample No.: B-1

Soil Identification: Silty Sand (SM), Very Dark Yellowish Brown.

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

Mold Volume (ft³) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5470	5605	5645	5593		
Weight of Mold (g)	3524	3524	3524	3524		
Net Weight of Soil (g)	1946	2081	2121	2069		
Wet Weight of Soil + Cont. (g)	1255.3	1353.0	1390.0	1342.1		
Dry Weight of Soil + Cont. (g)	1191.2	1264.5	1280.5	1220.0		
Weight of Container (g)	276.8	280.2	280.5	277.7		
Moisture Content (%)	7.0	9.0	11.0	13.0		
Wet Density (pcf)	128.4	137.4	140.0	136.6		
Dry Density (pcf)	120.0	126.0	126.2	120.9		

Maximum Dry Density (pcf) 126.8 — **Optimum Moisture Content (%)** 10.0

PROCEDURE USED

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

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

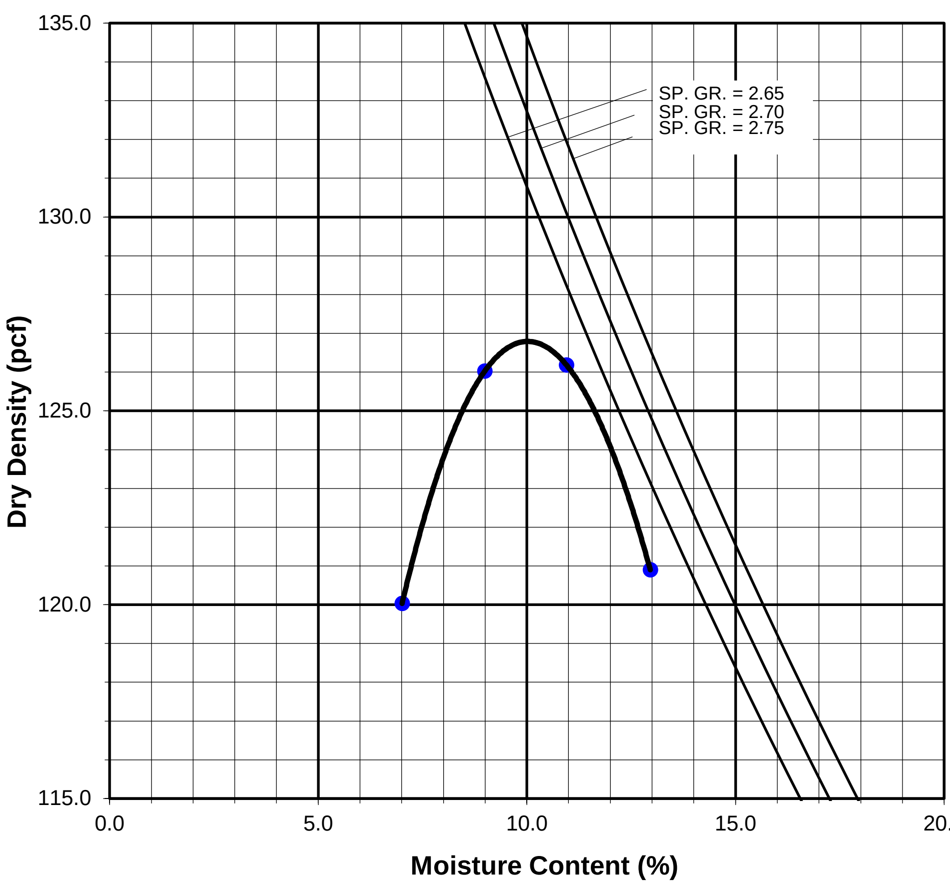
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Compaction; LB-4, B-1 (02-07-23)

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EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	M. Vinet	Date:	2/22/23
Project No. :	13807.001	Checked By:	M. Vinet	Date:	2/23/23
Boring No.:	LB-4	Depth:	0 - 5.0		
Sample No. :	B-1	Location:	N/A		
Sample Description:	Silty Sand (SM), Very Dark Yellowish Brown.				

Dry Wt. of Soil + Cont. (gm.)	2512.3
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	2512.3
Weight Soil Retained on #4 Sieve	22.1
Percent Passing # 4	99.1

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0060
Wt. Comp. Soil + Mold (gm.)	608.0	642.0
Wt. of Mold (gm.)	209.0	209.0
Specific Gravity (Assumed)	2.70	2.70
Container No.	10	10
Wet Wt. of Soil + Cont. (gm.)	350.1	642.0
Dry Wt. of Soil + Cont. (gm.)	324.1	364.4
Wt. of Container (gm.)	50.1	209.0
Moisture Content (%)	9.5	18.8
Wet Density (pcf)	120.4	129.8
Dry Density (pcf)	109.9	109.3
Void Ratio	0.534	0.543
Total Porosity	0.348	0.352
Pore Volume (cc)	72.0	73.3
Degree of Saturation (%) [S meas]	**48.1**	**93.6**

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.)
2/22/23	9:00	1.0	0	0.5000
2/22/23	9:10	1.0	10	0.5000
	Add Distilled Water to the Specimen			
2/23/23	8:00	1.0	1370	0.5060
2/23/23	9:00	1.0	1430	0.5060

Expansion Index (EI meas)	=	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	6.0
Expansion Index (Report)	=	Nearest Whole Number or Zero (0) if Initial Height is > than Final Height	**6**

Rev. 03-08

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ATTERBERG LIMITS
ASTM D 4318

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	M. Vinet	Date:	02/27/23
Project No. :	13807.001	Input By:	M. Vinet	Date:	02/28/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	S-3	Depth (ft.)	10.0		
Soil Identification:	Lean Clay (CL), Olive Brown.				

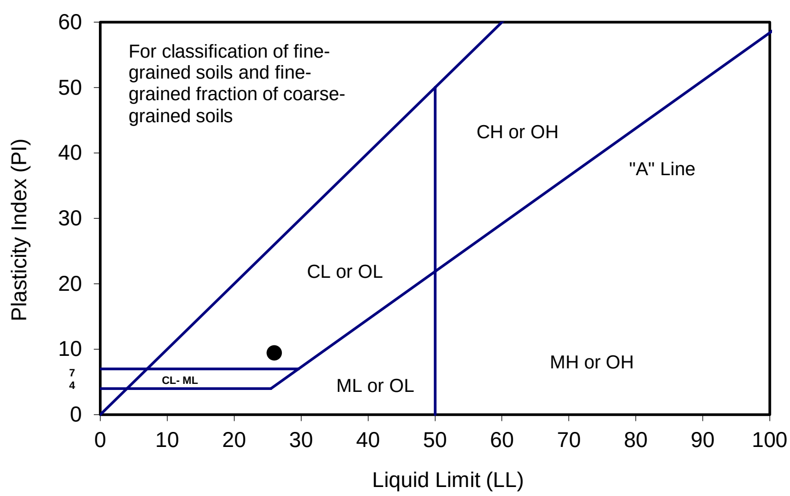
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	22	31	
Wet Wt. of Soil + Cont. (g)	21.19	21.02	23.71	24.84	27.58	
Dry Wt. of Soil + Cont. (g)	20.13	20.00	21.56	22.52	24.75	
Wt. of Container (g)	13.77	13.79	13.67	13.69	13.65	
Moisture Content (%) [Wn]	16.67	16.43	27.25	26.27	25.50	

Liquid Limit	**26**
Plastic Limit	**17**
Plasticity Index	**9**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 4.38

One - Point Liquid Limit Calculation

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



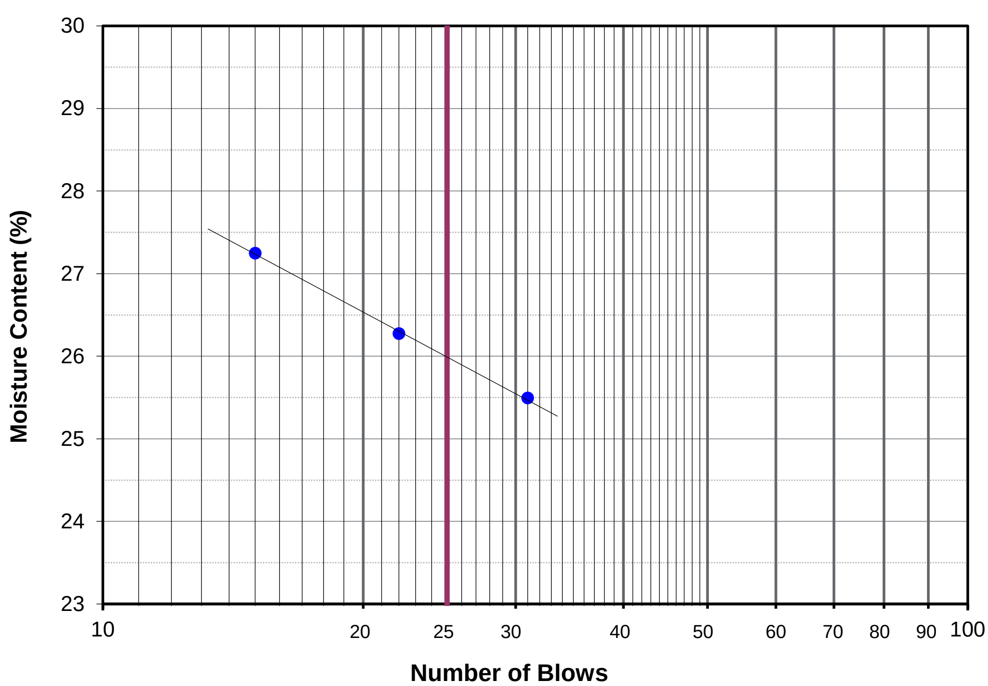
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS
ASTM D 4318

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	M. Vinet	Date:	02/27/23
Project No. :	13807.001	Input By:	M. Vinet	Date:	02/28/23
Boring No.:	LB-4	Checked By:	M. Vinet		
Sample No.:	S-4	Depth (ft.)	15.0		
Soil Identification:	Lean Clay (CL), Olive Brown.				

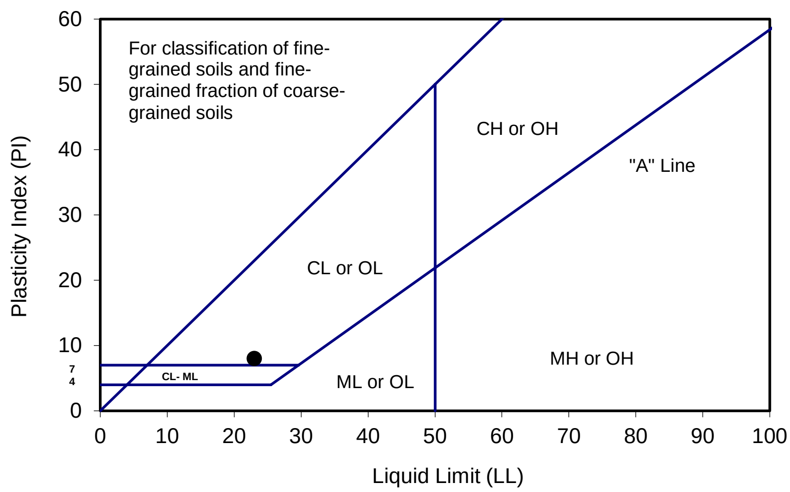
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	25	35	
Wet Wt. of Soil + Cont. (g)	23.77	21.58	25.69	25.53	28.49	
Dry Wt. of Soil + Cont. (g)	22.48	20.54	23.38	23.37	25.87	
Wt. of Container (g)	13.76	13.66	13.67	13.78	13.75	
Moisture Content (%) [Wn]	14.79	15.12	23.79	22.52	21.62	

Liquid Limit	**23**
Plastic Limit	**15**
Plasticity Index	**8**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 2.19

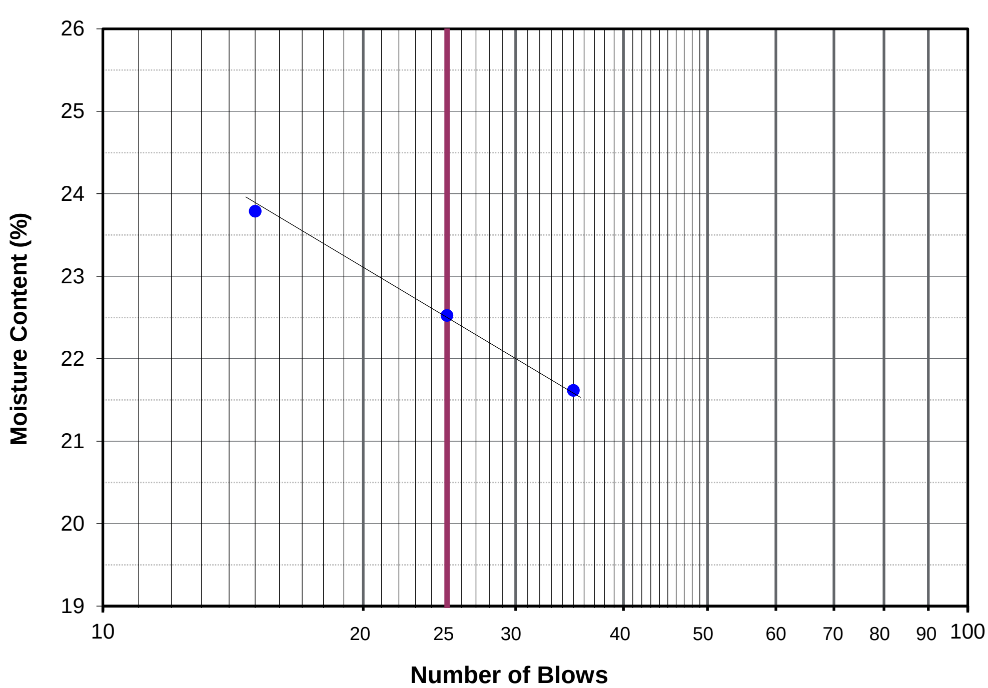
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



ATTERBERG LIMITS
ASTM D 4318

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	F. Mina	Date:	02/22/23
Project No. :	13807.001	Input By:	M. Vinet	Date:	02/28/23
Boring No.:	LB-5	Checked By:	M. Vinet		
Sample No.:	R-3	Depth (ft.)	10.0		
Soil Identification:	Lean Clay (CL), Olive Brown.				

