GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
14005 LIVE OAK AVENUE
IRWINDALE, CALIFORNIA

Prepared For

REXFORD INDUSTRIAL REALTY & MANAGEMENT, INC.
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Project Number 13384.001

March 21, 2023

Leighton

Leighton Consulting, Inc.
A Leighton Group Company

March 21, 2023

Project No. 13384.001

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

Attention: Mr. James Hwang

Subject: Geotechnical Exploration Report
Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

Per your request and authorization, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. We understand the proposed development concept for the site includes demolition of the existing building and site improvements to allow construction of a new one-story concrete tilt-up industrial building. The proposed building has a total building area of 102,400 square feet and will be constructed at grade in the western portion of the site with dock-high truck loading on the eastern side of the building. Associated trailer parking and surface parking are also planned in the northeastern and eastern portions of the site. Ancillary improvements will likely consist of utility infrastructure, pavement, flatwork, and landscaping.

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

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

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

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 14005 Live Oak Avenue in the city of Irwindale, California. The site location (latitude 34.107659°, longitude -117.968701°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

The project site is irregular in shape and covers approximately 5.1 acres. The site is bordered by Live Oak Avenue to the south, Stewart Avenue to the west, Rivergrade Road to the northwest, an existing paved parking lot to the northeast, and a vacant lot to the east. Access to the site is via Live Oak Avenue from the south, Stewart Avenue from the west and Rivergrade Road from the northwest. The site is currently occupied by one (1) existing 2-story building located in the central portion of the site and associated paved surface parking and access.

The project site is relatively flat with sheet flow generally directed to the southwest over paved surfaces to curbs and gutters. Review of the United States Geological Survey (USGS) 7.5-Minute Baldwin Park Quadrangle (USGS, 2018) indicates the site is between approximately Elevation (El.) +410 feet and +420 feet mean sea level (msl).

Based on review of historic aerial photographs, the project site appears to have been occupied since at least 1948 as several structures were present in and around the site boundaries during this time (NETR, 2022). Between approximately 1980 and 1992, many of the previously existing structures appear to have been removed, and by 1993 the existing commercial building appears to have been constructed in its current configuration.

Based on review of the *Site Plan* for the project, dated December 22, 2022, we understand the proposed development for the site includes site demolition of the existing building and site improvements to allow construction of a new one-story concrete tilt-up industrial building. The proposed building has a total building area of 102,400 square feet and will be constructed at grade in the western portion of the site with dock-high truck loading on the eastern side of the building. Associated trailer parking and surface parking are also planned in the northeastern and eastern portions of the site. Ancillary improvements will likely consist of utility infrastructure, pavement, flatwork, and landscaping.

1.2 Purpose and Scope

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

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

- *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 January 4, 2022, and included drilling, logging, and sampling of three (3) hollow-stem auger borings (designated LB-1 through LB-3) to depths between approximately 12 and 20 feet below the existing ground surface (bgs). Two (2) additional borings (designated LP-1 and LP-2) were drilled to approximate depths between 5.4 and 10 feet bgs for subsequent percolation testing. It should be noted that borings LB-1 through LB-3 and LP-1 encountered practical drilling refusal at various depths between approximately 5.4 and 20 feet bgs, likely due to oversized materials such as cobbles or boulders. The approximate locations of the explorations are shown on Figure 2, *Exploration Location Map.* The boring logs are presented in Appendix A, *Exploration Logs.*

  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 2.5-foot to 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, borings LB-1 through LB-3 were backfilled to the ground surface with soils generated during the exploration and patched with cold-mix asphalt concrete to match existing surface conditions. Excess soil cuttings from the borings were spread in planter areas.

- *Percolation Testing* – Borings LP-1 and LP-2 (Figure 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 January 5, 2022 in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation* and Reporting, *Low Impact Development Stormwater Infiltration* (LADPW, 2021). 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);
  - Direct Shear (ASTM D3080);
  - Consolidation (ASTM D 2435);
  - Maximum Dry Density (ASTM D1557);

- Expansion Index (ASTM D4829);
- Particle-Size Distribution (ASTM D 6913);
- 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*

- *Geophysical Evaluation* – Due to relatively shallow refusal of the hollow-stem auger borings advanced at the site, a geophysical evaluation was performed at the site by Atlas Technical Consultants, LLC (Atlas, 2022) to evaluate the subsurface soils at depth and to develop a shear-wave velocity profile of the subsurface earth materials at the site to definitively classify the Site Class for seismic design. The geophysical survey consisted of performing two (2) refraction microtremor (ReMi) profiles approximately 230 feet in length in the northern and western portions of the site. The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a shear-wave velocity profile of the study area down to a depth of approximately 100 feet. The locations of the ReMi profiles are shown on Figure 2, *Exploration Location Map*, and a copy of the report prepared by Atlas (2022) documenting the results of the geophysical evaluation is included in Appendix D, *Geophysical Evaluation* (Atlas, 2022).

- *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 preliminary recommendations for the proposed warehouse development.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

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

The San Gabriel Valley is an almost enclosed basin drained by the Rio Hondo and San Gabriel Rivers. Sediments within the Valley include about 6,000 feet of marine and non-marine sedimentary rocks of Quaternary age (Yerkes, et al. 1965). The San Gabriel Valley is bound on the north by the San Gabriel Mountains, on the northwest by the Verdugo Mountains and Raymond Hill fault, on the east and southeast by the San Jose Hills, and on the south by the Puente Hills, Montebello Hills and Repetto Hills.

2.2 Surficial Geology

The site is located approximately 1,000 feet to the southeast of the channelized San Gabriel River and immediately southwest of the Santa Fe Dam in an area geologically mapped to be underlain by Quaternary-age (Holocene) young alluvial fan deposits generally consisting of unconsolidated to slightly consolidated coarse-grained sand to boulder sediments (Morton and Miller, 2006; Dibblee Jr., 1999). 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 thin layer of undocumented artificial fill materials (Afu) overlying Quaternary-aged alluvial gravel and sand (Qg). The artificial fill encountered in our borings at the explored locations is generally about 2 feet in thickness across the site, likely associated with existing and previous site improvements. The fill soils consist primarily of locally derived silty sand with gravel. Localized thicker accumulations of the fill materials should be anticipated between explored locations during future earthwork construction, in particular beneath the existing building.

Below the veneer of artificial fill, Quaternary-aged alluvial sand and gravel deposits (Qg) were encountered in the borings to the maximum depth explored of 20 feet bgs. The alluvial deposits encountered generally consist of light gray, brown to brown, slightly moist to moist, very dense gravelly sand, sandy gravel, and gravel. It should be noted that borings LB-1 through LB-3 and LP-1 encountered practical drilling refusal at various depths between approximately 5.4 and 20 feet bgs, likely due to oversized materials such as cobbles or boulders.

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, *Exploration Location Map*.

2.3.1 Expansive Soil Characteristics

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

One (1) near-surface soil sample obtained during our subsurface exploration was tested for expansion potential. The test results indicate an Expansion Index (EI) value of 0 (“very low” potential for expansion). The Expansion Index laboratory test results are included in Appendix C of this report.

Expansive soils will likely not impact the proposed construction. 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 bulk soil sample of the onsite soils recovered as a part of 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 119 parts per million (ppm), chloride content of 120 ppm, pH value of 8.63, and minimum resistivity value of 3,600 ohm-cm.

The results of the resistivity tests indicate the underlying soil is moderately corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil samples, concrete in contact with the soil is expected to have negligible exposure to sulfate attack 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.

2.3.3 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing. The results of testing are included in Appendix C as well as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.3.4 Shear Wave Velocity

Based on the results of the geophysical survey performed the site by Atlas (2022), the average characteristic site shear-wave velocity of the subsurface earth materials down to a depth of 100 feet ranges between 1,598 and 1,648 feet per second (f/s) and correspond to Site Class C per the 2022 California Building Code (CBC). A copy of the report prepared by Atlas (2022) documenting the results of the geophysical evaluation is included in Appendix D, *Geophysical Evaluation* (Atlas, 2022).

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 consist of layers that contain granular, unconsolidated soils with little or no cementation and very few fines with varying proportions of gravel and cobbles. These materials are prone to cave in or collapse in unshored excavations. See Section 3.8, *Temporary Excavations* for additional information on soil type and excavation characteristics.

In addition, oversized materials greater than 6 inches in diameter (i.e. cobbles and/or boulders) will be encountered during excavation. Oversized materials larger than 6 inches in diameter encountered during site grading will 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. *It is important that a contractor with excavation experience in similar conditions should be consulted for the proper handling of oversized materials*.

2.4 Groundwater Conditions

Groundwater was not encountered in any of the explorations performed at the site to the maximum depth explored of 20 feet bgs. Based on the Seismic Hazard Zone Report for the Baldwin Park Quadrangle (CGS, 1998), the historically shallowest groundwater depth is reported to be approximately 100 and 150 feet bgs. In addition, based on review of available groundwater information from the California Department of Water Resources for a nearby groundwater monitoring well located immediately to the east of the project site (Station 341074N1179660W001), the shallowest groundwater level measured for a monitoring period between July 2011 and January 2013 was approximately 178.4 feet bgs. Therefore, based on these findings, groundwater is not expected to pose a constraint during or after construction.

2.4.1 Infiltration

Percolation testing was performed within temporary percolation wells installed in borings LP-1 and LP-2 to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation and Reporting Low*

Impact Development Stormwater Infiltration (LADPW, 2021). Results of the percolation testing are presented in Appendix B, *Percolation Test Data*. The test locations and zones tested are shown on Figure 2, *Exploration Location Map*.

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 near-surface infiltration devices.

A falling-head test method was employed for test well LP-1 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.