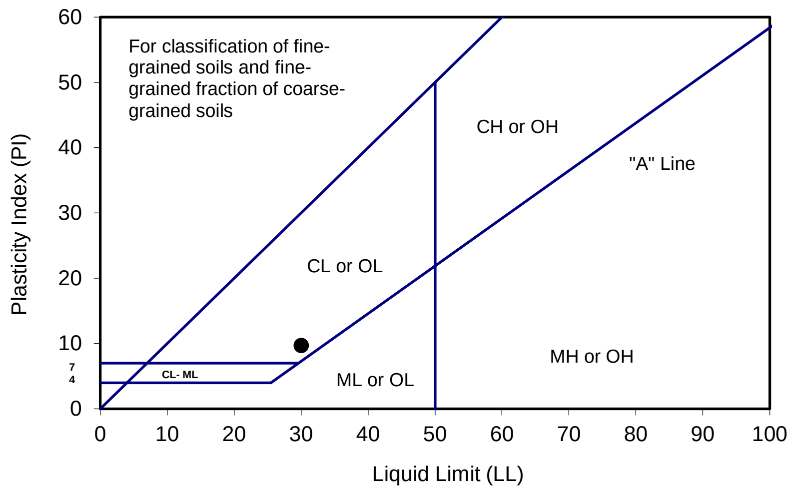
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	25	35	
Wet Wt. of Soil + Cont. (g)	28.45	21.32	22.62	21.76	24.02	
Dry Wt. of Soil + Cont. (g)	25.95	20.06	20.50	19.93	21.74	
Wt. of Container (g)	13.76	13.77	13.70	13.74	13.76	
Moisture Content (%) [Wn]	20.51	20.03	31.18	29.56	28.57	

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

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

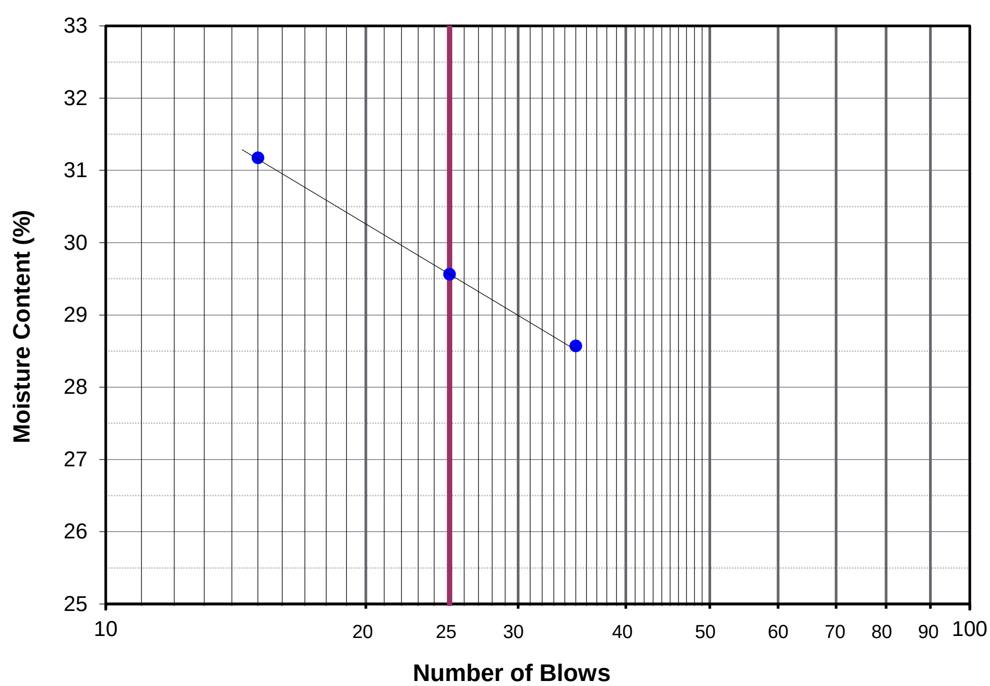
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



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ATTERBERG LIMITS
ASTM D 4318

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	F. Mina	Date:	02/22/23
Project No. :	13807.001	Input By:	M. Vinet	Date:	02/28/23
Boring No.:	LB-5	Checked By:	M. Vinet		
Sample No.:	S-4	Depth (ft.)	15.0		
Soil Identification:	Lean Clay (CL), Olive Brown.				

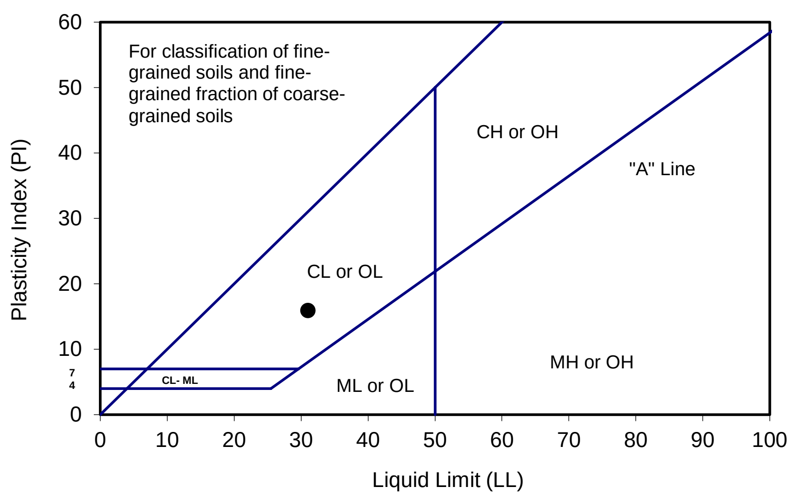
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	25	35	
Wet Wt. of Soil + Cont. (g)	23.27	26.29	22.33	21.59	23.68	
Dry Wt. of Soil + Cont. (g)	22.00	24.66	20.16	19.70	21.41	
Wt. of Container (g)	13.67	13.73	13.71	13.66	13.75	
Moisture Content (%) [Wn]	15.25	14.91	33.64	31.29	29.63	

Liquid Limit	**31**
Plastic Limit	**15**
Plasticity Index	**16**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 8.03

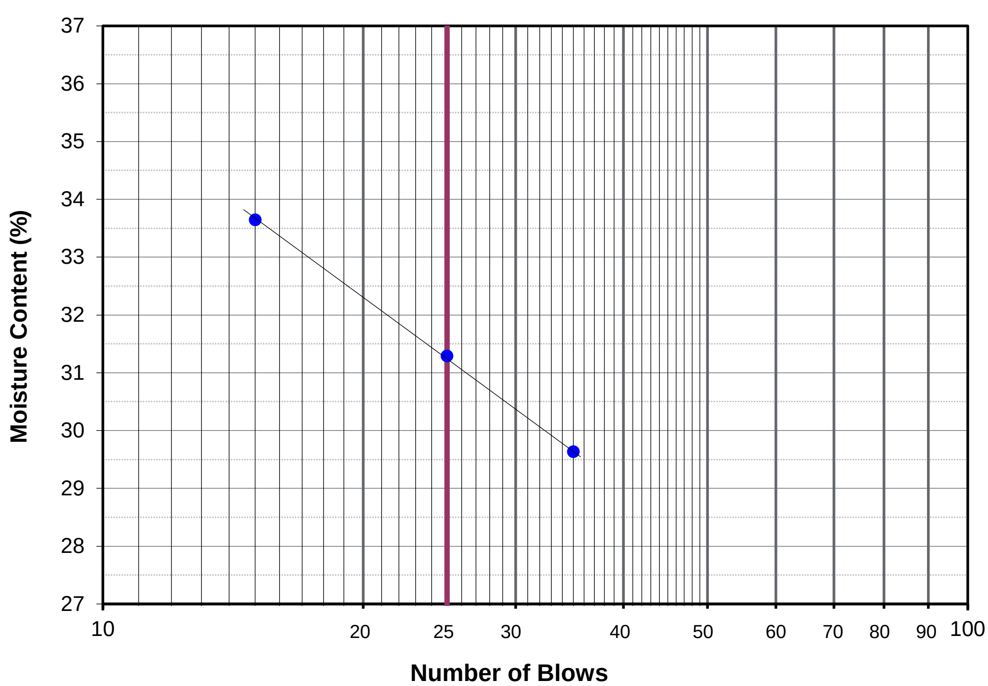
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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

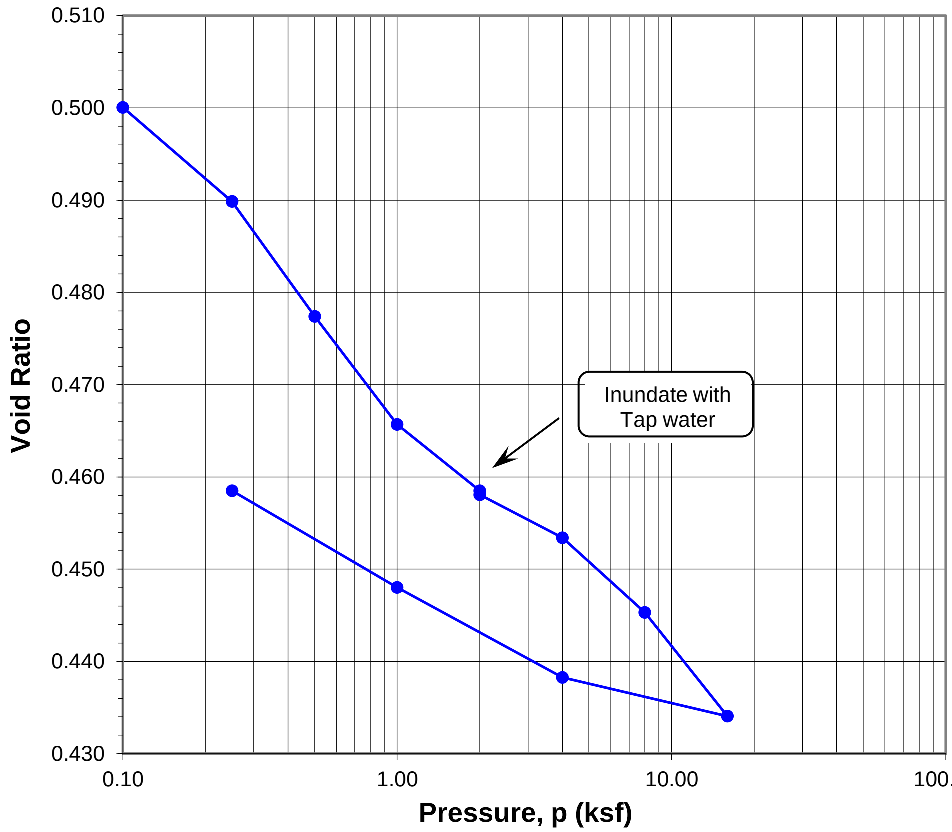


ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By: M. Vinet	Date: 02/23/23
Project No.:	13807.001	Checked By: M. Vinet	Date: 03/08/23
Boring No.:	LB-4	Depth (ft.): 0 - 5.0	
Sample No.:	B-1	Sample Type:	90% Remold
Soil Identification:	Silty Sand (SM), Very Dark Yellowish Brown.		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	194.00
Weight of Ring (g):	43.21
Height after consol. (in.):	0.9723
Before Test	
Wt. of Wet Sample+Cont. (g):	288.64
Wt. of Dry Sample+Cont. (g):	262.89
Weight of Container (g):	32.76
Initial Moisture Content (%)	11.2
Initial Dry Density (pcf)	112.8
Initial Saturation (%):	61
Initial Vertical Reading (in.)	0.0000
After Test	
Wt. of Wet Sample+Cont. (g):	251.27
Wt. of Dry Sample+Cont. (g):	230.32
Weight of Container (g):	50.58
Final Moisture Content (%)	15.34
Final Dry Density (pcf):	116.8
Final Saturation (%):	93
Final Vertical Reading (in.)	0.0310
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43

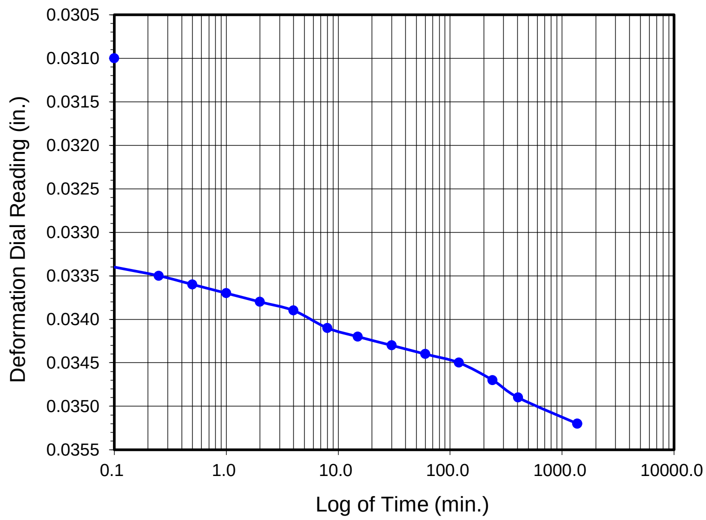


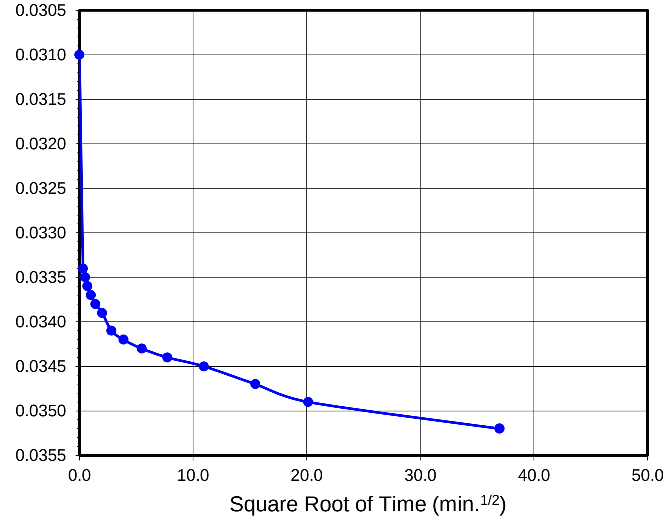
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.0000	1.0000	0.00	0.00	0.500	0.00
0.25	0.0072	0.9928	0.04	0.72	0.490	0.68
0.50	0.0160	0.9840	0.09	1.60	0.477	1.51
1.00	0.0248	0.9752	0.19	2.48	0.466	2.29
2.00	0.0296	0.9704	0.19	2.96	0.459	2.77
2.00	0.0310	0.9690	0.30	3.10	0.458	2.80
4.00	0.0352	0.9648	0.41	3.52	0.453	3.11
8.00	0.0420	0.9580	0.55	4.20	0.445	3.65
16.00	0.0512	0.9488	0.72	5.12	0.434	4.40
4.00	0.0468	0.9532	0.56	4.68	0.438	4.12
1.00	0.0391	0.9609	0.44	3.91	0.448	3.47
0.25	0.0310	0.9690	0.33	3.10	0.459	2.77

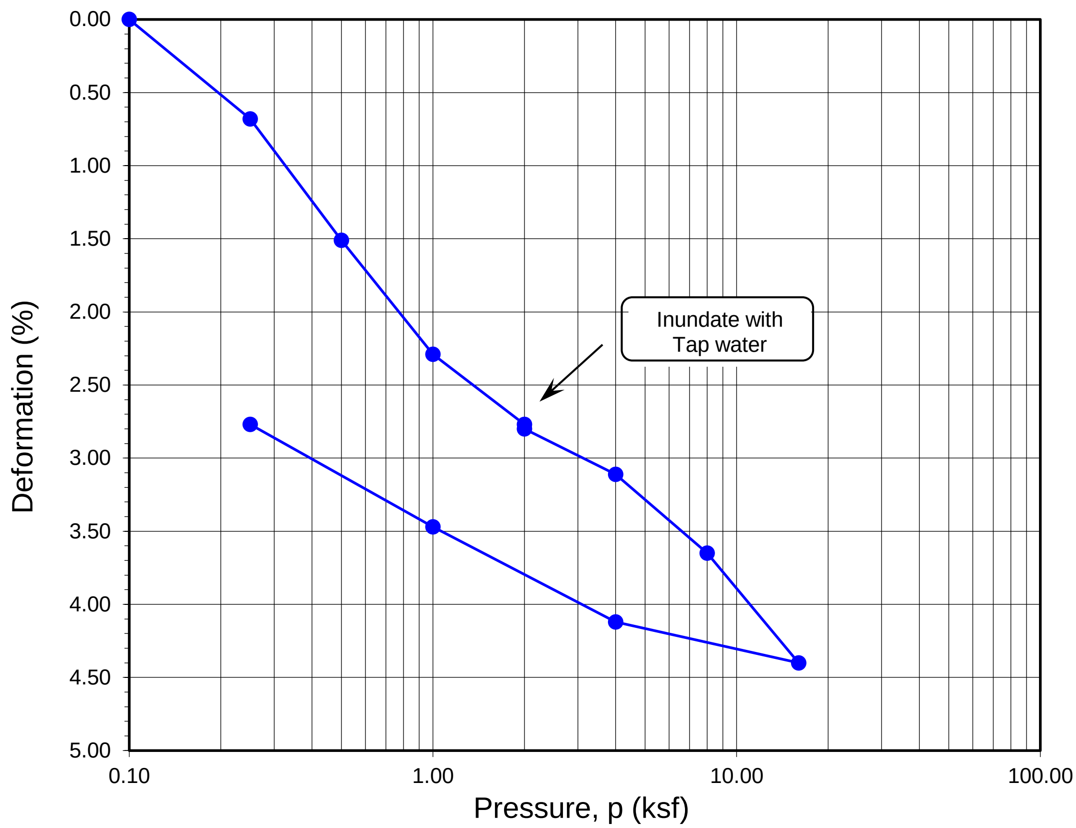
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/3/23	8:15:00	0.0	0.0	0.0310
3/3/23	8:15:06	0.1	0.3	0.0334
3/3/23	8:15:15	0.2	0.5	0.0335
3/3/23	8:15:30	0.5	0.7	0.0336
3/3/23	8:16:00	1.0	1.0	0.0337
3/3/23	8:17:00	2.0	1.4	0.0338
3/3/23	8:19:00	4.0	2.0	0.0339
3/3/23	8:23:00	8.0	2.8	0.0341
3/3/23	8:30:00	15.0	3.9	0.0342
3/3/23	8:45:00	30.0	5.5	0.0343
3/3/23	9:15:00	60.0	7.7	0.0344
3/3/23	10:15:00	120.0	11.0	0.0345
3/3/23	12:15:00	240.0	15.5	0.0347
3/3/23	15:00:00	405.0	20.1	0.0349
3/4/23	7:03:00	1368.0	37.0	0.0352
3/4/23	7:03:00	1368.0	37.0	0.0352

Time Readings @ 4.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-4	**B-1**	**0 - 5.0**	**11.2**	**15.3**	**112.8**	**116.8**	**0.500**	**0.459**	**61**	**93**

Soil Identification: Silty Sand (SM), Very Dark Yellowish Brown.

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

03-23

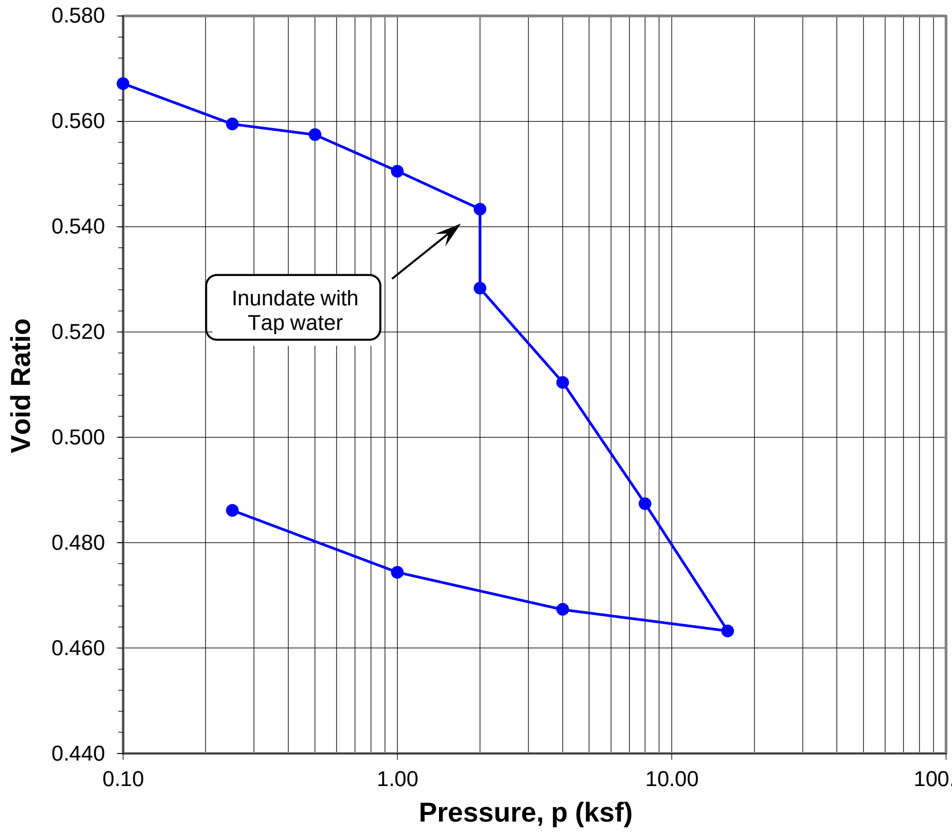
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By: M. Vinet	Date: 02/22/23
Project No.:	13807.001	Checked By: M. Vinet	Date: 03/08/23
Boring No.:	LB-4	Depth (ft.): 5.0	
Sample No.:	R-1	Sample Type:	Ring
Soil Identification:	Sandy Silt s(ML), Brown.		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	197.43
Weight of Ring (g):	45.36
Height after consol. (in.):	0.9483
Before Test	
Wt. of Wet Sample+Cont. (g):	192.22
Wt. of Dry Sample+Cont. (g):	171.43
Weight of Container (g):	50.20
Initial Moisture Content (%)	17.1
Initial Dry Density (pcf)	108.0
Initial Saturation (%):	82
Initial Vertical Reading (in.)	0.0000
After Test	
Wt. of Wet Sample+Cont. (g):	249.18
Wt. of Dry Sample+Cont. (g):	226.95
Weight of Container (g):	49.80
Final Moisture Content (%)	16.87
Final Dry Density (pcf):	115.6
Final Saturation (%):	99
Final Vertical Reading (in.)	0.0550
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43