A constant-head test, or high flowrate test, was implemented at test well LP-2 due to the permeable characteristics of the site soils. The infiltration rate was calculated by recording the approximate volume of water delivered to the test zone while maintaining a relatively constant height of water in the well over the testing period. A water source (garden hose from onsite water source) was used to deliver water to the wells at a relatively constant rate. The measured infiltration rate was calculated according to the procedure for a high flowrate percolation test, by dividing the total volume of water by the total duration of the test and dividing by the percolation surface area.

The design infiltration rate incorporates a reduction factor for the test procedure, site variability, number of tests, thoroughness of subsurface investigation and long-term siltation, plugging and maintenance. As such, we have applied an appropriate reduction factor to the small-scale infiltration rates measured at test wells LP-1 and LP-2 for use in design of the system(s) in accordance with the County of Los Angeles Guidelines (LADPW, 2021). In addition, based on the variability of the results and the unknown location and depth of the planned stormwater infiltration device(s), additional testing may be required.

Detailed results of the field testing data, measured infiltration rate and design infiltration rate for the tests performed are presented in Appendix B, *Percolation Test Data*. 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 (inch per hour)	Design Infiltration Rate (inch per hour)
LP-1	2 to 5	3.82	1.27
LP-2	5 to 10	17.8	3.57

Based on County of Los Angeles requirements, the design infiltration rates at the tested locations and depths for both LP-1 and LP-2 meet the minimum feasibility criteria of 0.3 inch per hour. Therefore, based on the results of the testing presented above, stormwater infiltration **is** feasible at the project 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, 2008). The closest active faults to the site with the potential for surface fault rupture are the Sierra Madre fault, Raymond fault, Clamshell-Sawpit fault and San Jose fault, located approximately 3.2 miles, 4.0 miles, 5.1 miles and 6.9 miles from the site, respectively. The San Andreas fault, which is the largest active fault in California, is approximately 23 miles northeast of the site on the north side of the San Gabriel Mountains. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.6 Strong Ground Shaking

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

Accordingly, design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). The 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	34.107659°
Site Longitude	-117.968701°
Site Class	C
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.709 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.6 g
Short Period (0.2 sec) Site Coefficient, F_a	1.2
Long Period (1 sec) Site Coefficient, F_v	1.4
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	2.051 g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	0.84 g
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.367 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.56 g
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.875 g

2.7 Liquefaction Potential

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose and medium dense, near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

As shown on Figure 5, *Seismic Hazard Map*, the site is **not** mapped within a liquefaction hazard zone as delineated by the State of California (CGS, 1999). In addition, the historically shallowest groundwater depth is reported to be approximately 100 to 150 feet bgs (CGS, 1998). 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 site soils below the anticipated bearing grade of the proposed structure, the potential total earthquake-induced settlement is estimated to be less than 1 inch. The differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

2.9 Lateral Spreading

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

2.10 Earthquake-Induced Landsliding

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

2.11 Flooding

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

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. As shown on Figure 7, *Dam Inundation Map*, the site **is** mapped within an inundation zone associated with the Santa Fe Dam. However, the safety of dams and levees is regulated by the California Department of Water Resources (DWR), Division of Safety of Dams (DSOD) and Army Corp of Engineers (ACOE) and this dam is under continuous monitoring for safety against failure.

2.12 Seiches and Tsunamis

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

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

Based upon 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/structure footprint areas prior to placement of engineered fill. Oversized materials greater than 6 inches in diameter (i.e. cobbles and/or boulders) will be encountered during excavation, and will require special handling.

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

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

3.1 Site Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. The site soils include layers that contain granular, unconsolidated soils with little or no cementation and very few fines with varying proportions of gravel and cobbles. These materials are prone to cave in or collapse in unshored excavations.

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 areas 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. Removals and overexcavations should be performed such that all undocumented fill is removed. Deeper overexcavations in localized areas may be recommended during grading by a representative of the geotechnical engineer depending on observed subsurface conditions.

3.1.3 Excavation Bottom Preparation

All excavation or removal bottoms should be observed by a representative of the geotechnical engineer prior to placement of fill or other improvements to determine that geotechnically suitable soil is exposed. Excavation bottoms observed to be suitable for fill placement or other improvements should be scarified to a depth of at least 8 inches, moisture-conditioned as necessary to achieve a moisture content within 2 percentage points of the optimum moisture content, and then compacted to a minimum of 95 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor).

Gravelly and cobbly soils are anticipated at the bottom of excavations deeper than 5 feet bgs. It will be difficult to establish and maintain a smooth subgrade in gravelly and cobbly soils. In these types of soil, fill or other improvements may be constructed directly over a rough subgrade free of disturbed materials. A working surface may be established about 6 inches above the design subgrade elevation to accommodate removal of disturbed materials prior to pouring concrete directly over the rough subgrade.

3.1.4 Fill Materials

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

3.1.6 Shrinkage

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

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

3.1.7 Reuse of Concrete and Asphalt Rubble

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

3.2 Foundation Design

Conventional spread footings established in engineered fill or undisturbed natural soils may be used to support the proposed building. Footings should be embedded a minimum 18 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 18 inches for continuous footings and 24 inches for isolated footings.

A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used. The 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. For calculating lateral resistance, a passive pressure of 300 psf per foot of depth to a maximum of 3,000 psf and a frictional coefficient of 0.30 may be used. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

3.3 Slabs-on-Grade

Unloaded concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 3.1. From a geotechnical standpoint, we recommend slab-on-grade be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 24 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil-thick vapor barrier between the slab and subgrade. A coefficient of friction of 0.35 can be used between the floor slab and the vapor barrier.

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

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

3.4 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with 2022 CBC requirements. However, concrete exposed to recycled water should be designed using Type V cement.

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

3.5 Retaining Walls

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

On-site soils are likely suitable to be used as retaining wall backfill due to its 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 for reuse behind retaining walls should be tested to ensure Expansion potential is less than 20 (EI<20). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 8, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 3 – Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	35
At-Rest (braced)	60
Passive Resistance (compacted fill)	300
Seismic Increment (add to active pressure)	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 (Figure 8). 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 (Green Book), 2021 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in

Section 300-8.1 of the Standard Specifications for Public Works Construction (Green Book), 2021 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

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

3.6 Paving

To provide support for paving, the subgrade soils should be prepared as recommended in the Section 3.1. Compaction of the subgrade, including trench backfills, to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course. 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 50, compacted to at least 90 percent as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on near surface samples of existing onsite soils indicate a value of 75.

Table 4 – Asphalt Concrete Pavement Sections

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

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

3.6.2 Portland Cement Concrete Paving

We have assumed that such a subgrade will have an R-value of at least 50, 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 3,000 pounds per square inch.

Table 5 – PCC Pavement Sections

Traffic Index	PCC (inches)	Base Course (inches)
5	5	4
6	6	4
7	6½	4
8	7	4
9	8	4

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

3.6.3 Base Course

The base course for both asphalt concrete and Portland cement concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

3.7 Infiltration BMP Design Considerations

Per County of Los Angeles *Guidelines for Geotechnical Investigation and Reporting – Low Impact Development Stormwater Infiltration* (LADPW, 2021), we have applied a reduction factor to the measured infiltration rate to be used in design of the infiltration system. The infiltration rates measured at the site are required to be reduces for design of the infiltration system per County requirements (LADPW, 2021) based on the test procedure, site variability, number of tests, thoroughness of subsurface investigation, long term siltation, plugging and maintenance. The design infiltration rates at the tested locations and depths for both LP-1 and LP-2 are 1.27 and 3.57 inches per hour, respectively. Due to the variable infiltration rates that were measured from the two tests performed at the site, the lower measured rate should be considered for design purposes and additional testing may be warranted to confirm the actual infiltration rate at the location and depth of the planned infiltration device.

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

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

3.8 <u>Temporary Excavations</u>

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.

The site soils include layers that contain granular, unconsolidated soils with little or no cementation and very few fines with varying proportions of gravel and cobbles. These materials are prone to cave in or collapse in unshored excavations. Onsite soils are to be considered Type C soils which are subject to collapse in shallow unbraced excavations (i.e. approximately 3 feet in vertical height).

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

3.9 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the Standard Specifications for Public Works Construction, ("Greenbook"), 2021 Edition. Utility trenches can be backfilled with onsite sandy material free of rubble, debris, organic and oversized material up to (≤) 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.

3.10 Drainage and Landscaping

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

3.11 Additional Geotechnical Services

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

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

- Grading and excavation of the site;
- Subgrade Preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This geotechnical exploration does not address the potential for encountering hazardous soil at this site. In addition, this report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is, by necessity, incomplete. Please also refer GBA's *Important Information About Your Geotechnical Report* (included at the rear of the text), presenting additional information and limitations regarding geotechnical engineering studies and reports. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton Consulting, Inc. has the opportunity to observe subsurface conditions during grading and construction, to confirm that our data are representative for the site. Leighton Consulting, Inc. should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing at this time in Los Angeles County. We do not make any warranty, either expressed or implied.