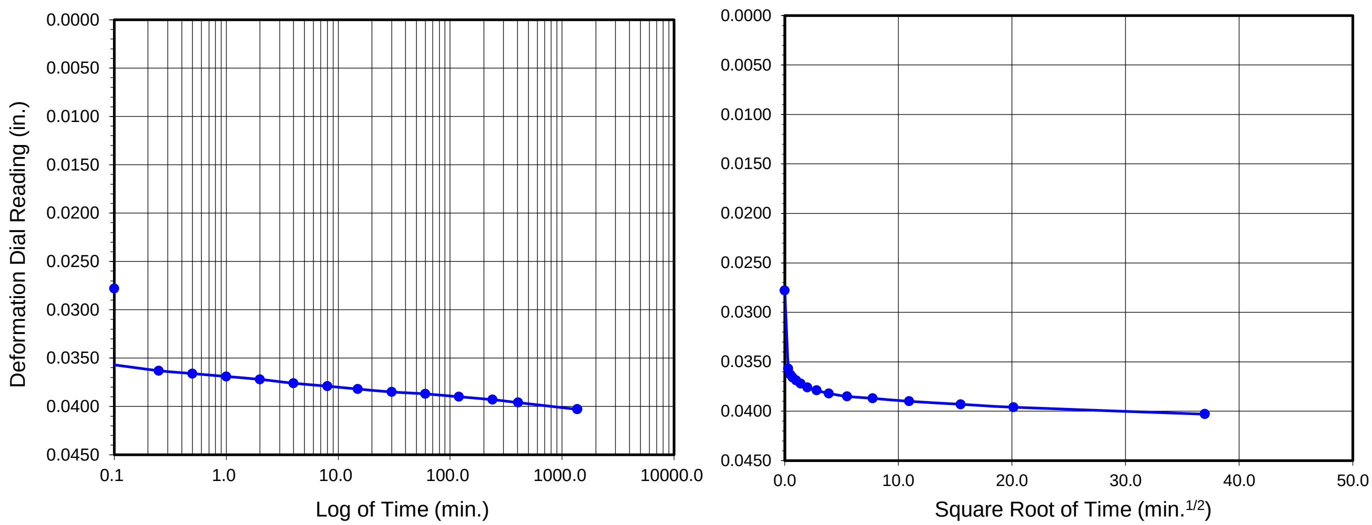


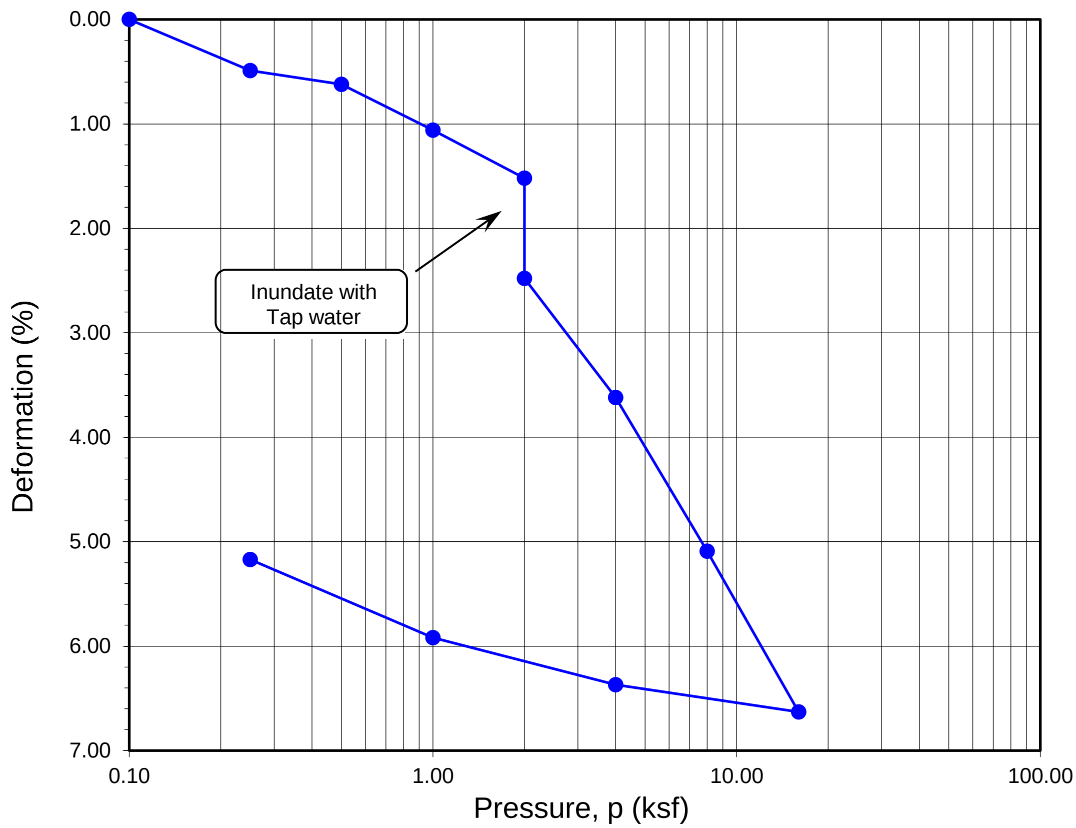
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.0000	1.0000	0.00	0.00	0.567	0.00
0.25	0.0053	0.9947	0.04	0.53	0.559	0.49
0.50	0.0071	0.9929	0.09	0.71	0.557	0.62
1.00	0.0125	0.9875	0.19	1.25	0.551	1.06
2.00	0.0171	0.9829	0.19	1.71	0.543	1.52
2.00	0.0278	0.9722	0.30	2.78	0.528	2.48
4.00	0.0403	0.9597	0.41	4.03	0.510	3.62
8.00	0.0564	0.9436	0.55	5.64	0.487	5.09
16.00	0.0735	0.9265	0.72	7.35	0.463	6.63
4.00	0.0693	0.9307	0.56	6.93	0.467	6.37
1.00	0.0636	0.9364	0.44	6.36	0.474	5.92
0.25	0.0550	0.9450	0.33	5.50	0.486	5.17

Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/2/23	8:15:00	0.0	0.0	0.0278
3/2/23	8:15:06	0.1	0.3	0.0357
3/2/23	8:15:15	0.2	0.5	0.0363
3/2/23	8:15:30	0.5	0.7	0.0366
3/2/23	8:16:00	1.0	1.0	0.0369
3/2/23	8:17:00	2.0	1.4	0.0372
3/2/23	8:19:00	4.0	2.0	0.0376
3/2/23	8:23:00	8.0	2.8	0.0379
3/2/23	8:30:00	15.0	3.9	0.0382
3/2/23	8:45:00	30.0	5.5	0.0385
3/2/23	9:15:00	60.0	7.7	0.0387
3/2/23	10:15:00	120.0	11.0	0.0390
3/2/23	12:15:00	240.0	15.5	0.0393
3/2/23	15:00:00	405.0	20.1	0.0396
3/3/23	7:03:00	1368.0	37.0	0.0403
3/3/23	7:03:00	1368.0	37.0	0.0403

Time Readings @ 4.0 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-4	**R-1**	**5**	**17.1**	**16.9**	**108.0**	**115.6**	**0.567**	**0.486**	**82**	**99**

Soil Identification: Sandy Silt s(ML), Brown.

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

03-23

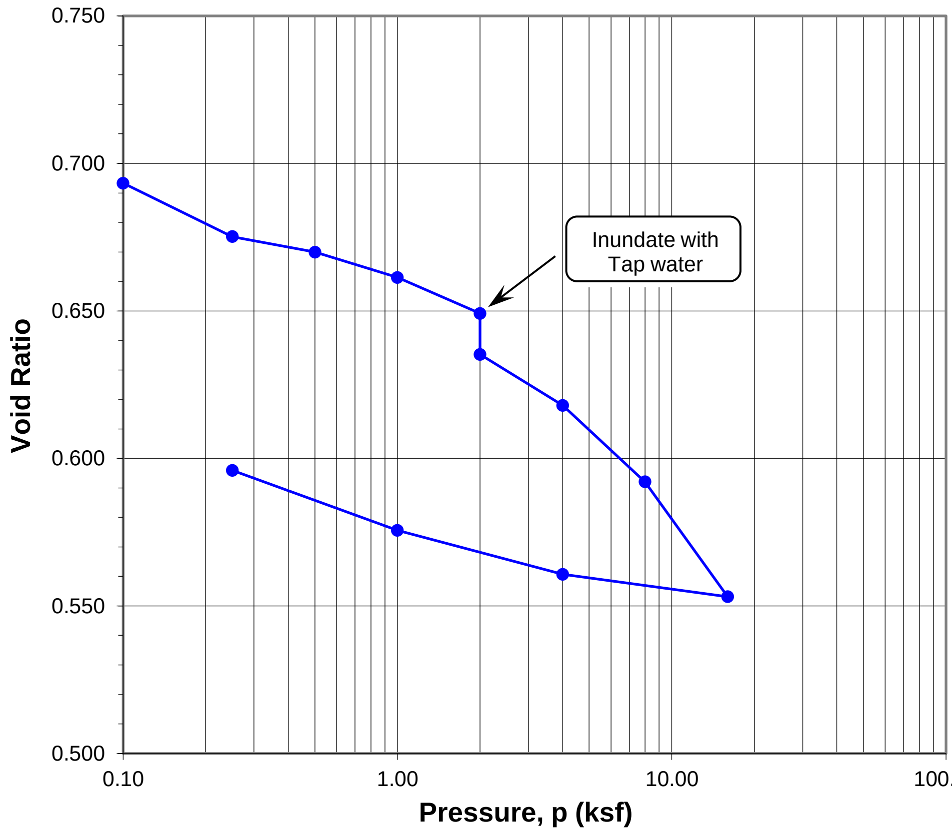
Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By: M. Vinet	Date: 02/23/23
Project No.:	13807.001	Checked By: M. Vinet	Date: 03/08/23
Boring No.:	LB-5	Depth (ft.): 10.0	
Sample No.:	R-3	Sample Type:	Ring
Soil Identification:	Lean Clay (CL), Olive Gray.		

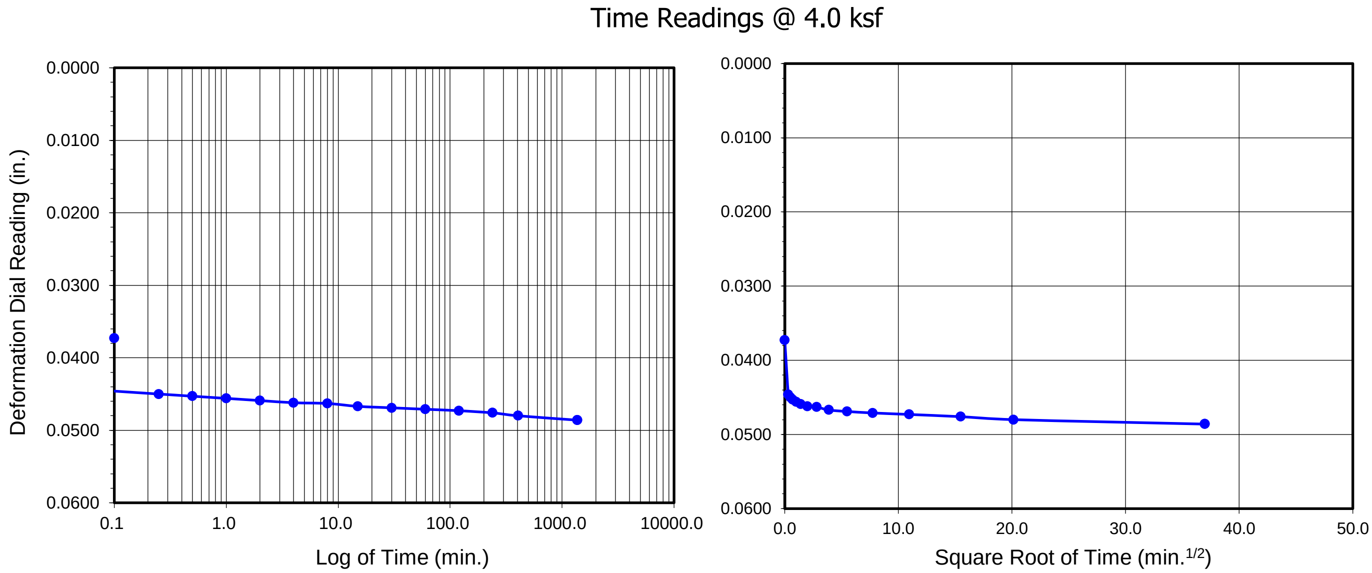
Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	190.71
Weight of Ring (g):	44.21
Height after consol. (in.):	0.9425
Before Test	
Wt. of Wet Sample+Cont. (g):	350.41
Wt. of Dry Sample+Cont. (g):	296.42
Weight of Container (g):	50.38
Initial Moisture Content (%)	21.9
Initial Dry Density (pcf)	99.9
Initial Saturation (%):	86
Initial Vertical Reading (in.)	0.0000
After Test	
Wt. of Wet Sample+Cont. (g):	239.70
Wt. of Dry Sample+Cont. (g):	211.25
Weight of Container (g):	49.80
Final Moisture Content (%)	24.27
Final Dry Density (pcf):	103.5
Final Saturation (%):	103
Final Vertical Reading (in.)	0.0608
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43