5.0 REFERENCES

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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

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

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

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

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

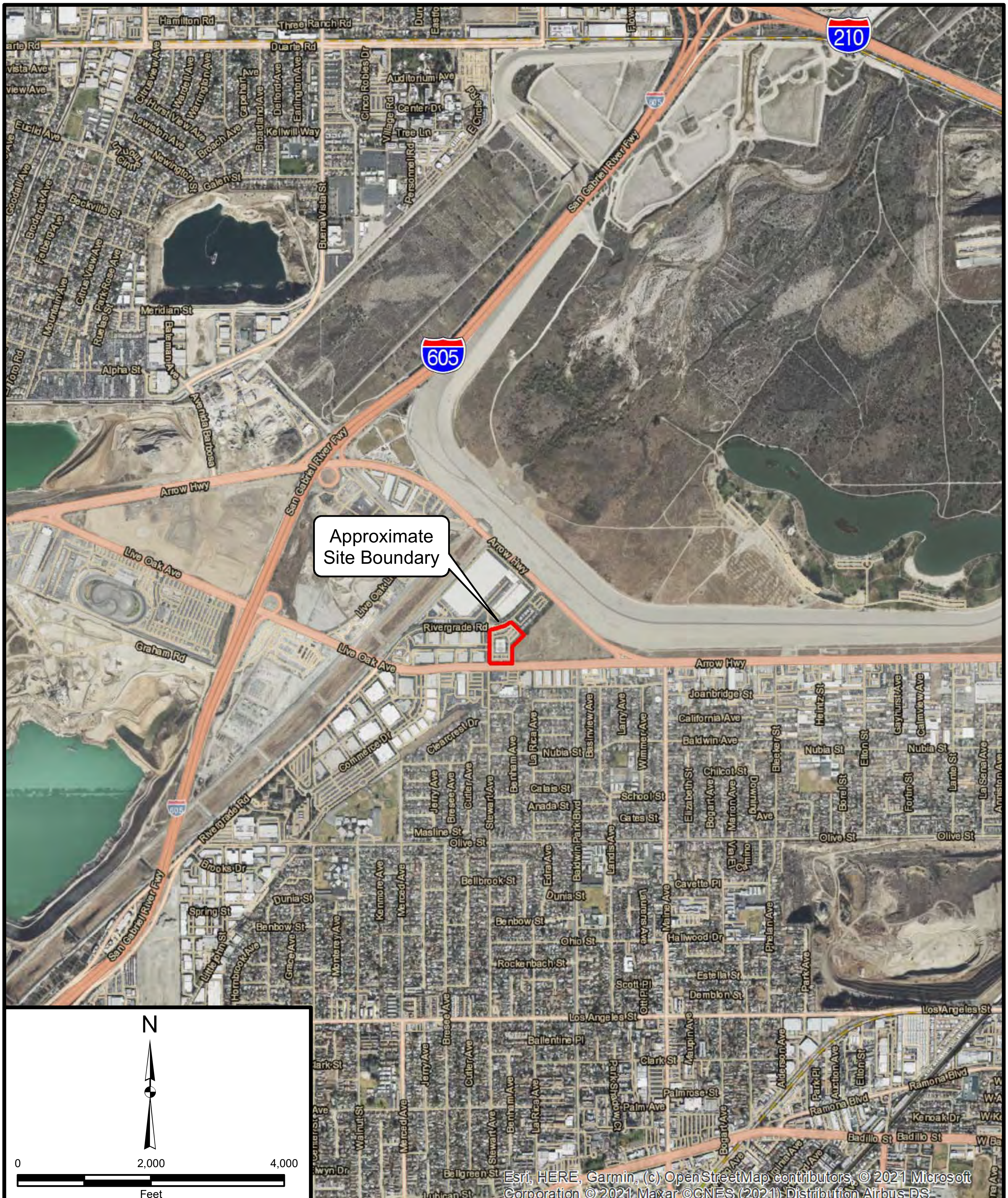
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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



Project: 13384.001	Eng/Geol: CCK/JMP	
Scale: 1 " = 2,000 '	Date: January 2022	
Base Map: ESRI ArcGIS Online 2022 Thematic Information: Leighton Author: Leighton Geomatics (btran)		

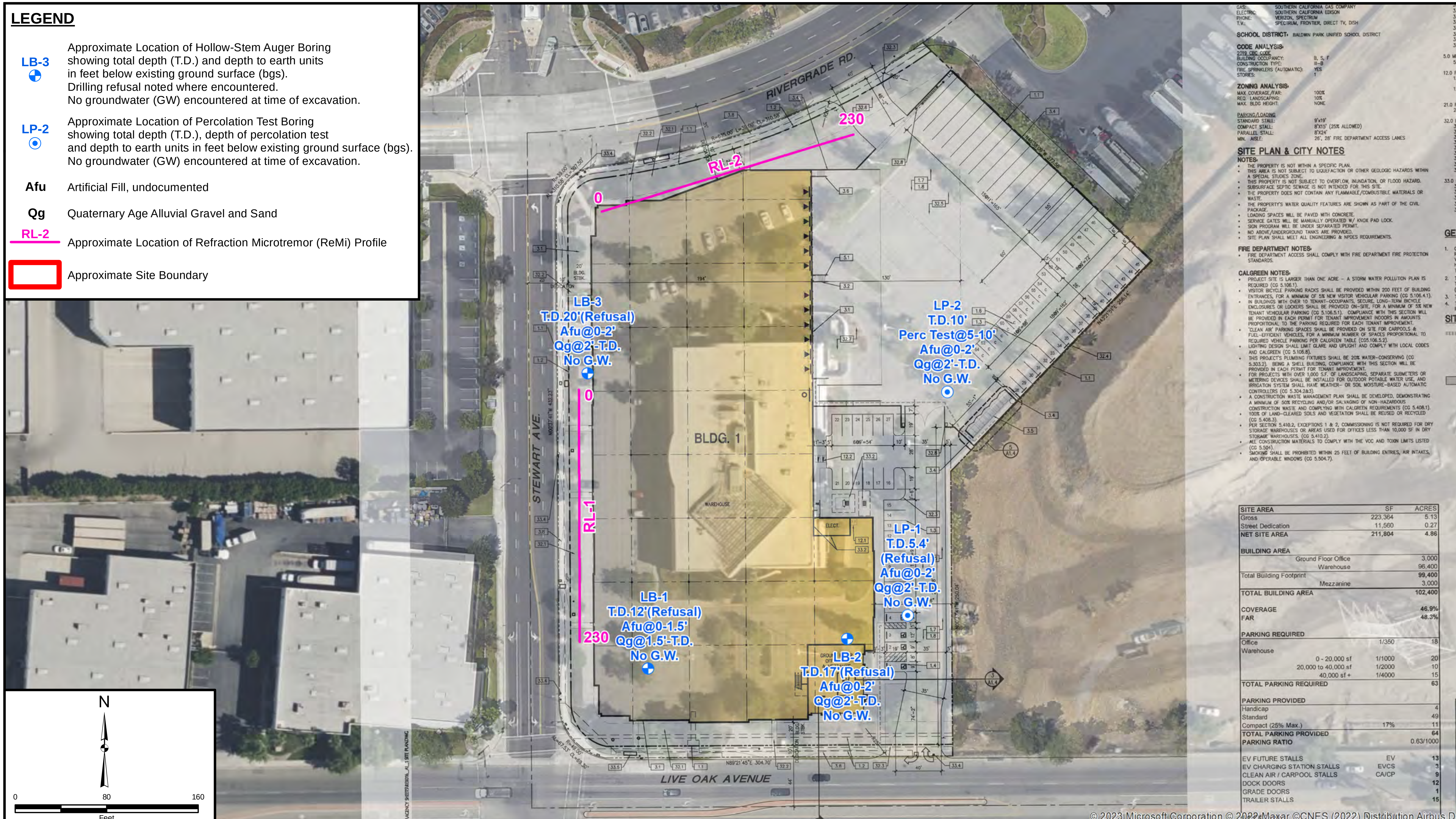
SITE LOCATION MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F01_SLM_2022-01-14.mxd on 1/13/2022 9:54:20 AM



Project: 13384.001	Eng/Geol: CCK/JMP
Scale: 1 " = 80 '	Date: March 2023

Base Map: Site Plan, Project Tabulations
Sheet A1.1, Dated: 12/22/2022 by GAA Architects

Author: Leighton Geomatics (btran)

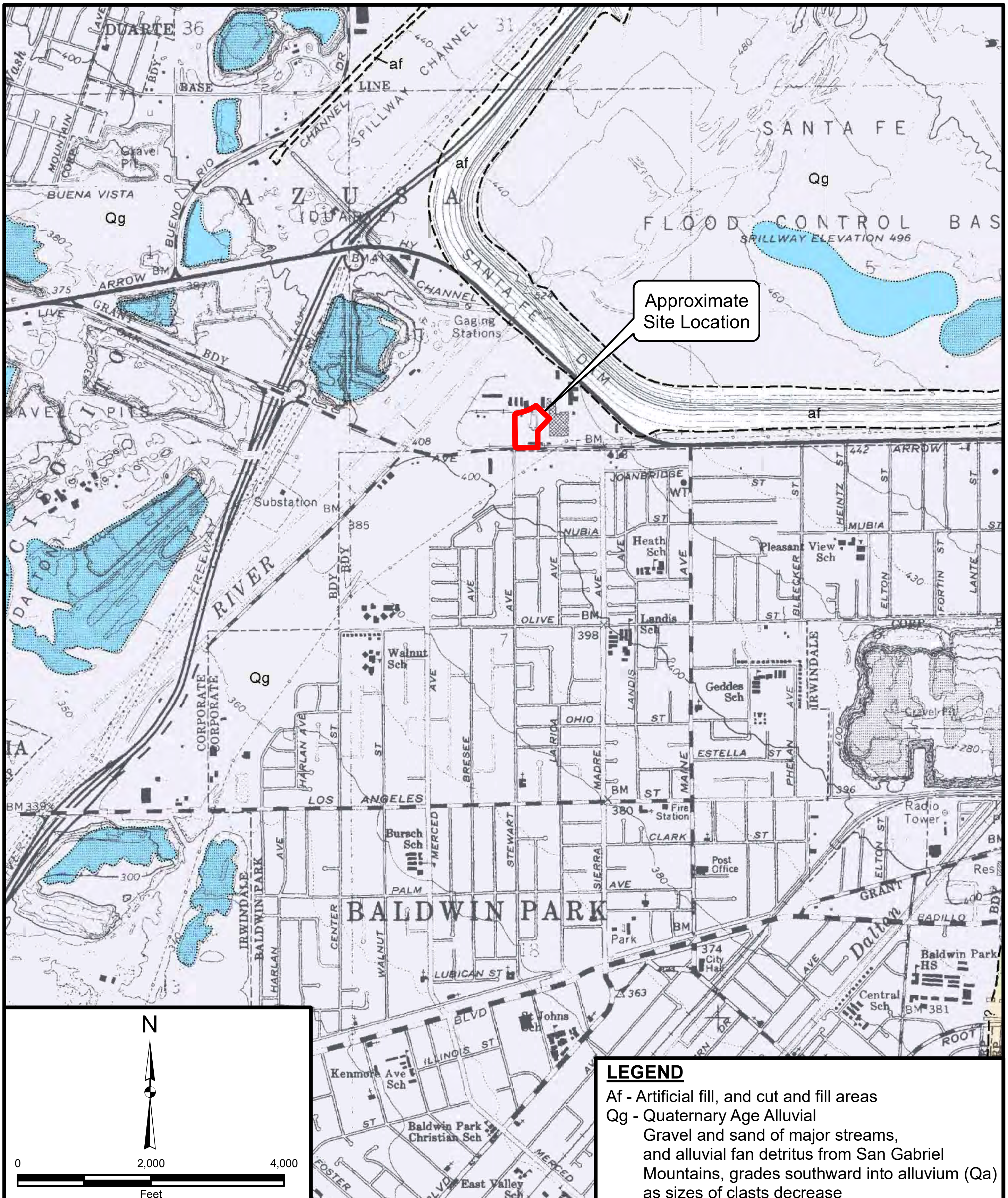
EXPLORATION LOCATION MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F02_ELM_2023-03-20.mxd on 3/20/2023 12:56:53 PM



LEGEND

Af - Artificial fill, and cut and fill areas

Qg - Quaternary Age Alluvial
Gravel and sand of major streams, and alluvial fan detritus from San Gabriel Mountains, grades southward into alluvium (Qa) as sizes of clasts decrease

Project: 13384.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: February 2022

Base Map: Geologic Map of the El Monte & Baldwin Park Quadrangles by Thomas W. Dibblee, JR., 1999

Author: Leighton Geomatics (btran)

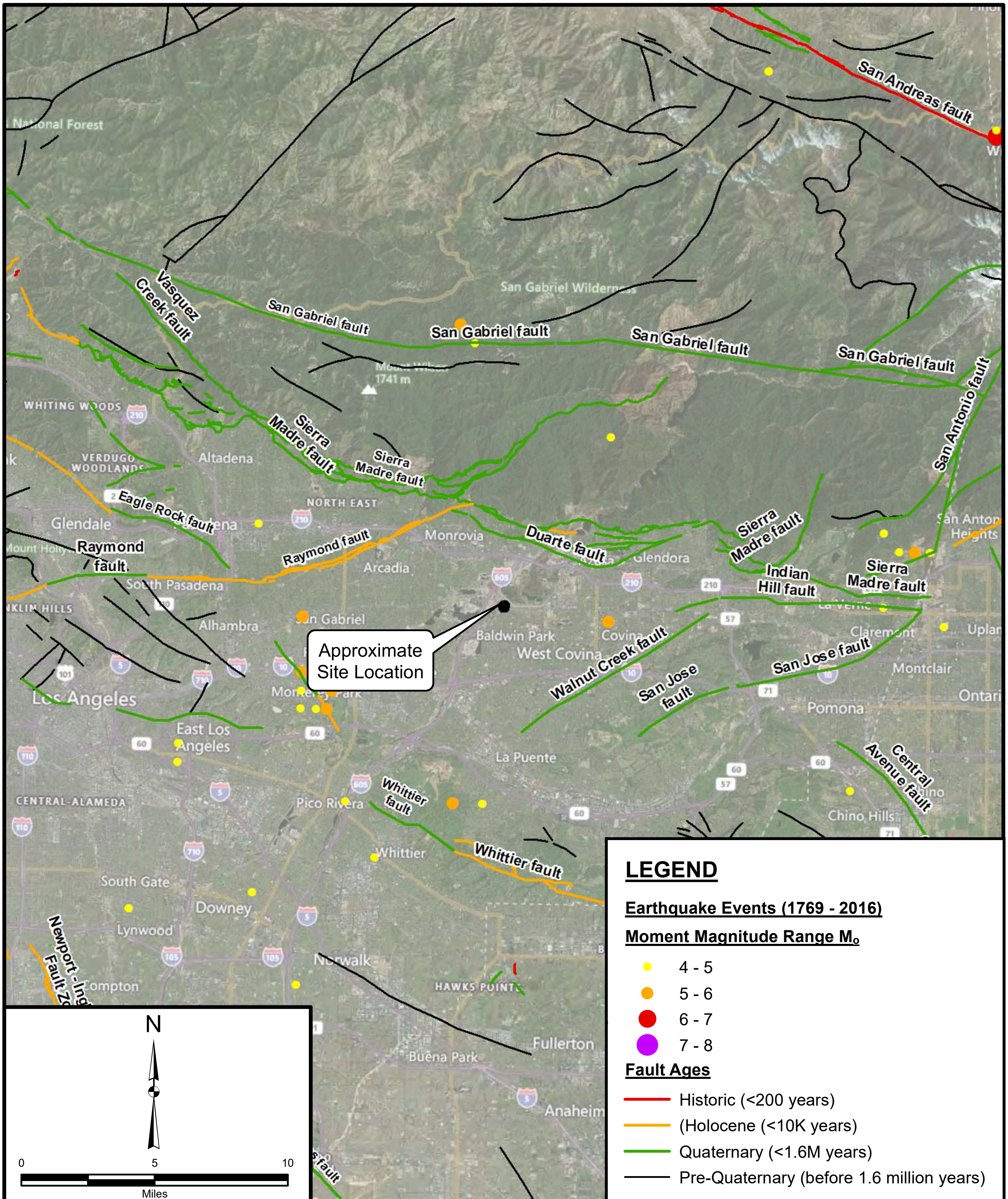
REGIONAL GEOLOGY MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 3

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F03_RGM_2022-02-11.mxd on 2/10/2022 9:54:27 AM



Project: 13384.001	Eng/Geol: CCK/JMP
Scale:1 " = 5 miles	Date: January 2022

Reference: ESRI ArcGIS Online 2022
Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 4

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F04_RF&HSM_2022-01-14.mxd on 1/13/2022 9:56:30 AM

Project: 13384.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: January 2022

Base Map: ESRI ArcGIS Online 2022

Author: Leighton Geomatics (btran)

SEISMIC HAZARD MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 5

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F05_SHM_2022-01-14.mxd on 1/13/2022 9:58:50 AM

Project: 13384.001	Eng/Geol: CCK/JMP
Scale:1 " = 4,000 '	Date: January 2022

Base Map: ESRI ArcGIS Online 2022
Reference: CA DWR, FEMA
Author: Leighton Geomatics (btran)

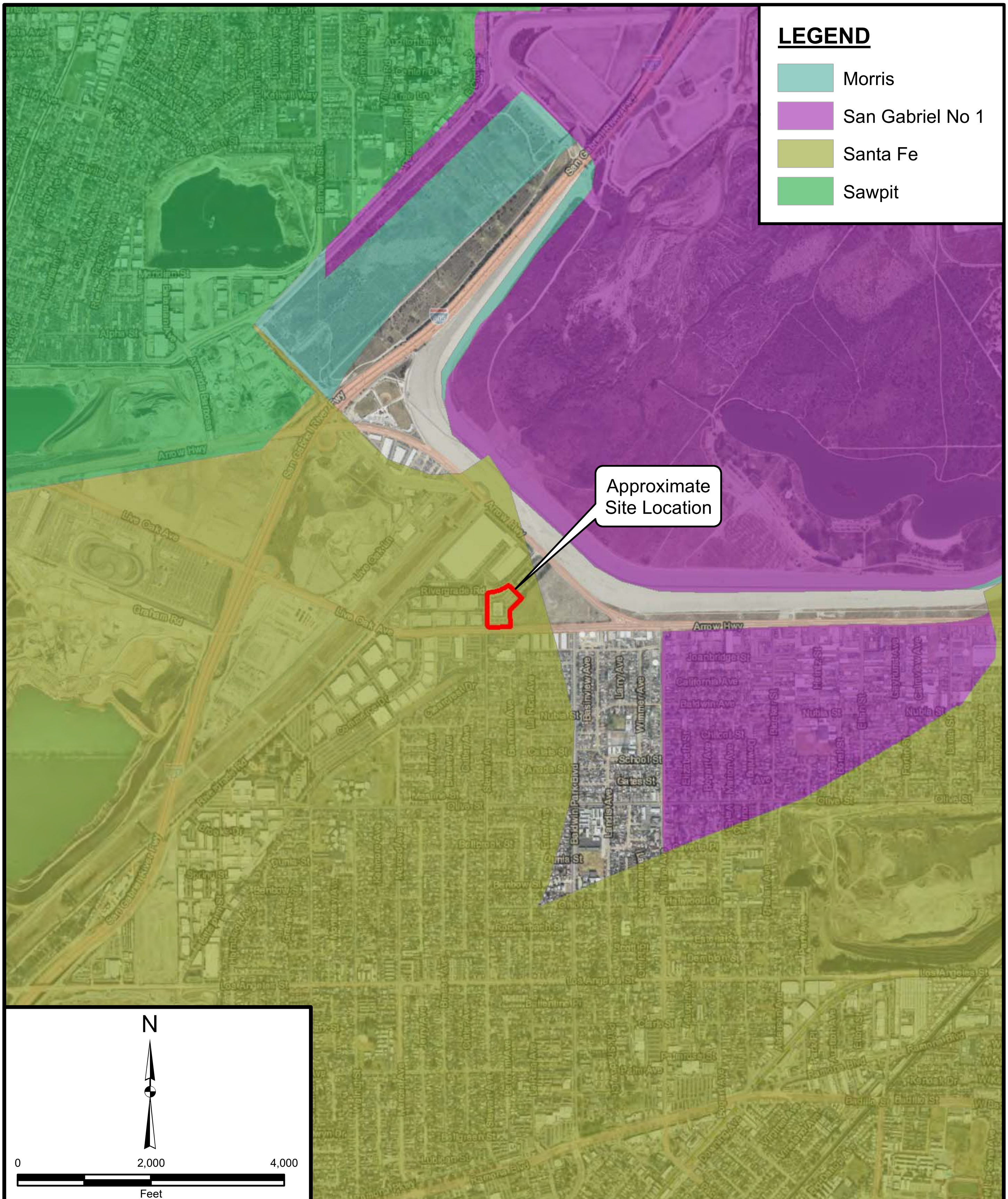
FLOOD HAZARD ZONE MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