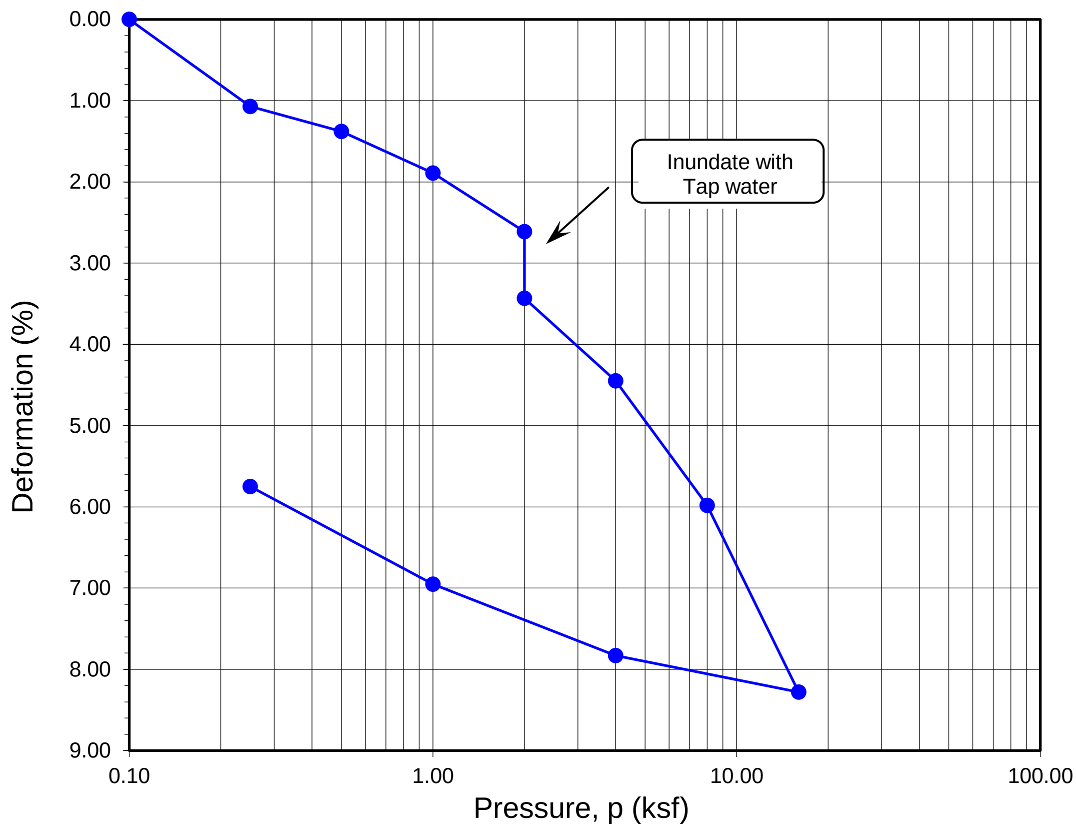


Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0000	1.0000	0.00	0.00	0.693	0.00
0.25	0.0111	0.9889	0.04	1.11	0.675	1.07
0.50	0.0147	0.9853	0.09	1.47	0.670	1.38
1.00	0.0208	0.9792	0.19	2.08	0.661	1.89
2.00	0.0280	0.9720	0.19	2.80	0.649	2.61
2.00	0.0373	0.9627	0.30	3.73	0.635	3.43
4.00	0.0486	0.9514	0.41	4.86	0.618	4.45
8.00	0.0653	0.9347	0.55	6.53	0.592	5.98
16.00	0.0900	0.9100	0.72	9.00	0.553	8.28
4.00	0.0839	0.9161	0.56	8.39	0.561	7.83
1.00	0.0739	0.9261	0.44	7.39	0.576	6.95
0.25	0.0608	0.9392	0.33	6.08	0.596	5.75

Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
3/6/23	8:15:00	0.0	0.0	0.0373
3/6/23	8:15:06	0.1	0.3	0.0446
3/6/23	8:15:15	0.2	0.5	0.0450
3/6/23	8:15:30	0.5	0.7	0.0453
3/6/23	8:16:00	1.0	1.0	0.0456
3/6/23	8:17:00	2.0	1.4	0.0459
3/6/23	8:19:00	4.0	2.0	0.0462
3/6/23	8:23:00	8.0	2.8	0.0463
3/6/23	8:30:00	15.0	3.9	0.0467
3/6/23	8:45:00	30.0	5.5	0.0469
3/6/23	9:15:00	60.0	7.7	0.0471
3/6/23	10:15:00	120.0	11.0	0.0473
3/6/23	12:15:00	240.0	15.5	0.0476
3/6/23	15:00:00	405.0	20.1	0.0480
3/7/23	7:03:00	1368.0	37.0	0.0486
3/7/23	7:03:00	1368.0	37.0	0.0486





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-5	**R-3**	**10**	**21.9**	**24.3**	**99.9**	**103.5**	**0.693**	**0.596**	**86**	**103**

Soil Identification: Lean Clay (CL), Olive Gray.

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

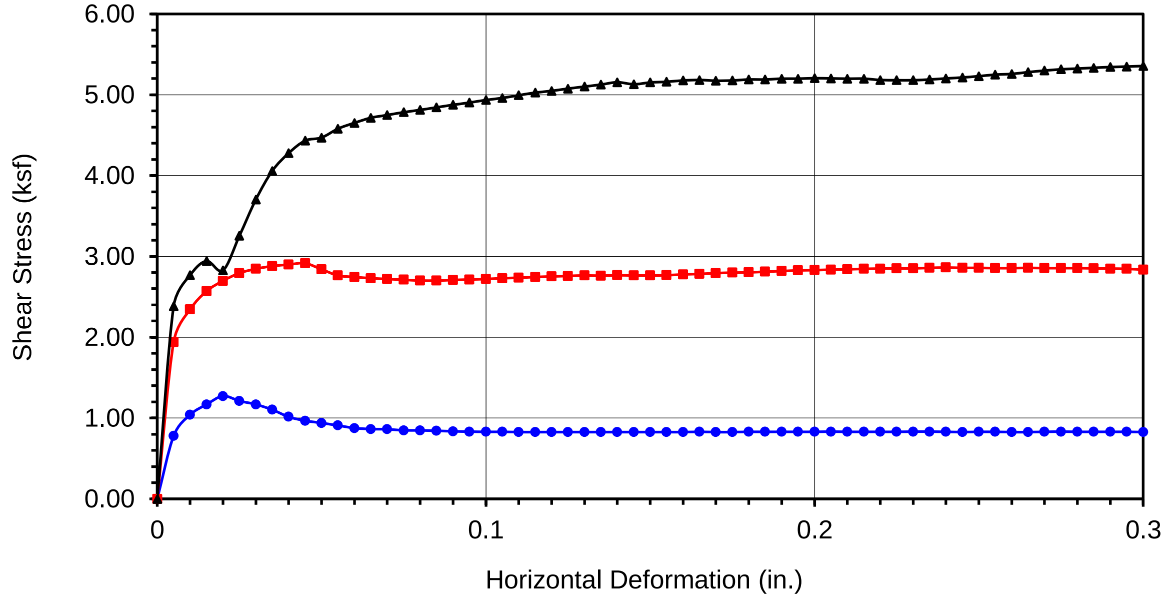
03-23

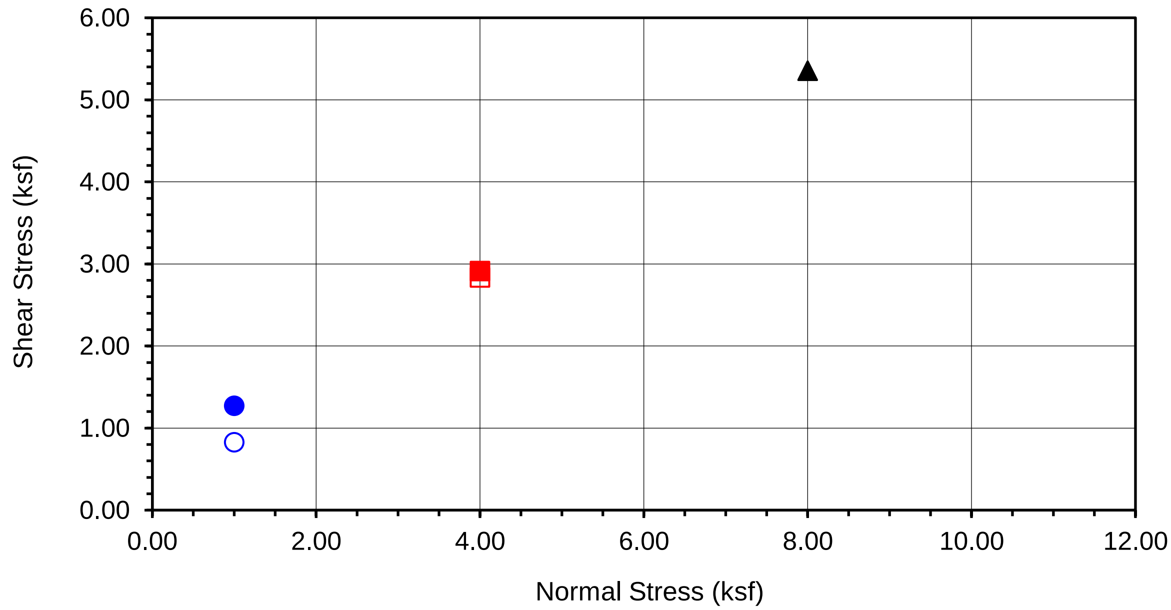
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: Rexford/13925 Benson Ave/Geo
Tested By: M. Vinet
Date: 02/23/23
Project No.: 13807.001
Checked By: M. Vinet
Date: 03/03/23
Boring No.: LB-4
Sample Type: 90% Remold
Sample No.: B-1
Depth (ft.): 0 - 5.0
Soil Identification: Silty Sand (SM), Very Dark Yellowish Brown.

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	193.01	197.16	196.23
Weight of Ring(gm):	41.80	45.52	45.84
Before Shearing			
Weight of Wet Sample+Cont.(gm):	288.64	288.64	288.64
Weight of Dry Sample+Cont.(gm):	262.89	262.89	262.89
Weight of Container(gm):	32.76	32.76	32.76
Vertical Rdg.(in): Initial	0.0000	0.2500	0.2500
Vertical Rdg.(in): Final	-0.0014	0.2624	0.2756
After Shearing			
Weight of Wet Sample+Cont.(gm):	209.45	210.53	208.40
Weight of Dry Sample+Cont.(gm):	185.68	186.34	184.32
Weight of Container(gm):	50.22	50.39	49.83
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 - 5.0**
Sample Type: 90% Remold	
Soil Identification: Silty Sand (SM), Very Dark Yellowish Brown.	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.272	■ 2.915	▲ 5.356
Shear Stress @ End of Test (ksf)	○ 0.826	□ 2.836	△ 5.356
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.19	11.19	11.19
Dry Density (pcf)	113.1	113.4	112.5
Saturation (%)	61.6	62.1	60.6
Soil Height Before Shearing (in.)	0.9986	0.9876	0.9744
Final Moisture Content (%)	17.5	17.8	17.9