FIGURE 6

Leighton

Map Saved as V:\Drafting\13384\001\Maps\13384-001_F06_FHM_2022-01-14.mxd on 1/13/2022 11:10:58 AM



Project: 13384.001	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: January 2022

Base Map: ESRI ArcGIS Online 2022

Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

Proposed Industrial Building
14005 Live Oak Avenue
Irwindale, California

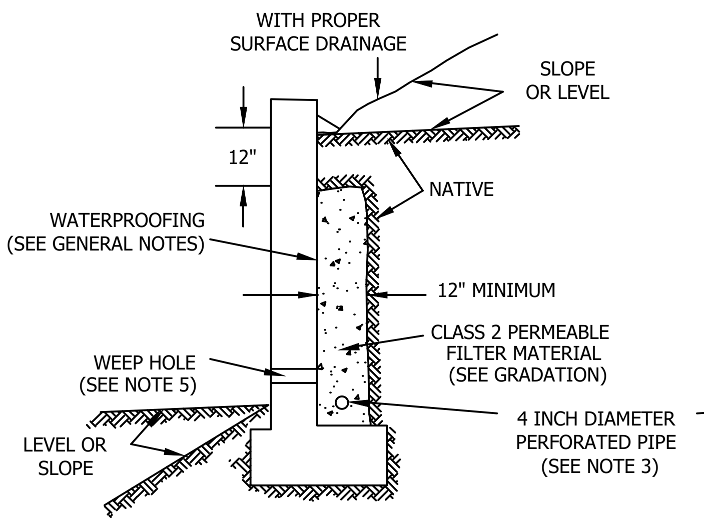
FIGURE 7

Leighton

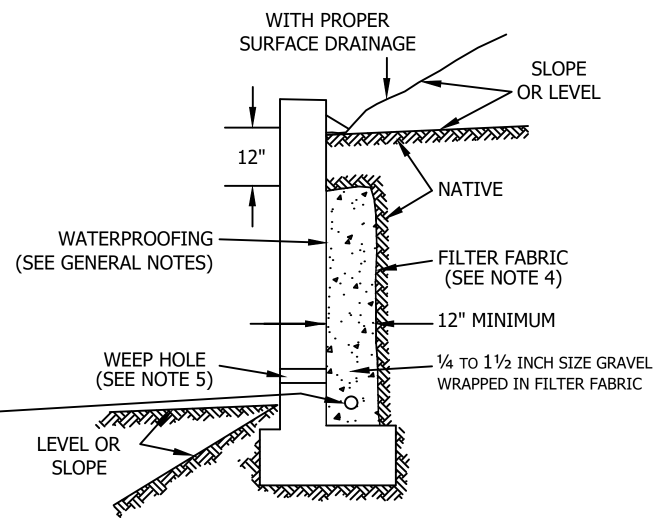
Map Saved as V:\Drafting\13384\001\Maps\13384-001_F07_DIM_2022-01-14.mxd on 1/13/2022 10:11:06 AM

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

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

FIGURE 8

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	13384.001	**Date Drilled**	1-4-22
Project	Rexford - 14005 Live Oak, Irwindale	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	410'
Location		**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
410	0			B-1				SM	@Surface: 2.5 inches of asphalt concrete **Artificial fill, undocumented (Afu)** @0.2': Silty SAND with gravel, brown, moist, fine sand and gravel **Quaternary Age Alluvial Gravel and Sand (Qg)** @1.5': Gravelly Silty SAND, light brown, moist, fine sand, fine to course gravel	CN, CR, DS, EI MD, RV
405	5			R-1	16 48 50/4"	131	1	SP	@5': Sandy GRAVEL, very dense, light brown, moist, fine sand, fine to course gravel, fine cobble	SA
				S-2	29 50/4"		1	GP	@7.5': Poorly graded GRAVEL, very dense, angular from mechanical fracturing, low recovery	
400	10			R-3	50/3"	118	1		@10': Poorly graded GRAVEL, very dense, light brown, moist, fine sand, fine to course gravel, fine to course cobble, subangular	
									@12': REFUSAL: Augers moving sideways Attempted to redrill and hit refusal at 6'	
									Total Depth: 12 feet due to refusal **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
395	15									
390	20									
385	25									
380	30									

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

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

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13384.001	**Date Drilled**	1-4-22
Project	Rexford - 14005 Live Oak, Irwindale	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	411'
Location		**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
411	0								@Surface: 3 inches of asphalt concrete over 6 inches of base	
410				B-1				SM	**Artificial fill, undocumented (Afu)** @0.75': Silty SAND with gravel, brown, moist, fine sand, fine gravel	
								SP	**Quaternary Age Alluvial Gravel and Sand (Qg)** @2': Gravelly SAND, brown, moist, fine sand, fine subangular gravel	
405	5			S-1	13 50/3"		4	GP	@5': Sandy GRAVEL, very dense, brown, moist, fine sand, fine to course gravel and cobble, subangular and mechanically fractured gravel	
				R-2	49 50/5"	150	1		@7.5': Poorly graded GRAVEL, very dense, light brown, slightly moist, fine sand, fine to course gravel and cobble, subangular and angular from mechanical fracturing	
400	10			S-3	15 20 18		1		@10': Sandy GRAVEL, very dense, light brown, slightly moist, fine sand,fine to course gravel and cobble, subangular	SA
395	15			R-4	50/5"	114	1		@15': Poorly graded GRAVEL, very dense, light brown, slightly moist, fine sand matrix, fine to course gravel and cobble, subangular, possible boulders	
									@17': REFUSAL: Augers moving sideways	
390	20								**Total Depth: 17 feet due to refusal** **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
385	25									
	30									

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

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

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

GEOTECHNICAL BORING LOG LB-3

Project No.	13384.001	**Date Drilled**	1-4-22
Project	Rexford - 14005 Live Oak, Irwindale	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	413'
Location		**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 inches of asphalt concrete **Artificial fill, undocumented (Afu)** @0.2': Silty SAND with gravel, brown, moist, fine sand, fine gravel	
410								SP	**Quaternary Age Alluvial Gravel and Sand (Qg)** @2': Gravelly Silty SAND, light brown, slightly moist, fine to course gravel, subround	
	5			S-1	7 43 47		1	GP	@5': Poorly graded GRAVEL with sand, very dense, light gray brown, slightly moist, fine sand, fine to course gravel, fine cobble	
405				R-2	50/5"	114	1		@7.5': Poorly graded GRAVEL with sand, very dense, light gray brown, slightly moist, fine sand, fine to course gravel, mostly course gravel, few cobble, top of sample is slough	
	10			S-3	50/5"		0		@10': Very dense, possible boulders, low recovery	
400										
	15			R-4	50/5"	132	1		@15': Sandy GRAVEL, very dense, light gray brown, slightly moist to moist, fine sand, fine to course gravel, little fine cobble	
395										
	20								@20': REFUSAL: Encountered cobble or boulder **Total Depth: 20 feet due to refusal** **No groundwater encountered during drilling.** **Backfilled with soil cuttings and patched with cold-mix asphalt.**	
390										
	25									
385										
	30									

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

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

GEOTECHNICAL BORING LOG LP-1

Project No.	13384.001	**Date Drilled**	1-4-22
Project	Rexford - 14005 Live Oak, Irwindale	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	413'
Location		**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								@Surface: 3.5 inches of asphalt concrete	
									Artificial fill, undocumented (Afu) @0.3': Silty SAND with gravel, moist, brown, fine sand and gravel	
410									**Quaternary Age Alluvial Gravel and Sand (Qg)** @2': Gravelly SAND, brown, moist, fine sand, fine gravel, subangular	
	5			S-1	50/5"		1	GP	@5': Poorly graded GRAVEL, very dense, light brown sand matrix, fine sand, fine to course gravel and cobble, possible boulder, low recovery	
405									**Total Depth: 5.4 feet due to refusal** **No groundwater encountered during drilling.** **Installed 2-inch diameter, 0.02-inch slotted PVC pipe from 0-5 feet.** **#3 Monterey SAND installed in annulus from 2-5 feet.**	
	10									
400										
	15									
395										
	20									
390										
	25									
385										
	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
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- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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