Leighton

DIRECT SHEAR TEST RESULTS

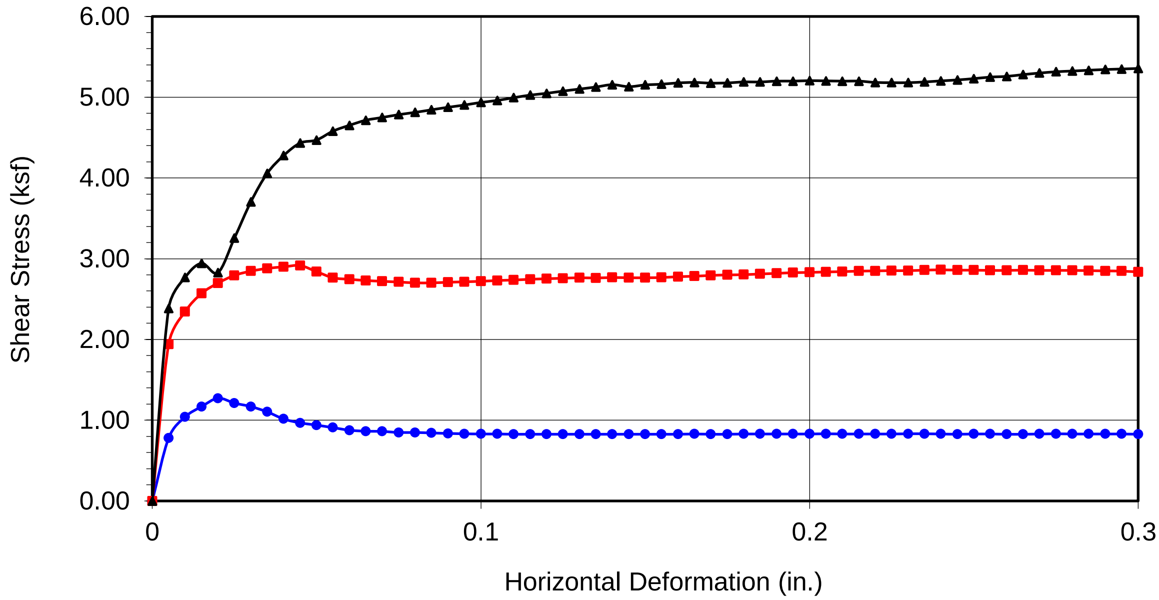
Consolidated Drained - ASTM D 3080

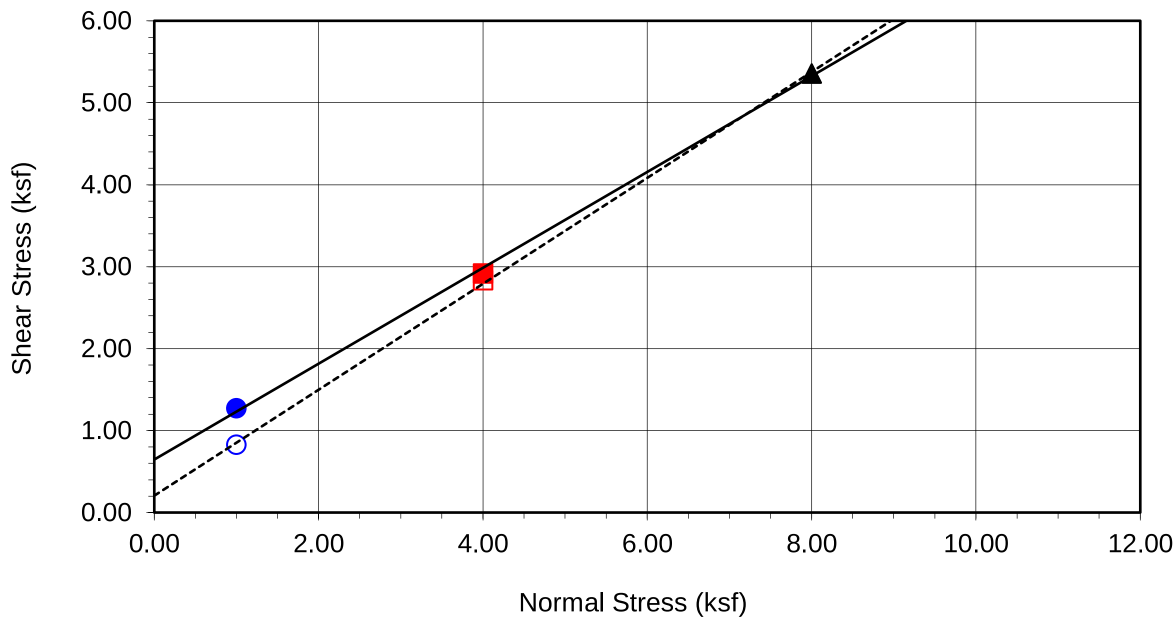
Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

02-23

Direct Shear; LB-4, B-1 (02-07-23)





Boring No.	**LB-4**
Sample No.	**B-1**
Depth (ft)	**0 - 5.0**
Sample Type:	90% Remold

Soil Identification:
Silty Sand (SM), Very Dark Yellowish Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	647	30
Ultimate	206	33

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.272	■ 2.915	▲ 5.356
Shear Stress @ End of Test (ksf)	○ 0.826	□ 2.836	△ 5.356
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.19	11.19	11.19
Dry Density (pcf)	113.1	113.4	112.5
Saturation (%)	61.6	62.1	60.6
Soil Height Before Shearing (in.)	0.9986	0.9876	0.9744
Final Moisture Content (%)	17.5	17.8	17.9

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

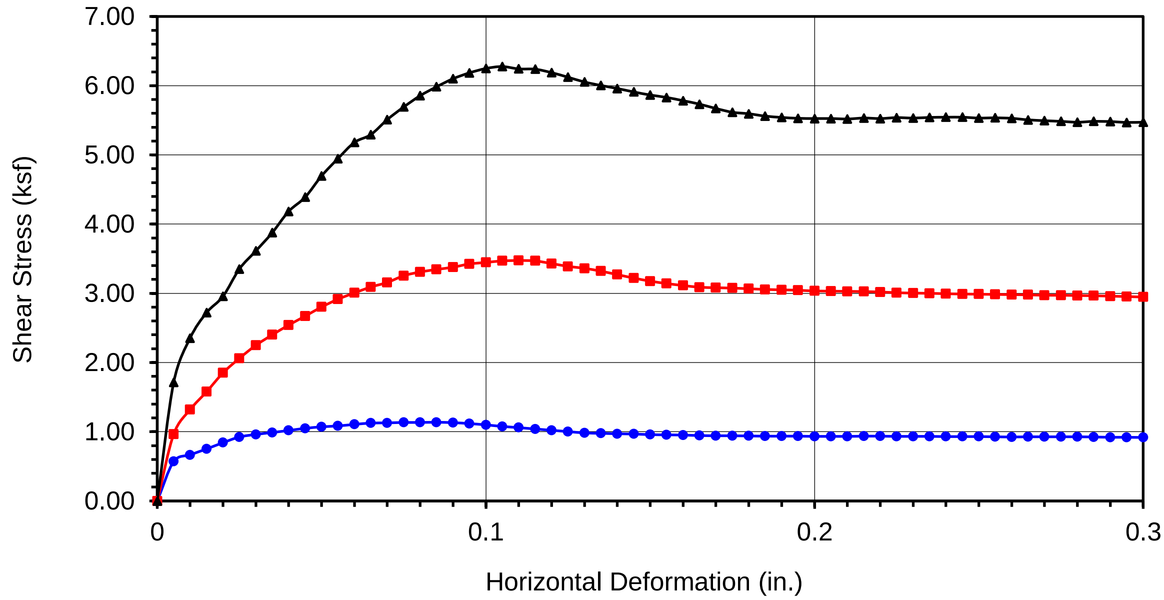
02-23

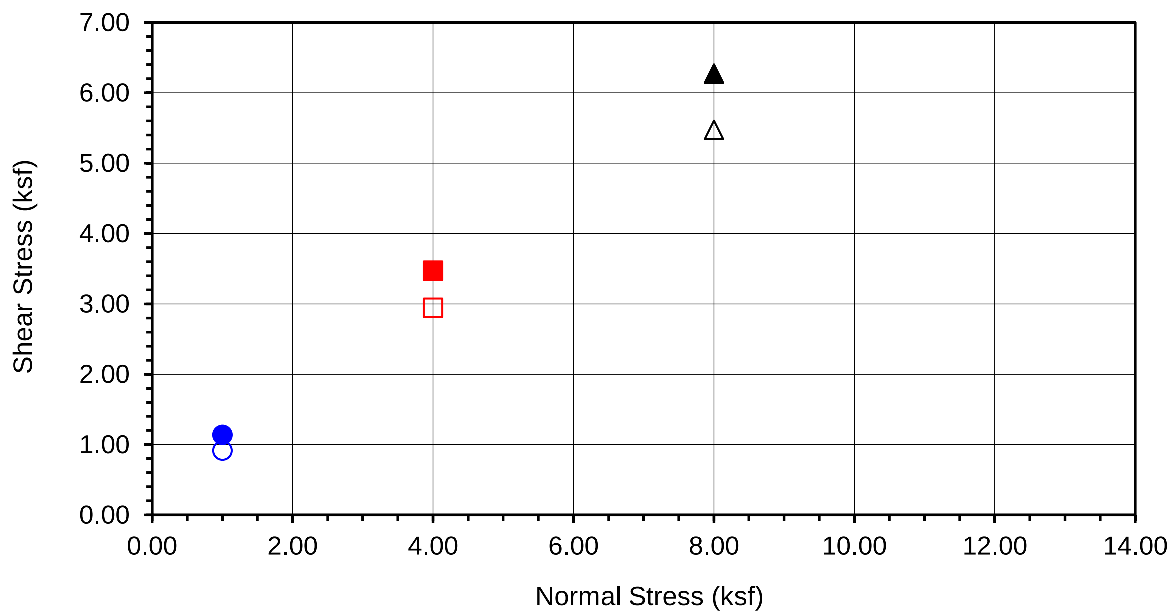
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Rexford/13925 Benson Ave/Geo	Tested By:	M. Vinet	Date:	02/22/23
Project No.:	13807.001	Checked By:	M. Vinet	Date:	03/03/23
Boring No.:	LB-4	Sample Type:	Ring		
Sample No.:	R-1	Depth (ft.):	5.0		
Soil Identification:	Sandy Silt s(ML), Brown.				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	188.23	194.86	206.87
Weight of Ring(gm):	44.50	46.07	54.46
Before Shearing			
Weight of Wet Sample+Cont.(gm):	192.22	192.22	192.22
Weight of Dry Sample+Cont.(gm):	171.43	171.43	171.43
Weight of Container(gm):	50.20	50.20	50.20
Vertical Rdg.(in): Initial	0.0000	0.2500	0.2500
Vertical Rdg.(in): Final	-0.0078	0.2784	0.2842
After Shearing			
Weight of Wet Sample+Cont.(gm):	198.30	200.79	206.13
Weight of Dry Sample+Cont.(gm):	173.27	177.48	184.49
Weight of Container(gm):	51.25	50.69	51.48
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type: Ring	
Soil Identification: Sandy Silt s(ML), Brown.	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.137	■ 3.474	▲ 6.276
Shear Stress @ End of Test (ksf)	○ 0.917	□ 2.946	△ 5.472
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 (%)	17.15	17.15	17.15
Dry Density (pcf)	102.0	105.6	108.2
Saturation (%)	71.0	77.7	83.0
Soil Height Before Shearing (in.)	0.9922	0.9716	0.9658
Final Moisture Content (%)	20.5	18.4	16.3