GEOTECHNICAL BORING LOG LP-2

Project No.	13384.001	**Date Drilled**	1-4-22
Project	Rexford - 14005 Live Oak, Irwindale	**Logged By**	MM
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	414'
Location		**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: 2.5 inches of asphalt concrte over 5 inches of base **Artificial fill, undocumented (Afu)** @0.6': Silty SAND with gravel, brown, moist, fine sand and gravel	
								SP	**Quaternary Age Alluvial Gravel and Sand (Qg)** @2': Gravelly SAND, brown, moist, fine sand, fine gravel, subangular gravel	
410	5			S-1	15 28 35		1	GP	@5': Poorly graded GRAVEL, very dense, slightly moist, fine to course gravel and cobble, subangular, fine sand matrix, possible boulder @7': Stepped over and redrilled due to refusal at 5'.	
405				S-2 G-3	4 10 14		1	SP	@8.5': Gravelly SAND, dense, light brown, slightly moist, fine sand, fine to course gravel, mechanically fractured gravel, no recovery @9': Gravelly SAND, light brown, slightly moist, fine sand, fine to course gravel and cobbles, mechanically fractured rock	
	10								**Total Depth: 10 feet** **No groundwater encountered during drilling** **Installed 2-inch diameter, solid PVC pipe from 0-5 feet.** **Installed 2-inch diameter, 0.02-inch slotted PVC pipe from 5-10 feet.** **#3 Monterey SAND installed in annulus from 4-10 feet.**	
400	15									
395	20									
390	25									
385	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
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- UC UNCONFINED COMPRESSIVE STRENGTH

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

APPENDIX B

PERCOLATION TEST DATA

Leighton

Boring Percolation Test Data Sheet

Project Number:	13384.001	**Test Hole Number:**	LP-1
Project Name:	Rexford Irwindale	**Date Excavated:**	1/4/2022
Earth Description:	Alluvium	**Date Tested:**	1/5/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	5
Tested By:	MM	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:		**Length of slotted of casing (ft):**	5
Start Time for Standard:		**Depth to Initial Water Depth (ft):**	2
Standard Time Interval Between Readings, mins:		**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.)
P1	11:12	15	2.78	26.6	24.8	4.79
	11:27		4.85	1.8		
P2	11:33	15	2.75	27.0	23.5	4.26
	11:48		4.71	3.5		
1	12:16	10	2.00	36.0	25.8	4.82
	12:26		4.15	10.2		
2	12:26	10	2.00	36.0	25.6	4.75
	12:36		4.13	10.4		
3	12:37	10	2.00	36.0	25.2	4.65
	12:47		4.10	10.8		
4	12:48	10	2.00	36.0	25.0	4.58
	12:58		4.08	11.0		
5	12:59	10	2.00	36.0	24.4	4.42
	13:09		4.03	11.6		
6	13:10	10	2.00	36.0	24.0	4.33
	13:20		4.00	12.0		
7	13:21	10	2.00	36.0	24.0	4.33
	13:31		4.00	12.0		
8	13:32	10	2.00	36.0	23.9	4.30
	13:42		3.99	12.1		
9	13:43	10	2.00	36.0	23.4	4.17
	13:53		3.95	12.6		
10	13:54	10	2.00	36.0	22.7	3.99
	14:04		3.89	13.3		
11	14:05	10	2.00	36.0	22.3	3.90
	14:15		3.86	13.7		
12	14:15	10	2.00	36.0	22.3	3.90
	14:25		3.86	13.7		
13	14:27	10	2.00	36.0	22.1	3.84
	14:37		3.84	13.9		
14	14:38	10	2.00	36.0	22.2	3.87
	14:48		3.85	13.8		
15	14:49	10	2.00	36.0	21.8	3.78
	14:59		3.82	14.2		

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

Measured Infiltration Rate, I (Average of Last 3 Readings) = 3.82 in./hr.

Design Infiltration Rate

Reduction Factor from Test Procedure, RF_t = 1

Reduction Factor for Site Variability, # of Tests and Investigation, RF_v = 1

Reduction Factor for Long Term Siltation, Plugging and Maintenance, RF_t = 1

Reduction Factor, RF = $RF_t + RF_v + RF_s$ = 3

Design Infiltration Rate = Measured Infiltration Rate / Reduction Factor (RF) = 1.27 in./hr.

Boring Percolation Test Data Sheet

Project Number:	13384.001	**Test Hole Number:**	LP-2
Project Name:	Rexford Irwindale	**Date Excavated:**	1/4/2022
Earth Description:	Alluvium	**Date Tested:**	1/5/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	MM	**Radius of boring, r (in):**	4
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	6
		Porosity of Annulus Material, *n* :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data - High Flow-Rate Constand Head Test

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	7:50	-	-	-	0.0
2	8:00	10	6.54	41.5	13.2
3	8:10	10	6.62	40.6	26.5
4	8:20	10	6.67	40.0	39.7
5	8:30	10	6.49	42.1	53.0
6	8:40	10	6.49	42.1	66.2
7	8:50	10	6.66	40.1	79.5
8	9:00	10	6.76	38.9	92.7
9	9:10	10	6.66	40.1	106.0
10	9:20	10	6.70	39.6	119.2
11	9:30	10	6.76	38.9	132.5
12	9:40	10	6.84	37.9	145.7
13	9:50	10	6.86	37.7	158.9
14	10:00	10	6.87	37.6	172.2
15	10:10	10	6.90	37.2	185.4
16	10:20	10	6.89	37.3	198.7
17	10:30	10	6.90	37.2	211.9
18	10:40	10	6.92	37.0	225.2
19	10:50	10	6.90	37.2	238.4
20	11:00	10	6.89	37.3	251.7

Total Volume of Water Delivered (gallons)	251.7
Total Volume of Water Delivered (cubic inches)	58132.45033
Average Water Height (inches)	39.0
Average Percolation Surface Area (cubic Inches)	1029.3
Duration of Test (minutes)	190
Duration of Test (hours)	3.17

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

Measured Infiltration Rate (inches per hour) = **17.8**

Design Infiltration Rate

Reduction Factor from Test Procedure, RF_t =	3
Reduction Factor for Site Variability, # of Tests and Investigation, RF_v =	1
Reduction Factor for Long Term Siltation, Plugging and Maintenance, RF_t =	1
Reduction Factor, RF = RF_t + RF_v + RF_s =	5
Design Infiltration Rate = Measured Infiltration Rate / Reduction Factor (RF) =	**3.57** in./hr.

APPENDIX C

LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Rexford Live Oak Irwindale	Tested By:	L. Monka	Date:	01/10/22
Project No.:	13384.001	Checked By:	A. Santos	Date:	01/12/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand with gravel (SM)g				

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

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

Scalp Fraction (%)	
#3/4	
#3/8	
#4	25.4

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) = 0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3858	3941	3914			
Weight of Mold (g)	1850	1850	1850			
Net Weight of Soil (g)	2008	2091	2064			
Wet Weight of Soil + Cont. (g)	737.2	781.3	787.8			
Dry Weight of Soil + Cont. (g)	695.1	722.0	716.4			
Weight of Container (g)	88.6	88.5	87.8			
Moisture Content (%)	6.94	9.36	11.36			
Wet Density (pcf)	132.9	138.4	136.6			
Dry Density (pcf)	124.3	126.6	122.7			

Maximum Dry Density (pcf) 126.8 **Optimum Moisture Content (%)** 9.0

Corrected Dry Density (pcf) 135.3 **Corrected Moisture Content (%)** 7.0

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

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

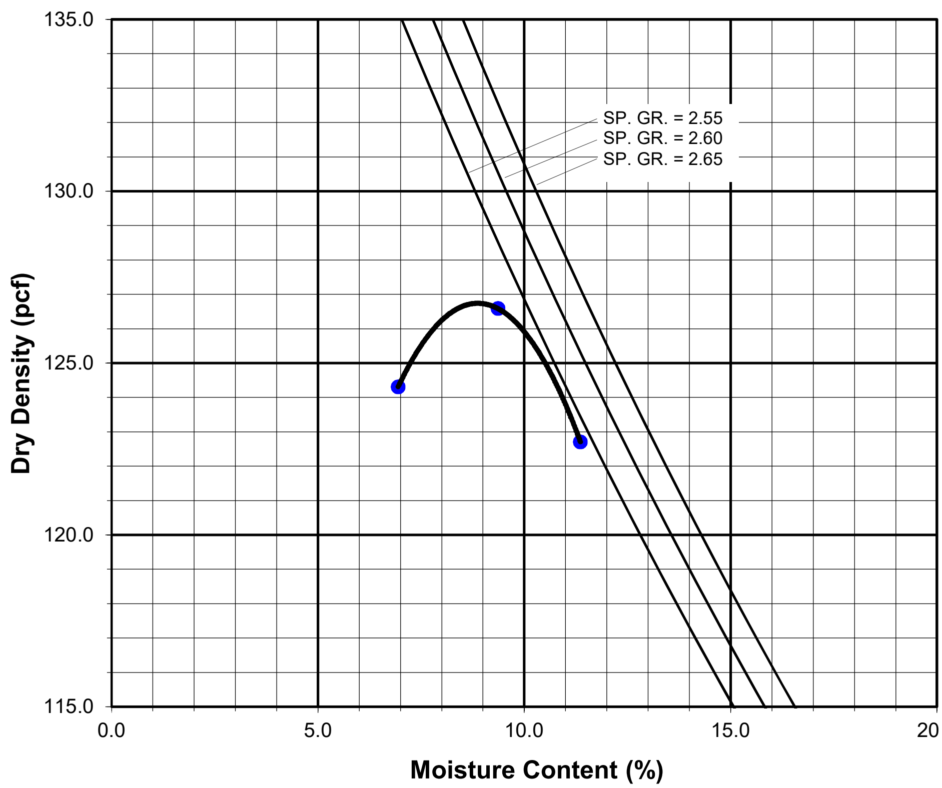
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Rexford Live Oak Irwindale	Tested By:	G. Berdy	Date:	01/13/22
Project No.:	13384.001	Checked By:	A. Santos	Date:	01/24/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand with gravel (SM)g				