Leighton

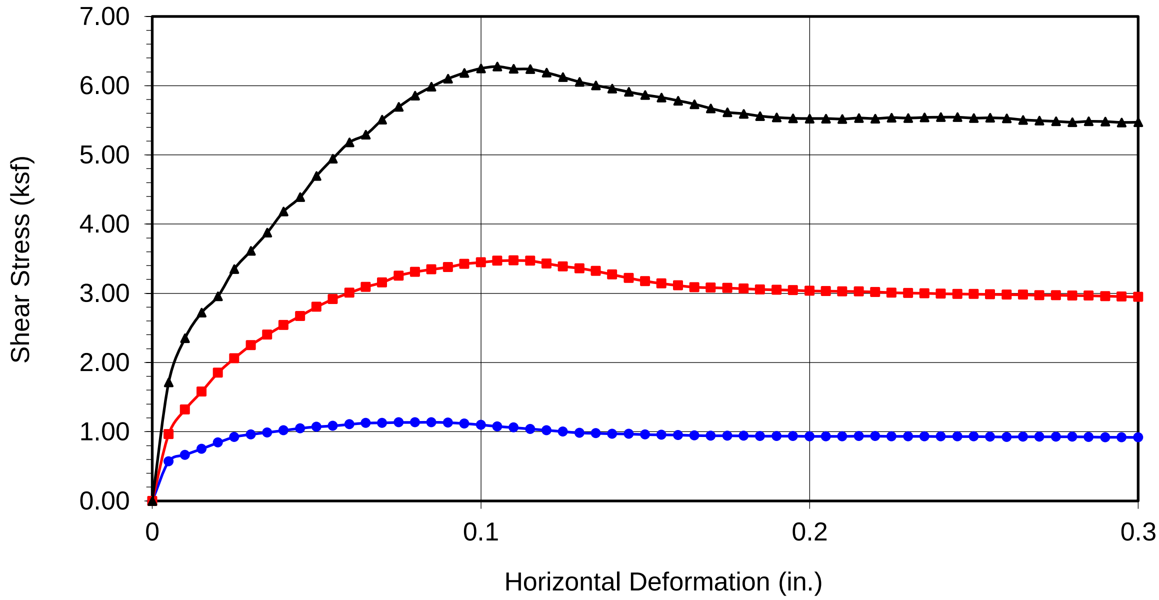
DIRECT SHEAR TEST RESULTS

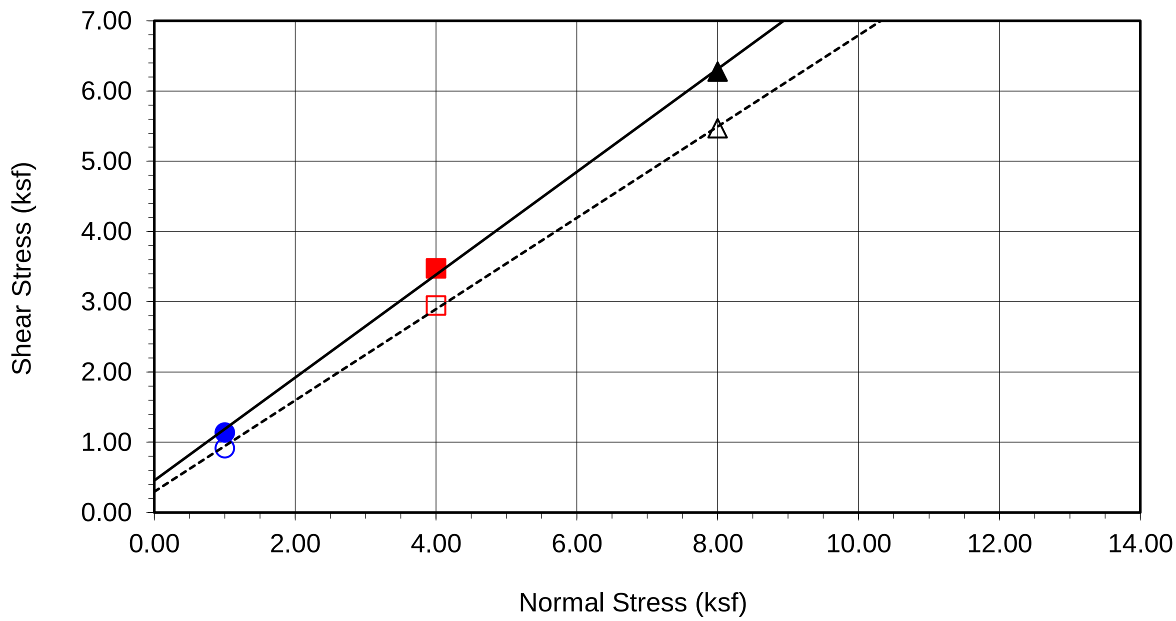
Consolidated Drained - ASTM D 3080

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

02-23





Boring No.	**LB-4**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:
Sandy Silt s(ML), Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	456	36
Ultimate	296	33

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.137	■ 3.474	▲ 6.276
Shear Stress @ End of Test (ksf)	○ 0.917	□ 2.946	△ 5.472
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 (%)	17.15	17.15	17.15
Dry Density (pcf)	102.0	105.6	108.2
Saturation (%)	71.0	77.7	83.0
Soil Height Before Shearing (in.)	0.9922	0.9716	0.9658
Final Moisture Content (%)	20.5	18.4	16.3

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

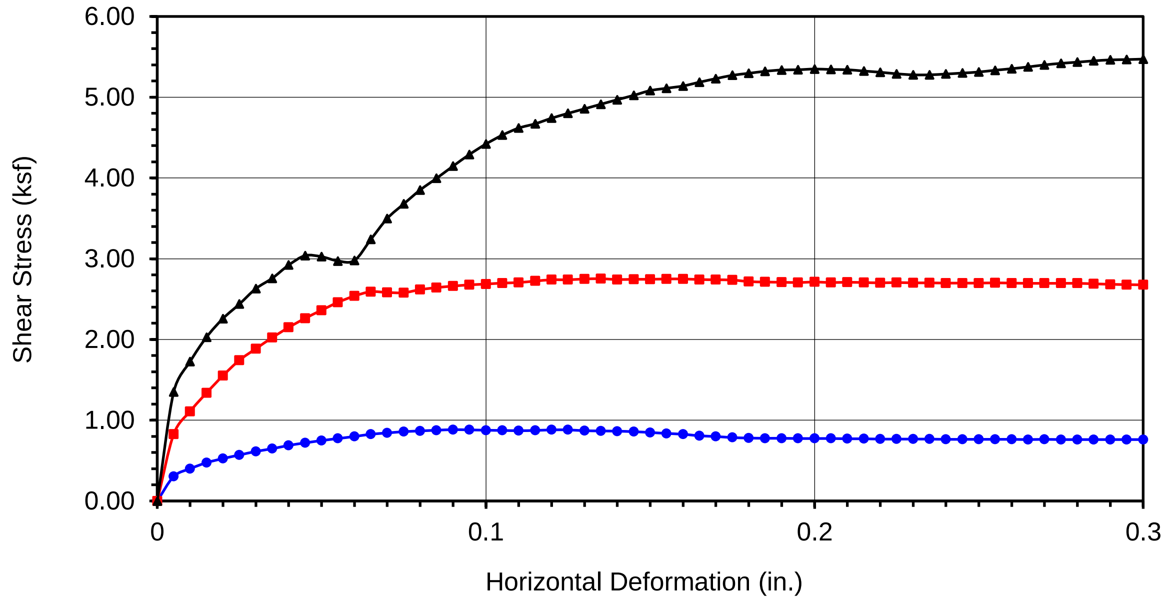
02-23

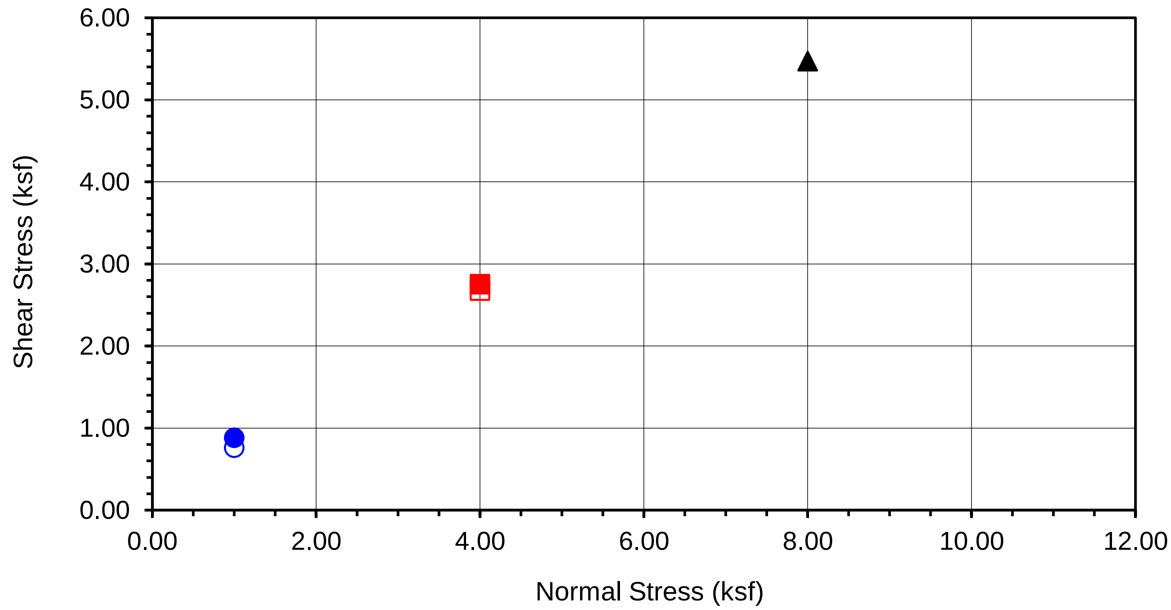
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: Rexford/13925 Benson Ave/Geo
Tested By: M. Vinet
Date: 02/23/23
Project No.: 13807.001
Checked By: M. Vinet
Date: 03/03/23
Boring No.: LB-4
Sample Type: Ring
Sample No.: R-3
Depth (ft.): 10.0
Soil Identification: Sandy Silt s(ML), Brown.

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	186.56	191.96	195.26
Weight of Ring(gm):	37.67	43.91	45.66
Before Shearing			
Weight of Wet Sample+Cont.(gm):	185.26	185.26	185.26
Weight of Dry Sample+Cont.(gm):	162.56	162.56	162.56
Weight of Container(gm):	50.56	50.56	50.56
Vertical Rdg.(in): Initial	0.0000	0.2500	0.2500
Vertical Rdg.(in): Final	-0.0061	0.2828	0.3004
After Shearing			
Weight of Wet Sample+Cont.(gm):	201.66	198.77	200.73
Weight of Dry Sample+Cont.(gm):	171.88	170.22	172.32
Weight of Container(gm):	50.59	50.55	49.79
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**R-3**
Depth (ft)	**10**
Sample Type: Ring	
Soil Identification: Sandy Silt s(ML), Brown.	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.883	■ 2.752	▲ 5.472
Shear Stress @ End of Test (ksf)	○ 0.760	□ 2.676	△ 5.472
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 (%)	20.27	20.27	20.27
Dry Density (pcf)	103.0	102.4	103.4
Saturation (%)	85.9	84.6	86.9
Soil Height Before Shearing (in.)	0.9939	0.9672	0.9496
Final Moisture Content (%)	24.6	23.9	23.2