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

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0000
Wt. Comp. Soil + Mold (g)	607.50	440.10
Wt. of Mold (g)	191.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	837.90	631.50
Dry Wt. of Soil + Cont. (g)	774.40	575.97
Wt. of Container (g)	0.00	191.40
Moisture Content (%)	8.20	14.44
Wet Density (pcf)	125.5	132.8
Dry Density (pcf)	116.0	116.0
Void Ratio	0.453	0.453
Total Porosity	0.312	0.312
Pore Volume (cc)	64.6	64.6
Degree of Saturation (%) [S_{meas}]	**48.8**	86.0

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
01/13/22	10:50	1.0	0	0.5100
01/13/22	11:00	1.0	10	0.5100
Add Distilled Water to the Specimen				
01/13/22	11:30	1.0	30	0.5100
01/14/22	7:00	1.0	1200	0.5100
01/14/22	9:10	1.0	1330	0.5100

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**0**

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PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D6913

Project Name: Rexford Live Oak Irwindale — Tested By: J. Domingo — Date: 01/20/22
Project No.: 13384.001 — Checked By: A. Santos — Date: 01/21/22
Boring No.: LB-1 — Depth (feet): 5.0
Sample No.: R-1
Soil Identification: Light brownish gray poorly-graded gravel with sand (GP)s

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	884	1532	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	987.5	399.8	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	74.9	77.4	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	912.6	322.4	Moisture Content (%)	0.0	0.0

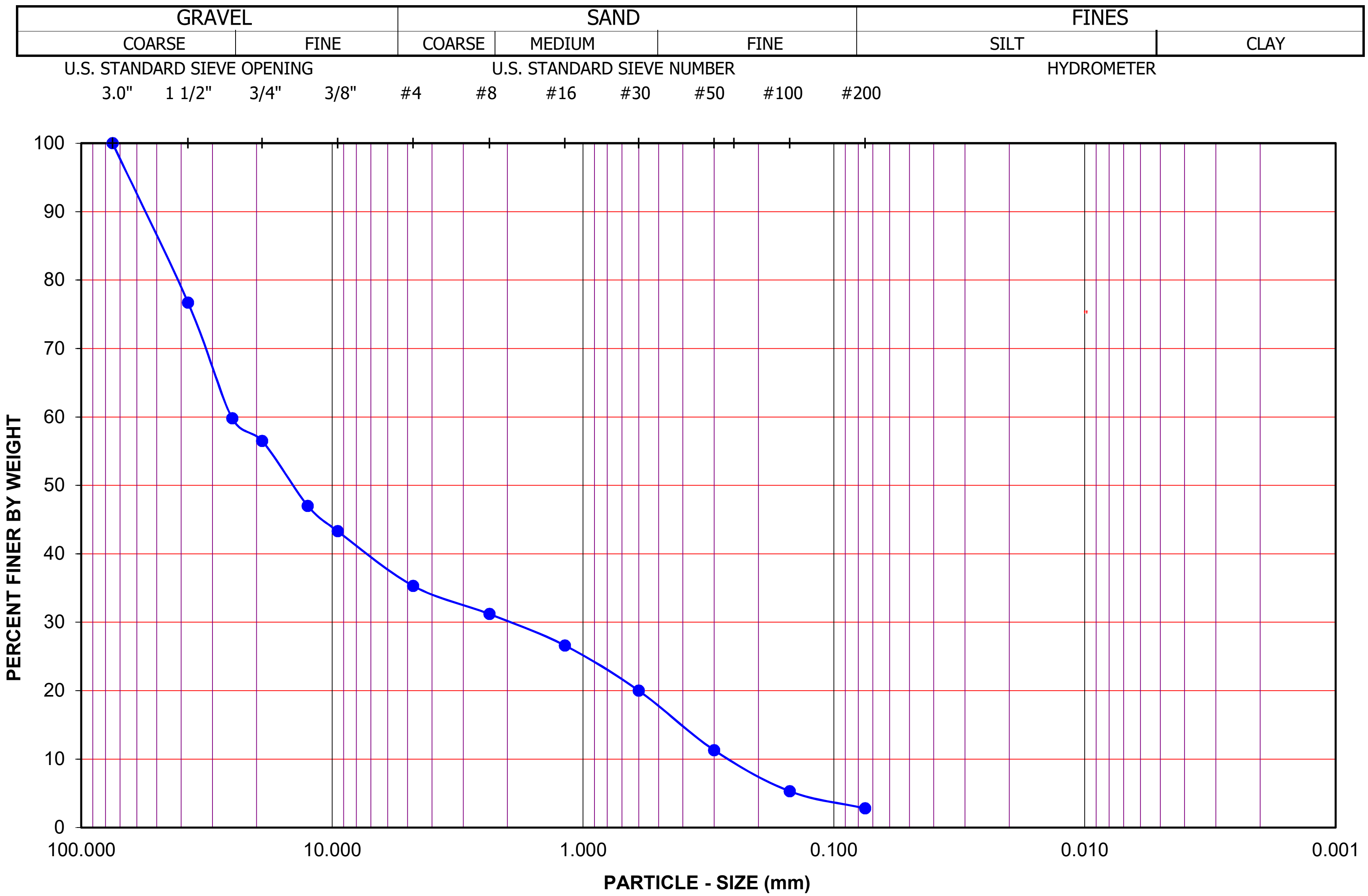
Passing #4 Material After Wet Sieve	Container No.	1532
	Wt. of Dry Soil + Container (g)	275.9
	Wt. of Container (g)	77.4
	Dry Wt. of Soil Retained on # 200 Sieve (g)	198.5

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0	0.0		100.0
1 1/2"	37.5	212.5		76.7
1"	25.0	366.6		59.8
3/4"	19.0	396.6		56.5
1/2"	12.5	483.4		47.0
3/8"	9.5	517.1		43.3
#4	4.75	590.2		35.3
#8	2.36		37.1	31.2
#16	1.18		79.3	26.6
#30	0.600		139.8	20.0
#50	0.300		219.5	11.3
#100	0.150		274.2	5.3
#200	0.075		296.9	2.8
PAN				

GRAVEL: **65 %**
SAND: **32 %**
FINES: **3 %**
GROUP SYMBOL: **(GP)s**

Cu = D60/D10 = 92.59
Cc = $(D30)^2/(D60*D10)$ = 0.59

Remarks:



Project Name: Rexford Live Oak Irwindale

Project No.: 13384.001

Boring No.: LB-1

Sample No.: R-1

Depth (feet): 5.0

Soil Type : (GP)s

Soil Identification: Light brownish gray poorly-graded gravel with sand (GP)s

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 65 : 32 : 3

Jan-22

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D6913

Leighton

Project Name:	Rexford Live Oak Irwindale	Tested By:	J. Domingo	Date:	01/14/22
Project No.:	13384.001	Checked By:	A. Santos	Date:	01/21/22
Boring No.:	LB-2	Depth (feet):	10.0		
Sample No.:	S-3				
Soil Identification:	Brown poorly-graded gravel with sand and silt (GP-GM)s				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	788	750	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	579.7	297.8	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	75.5	76.6	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	504.2	221.2	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	750
	Wt. of Dry Soil + Container (g)	275.9
	Wt. of Container (g)	76.6
	Dry Wt. of Soil Retained on # 200 Sieve (g)	199.3

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5	0.0		100.0
1"	25.0	55.2		89.1
3/4"	19.0	109.5		78.3
1/2"	12.5	203.4		59.7
3/8"	9.5	232.5		53.9
#4	4.75	282.6		44.0
#8	2.36		39.8	36.1
#16	1.18		75.8	28.9
#30	0.600		114.0	21.3
#50	0.300		154.1	13.3
#100	0.150		180.8	8.0
#200	0.075		196.9	4.8
PAN				