Leighton

DIRECT SHEAR TEST RESULTS

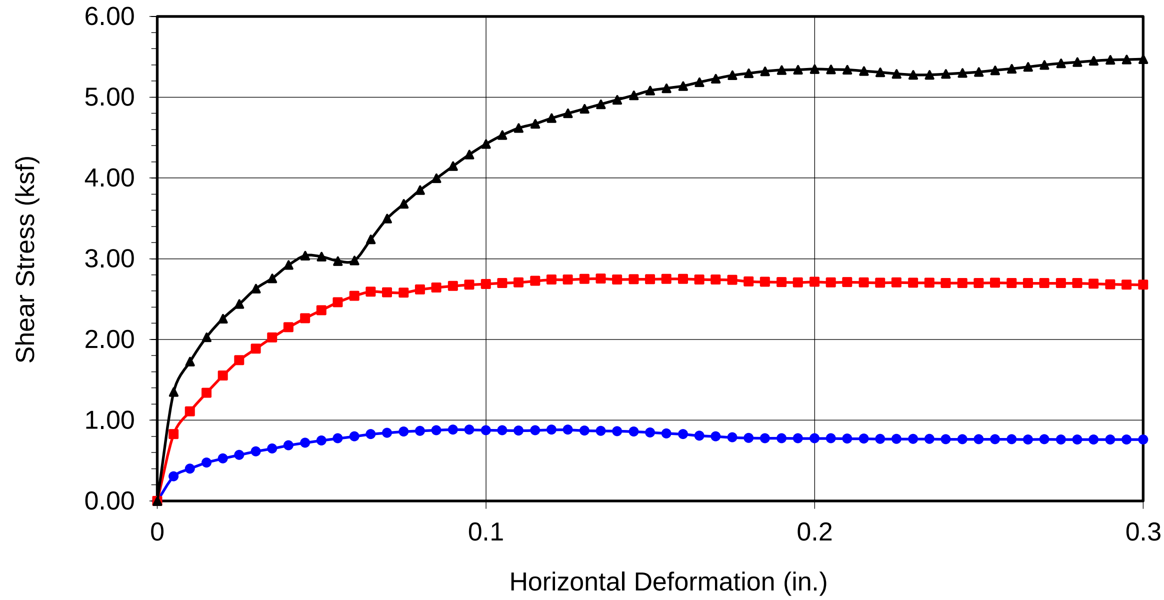
Consolidated Drained - ASTM D 3080

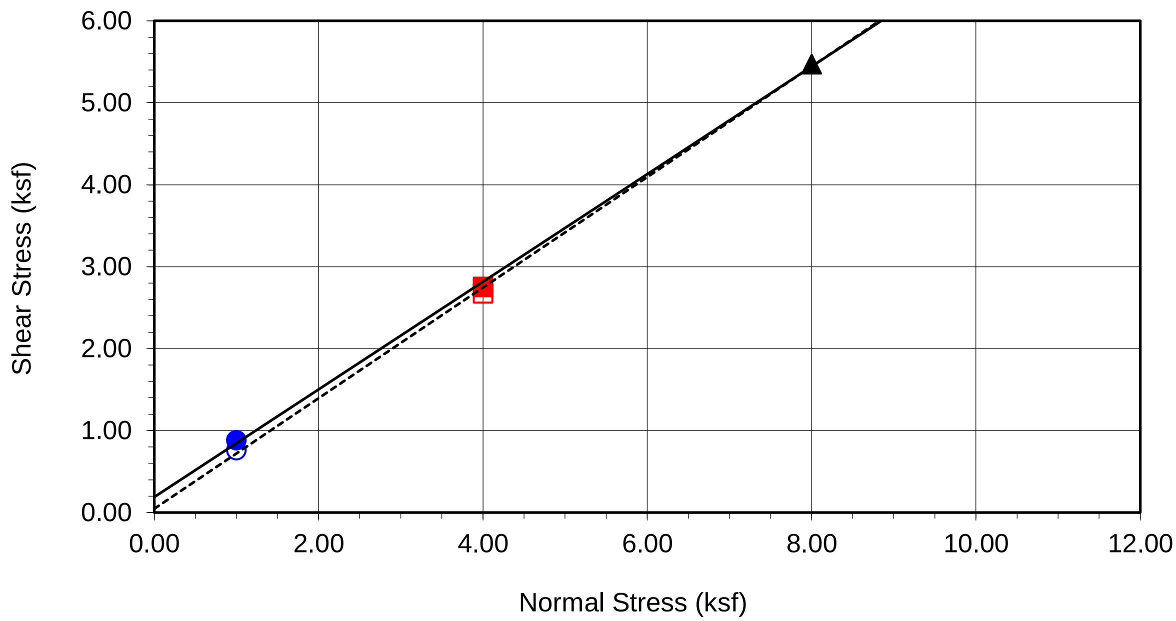
Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

02-23

Direct Shear; LB-4, R-3 (02-07-23)





Boring No.	**LB-4**
Sample No.	**R-3**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Sandy Silt s(ML), Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	189	33
Ultimate	46	34

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.883	■ 2.752	▲ 5.472
Shear Stress @ End of Test (ksf)	○ 0.760	□ 2.676	△ 5.472
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 (%)	20.27	20.27	20.27
Dry Density (pcf)	103.0	102.4	103.4
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Soil Height Before Shearing (in.)	0.9939	0.9672	0.9496
Final Moisture Content (%)	24.6	23.9	23.2

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13807.001

Rexford/13925 Benson Ave/Geo

02-23

Direct Shear; LB-4, R-3 (02-07-23)

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Rexford/13925 Benson Ave/Geo | Tested By : M. Vinet | Date: 02/27/23

Project No. : 13807.001 | Checked By: M. Vinet | Date: 03/03/23

Boring No.	LB-4			
Sample No.	B-1			
Sample Depth (ft)	0 - 5.0			
Soil Identification:	Silty Sand (SM)			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.0399			
Wt. of Crucible (g)	25.0361			
Wt. of Residue (g) (A)	0.0038			
PPM of Sulfate (A) x 41150	156.37			
PPM of Sulfate, Dry Weight Basis	**156**			

CHLORIDE CONTENT, DOT California Test 422

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

pH TEST, DOT California Test 643

pH Value	**7.80**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 643

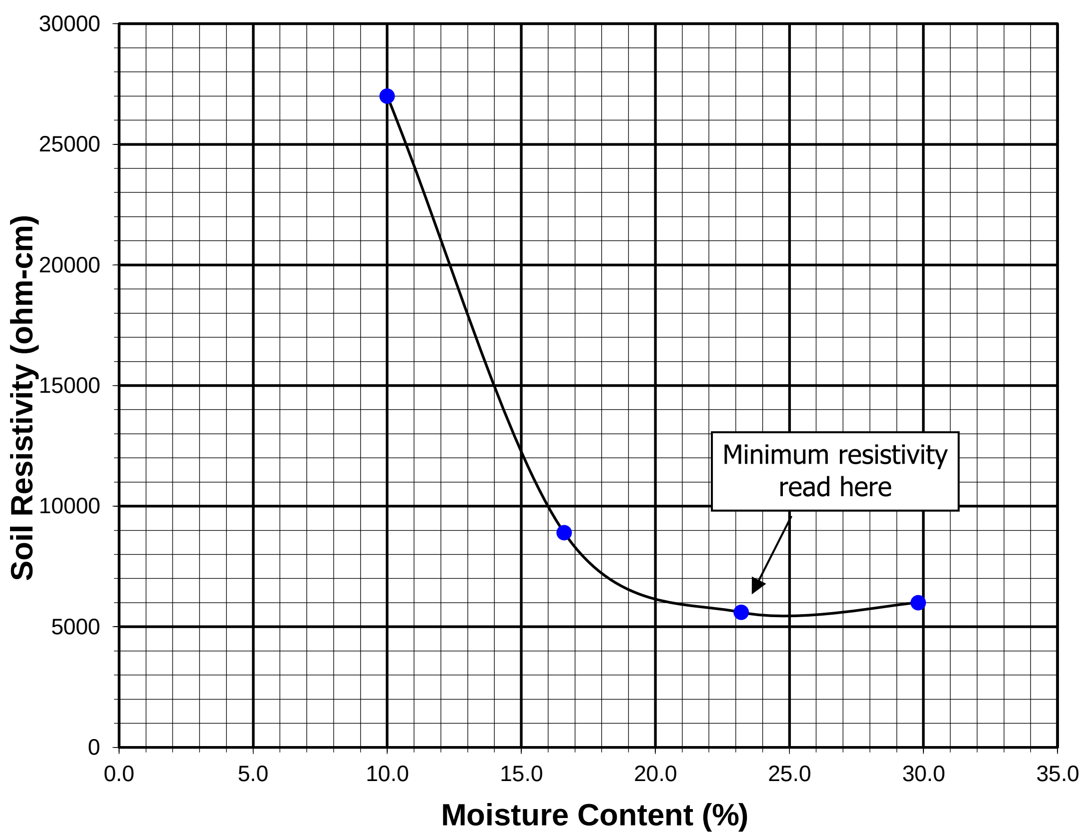
Project Name:	Rexford/13925 Benson Ave/Geo	Tested By :	M. Vinet Date: 02/28/23
Project No. :	13807.001	Checked By:	M. Vinet Date: 03/03/23
Boring No.:	LB-4	Depth (ft.) :	0 - 5.0
Sample No. :	B-1		
Soil Identification:*	Silty Sand (SM)		

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	27000	27000
2	83	16.60	8900	8900
3	116	23.20	5600	5600
4	149	29.80	6000	6000
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
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	
5600	**23.2**	**156**	**40**	**7.80**	**21.0**



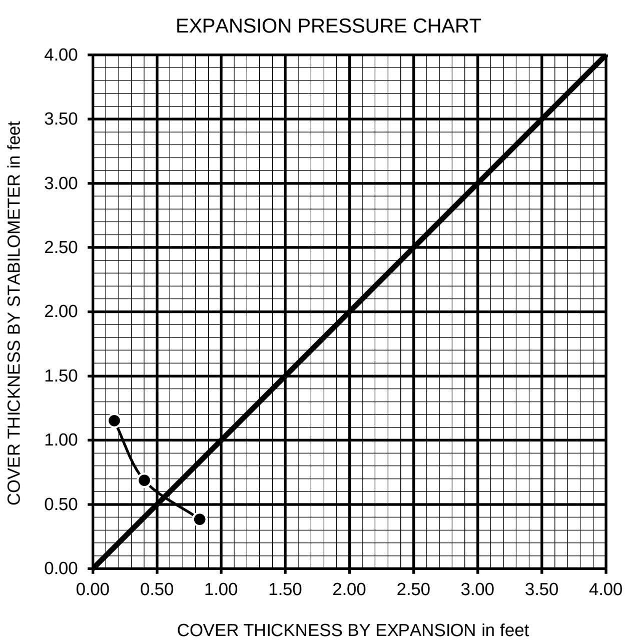
Leighton

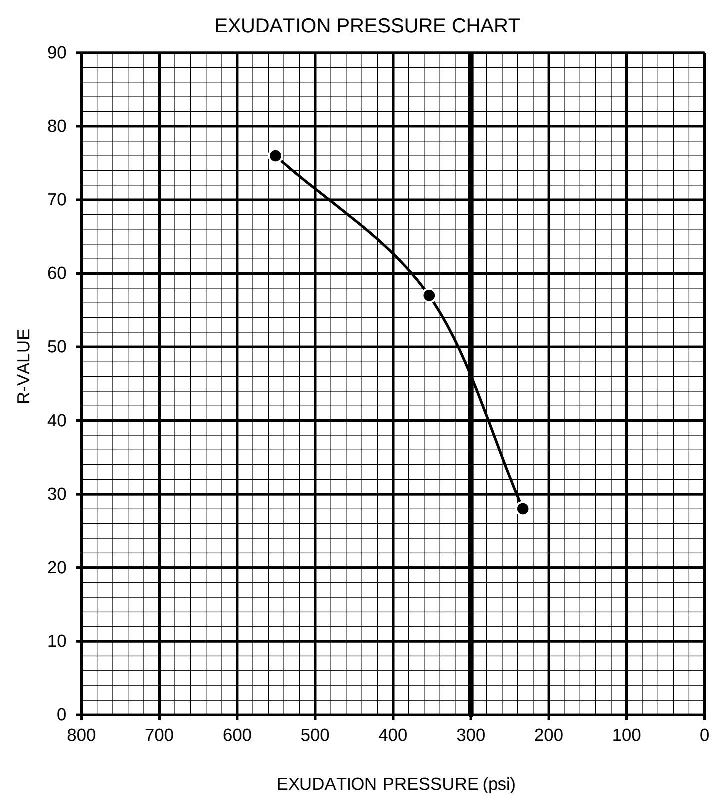
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Rexford/13925 Benson Ave/Geo	PROJECT NUMBER:	13807.001
BORING NUMBER:	LB-4	DEPTH (FT.):	0 - 5.0
SAMPLE NUMBER:	B-1	TECHNICIAN:	F. Mina
SAMPLE DESCRIPTION:	Silty Sand (SM), Very Dark Yellowish Brown.	DATE COMPLETED:	2/28/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.0	12.0	13.0
HEIGHT OF SAMPLE, Inches	2.49	2.51	2.55
DRY DENSITY, pcf	112.7	112.3	110.9
COMPACTOR PRESSURE, psi	250	225	200
EXUDATION PRESSURE, psi	551	354	233
EXPANSION, Inches x 10exp-4	25	12	5
STABILITY Ph 2,000 lbs (160 psi)	23	45	90
TURNS DISPLACEMENT	4.61	4.77	4.90
R-VALUE UNCORRECTED	76	57	28
R-VALUE CORRECTED	76	57	28

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.38	0.69	1.15
EXPANSION PRESSURE THICKNESS, ft.	0.83	0.40	0.17





R-VALUE BY EXPANSION:	66
R-VALUE BY EXUDATION:	46
EQUILIBRIUM R-VALUE:	46

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

D-1.0 GENERAL

D-1.1 Intent

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

D-1.2 Role of Leighton Consulting, Inc.

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

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

D-1.3 The Earthwork Contractor

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

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

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

D-2.0 PREPARATION OF AREAS TO BE FILLED

D-2.1 Clearing and Grubbing

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

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

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

D-2.2 Processing

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

D-2.3 Overexcavation

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

D-2.4 Benching

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

D-2.5 Evaluation/Acceptance of Fill Areas

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

D-3.0 FILL MATERIAL

D-3.1 Fill Quality

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

D-3.2 Oversize

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

D-3.3 Import

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

D-4.0 FILL PLACEMENT AND COMPACTION

D-4.1 Fill Layers

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

D-4.2 Fill Moisture Conditioning

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

D-4.3 Compaction of Fill

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

D-4.4 Compaction of Fill Slopes

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

D-4.5 Compaction Testing

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

D-4.6 Compaction Test Locations

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

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

D-5.0 EXCAVATION

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

D-6.0 TRENCH BACKFILLS

D-6.1 **Safety**

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

D-6.2 **Bedding and Backfill**

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

D-6.3 Lift Thickness

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