GRAVEL: **56 %**

SAND: **39 %**

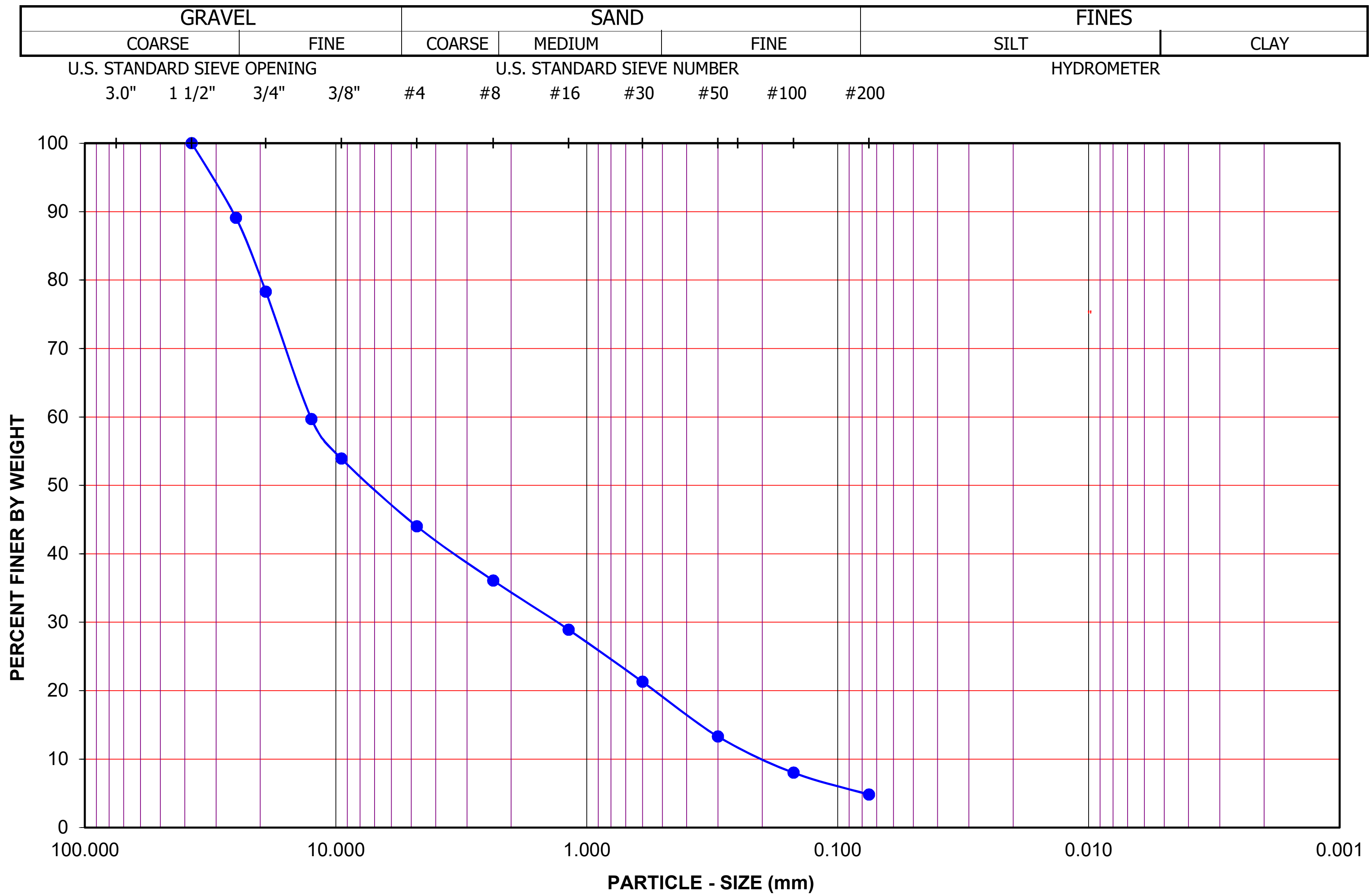
FINES: **5 %**

GROUP SYMBOL: **(GP-GM)s**

$Cu = D60/D10 =$ 52.00

$Cc = (D30)^2/(D60*D10) =$ 0.94

Remarks:



Project Name: Rexford Live Oak Irwindale

Project No.: 13384.001

Boring No.: LB-2 Sample No.: S-3

Depth (feet): 10.0 Soil Type : (GP-GM)s

Soil Identification: Brown poorly-graded gravel with sand and silt (GP-GM)s

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 56 : 39 : 5

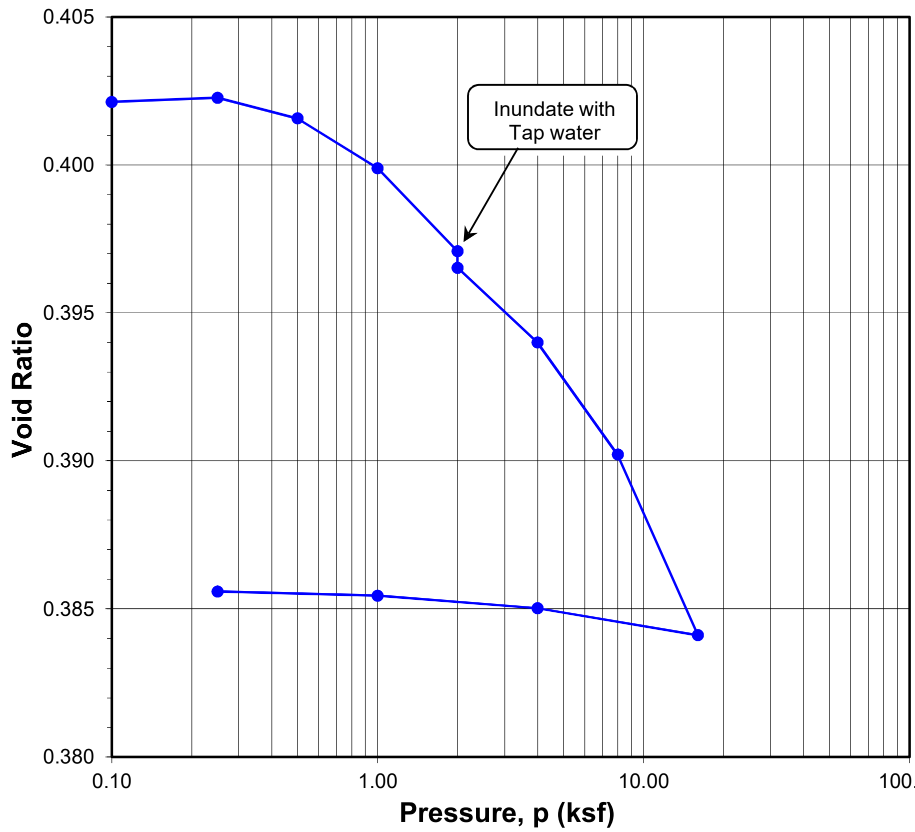
Jan-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Rexford Live Oak Irwindale	Tested By:	G. Bathala	Date:	01/13/22
Project No.:	13384.001	Checked By:	A. Santos	Date:	01/21/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1	Sample Type:	95% Remold		
Soil Identification:	Olive brown silty sand with gravel (SM)g				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	200.28
Weight of Ring (g)	42.86
Height after consol. (in.)	0.9882
Before Test	
Wt.Wet Sample+Cont. (g)	178.11
Wt.of Dry Sample+Cont. (g)	168.37
Weight of Container (g)	58.96
Initial Moisture Content (%)	8.9
Initial Dry Density (pcf)	120.2
Initial Saturation (%)	60
Initial Vertical Reading (in.)	0.3223
After Test	
Wt.of Wet Sample+Cont. (g)	264.70
Wt. of Dry Sample+Cont. (g)	246.72
Weight of Container (g)	59.16
Final Moisture Content (%)	12.43
Final Dry Density (pcf)	121.8
Final Saturation (%)	87
Final Vertical Reading (in.)	0.3073
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

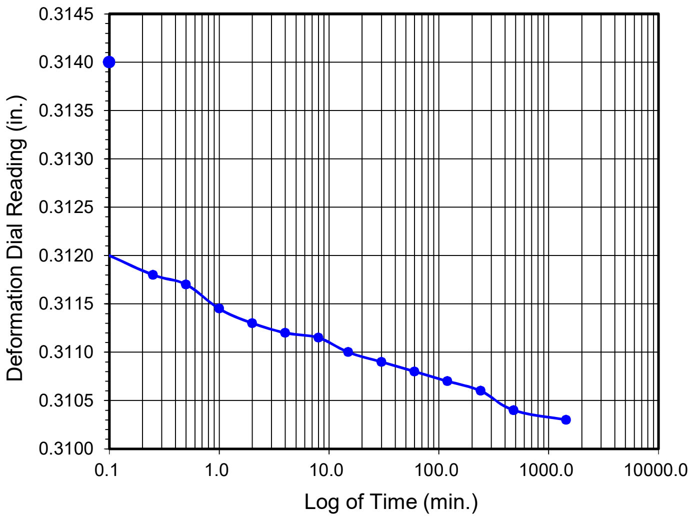


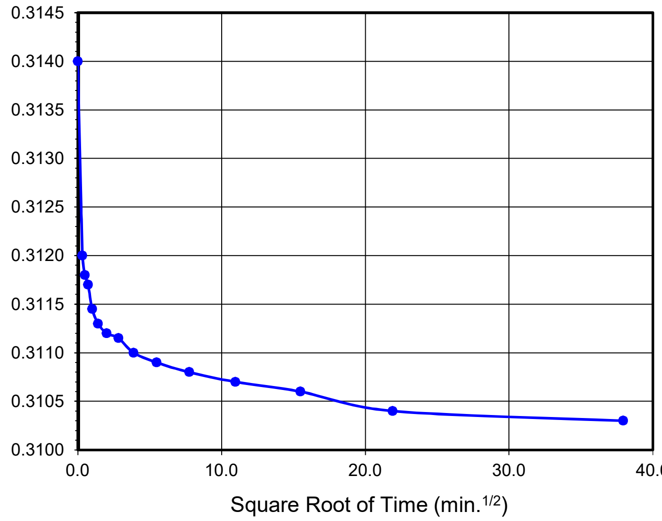
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3223	1.0000	0.00	0.00	0.402	0.00
0.25	0.3215	0.9992	0.09	0.08	0.402	-0.01
0.50	0.3201	0.9978	0.18	0.22	0.402	0.04
1.00	0.3178	0.9955	0.29	0.45	0.400	0.16
2.00	0.3144	0.9921	0.43	0.79	0.397	0.36
2.00	0.3140	0.9917	0.43	0.83	0.397	0.40
4.00	0.3103	0.9880	0.62	1.20	0.394	0.58
8.00	0.3054	0.9831	0.84	1.69	0.390	0.85
16.00	0.2988	0.9765	1.07	2.36	0.384	1.29
4.00	0.3018	0.9795	0.83	2.05	0.385	1.22
1.00	0.3050	0.9827	0.54	1.73	0.385	1.19
0.25	0.3073	0.9850	0.32	1.50	0.386	1.18

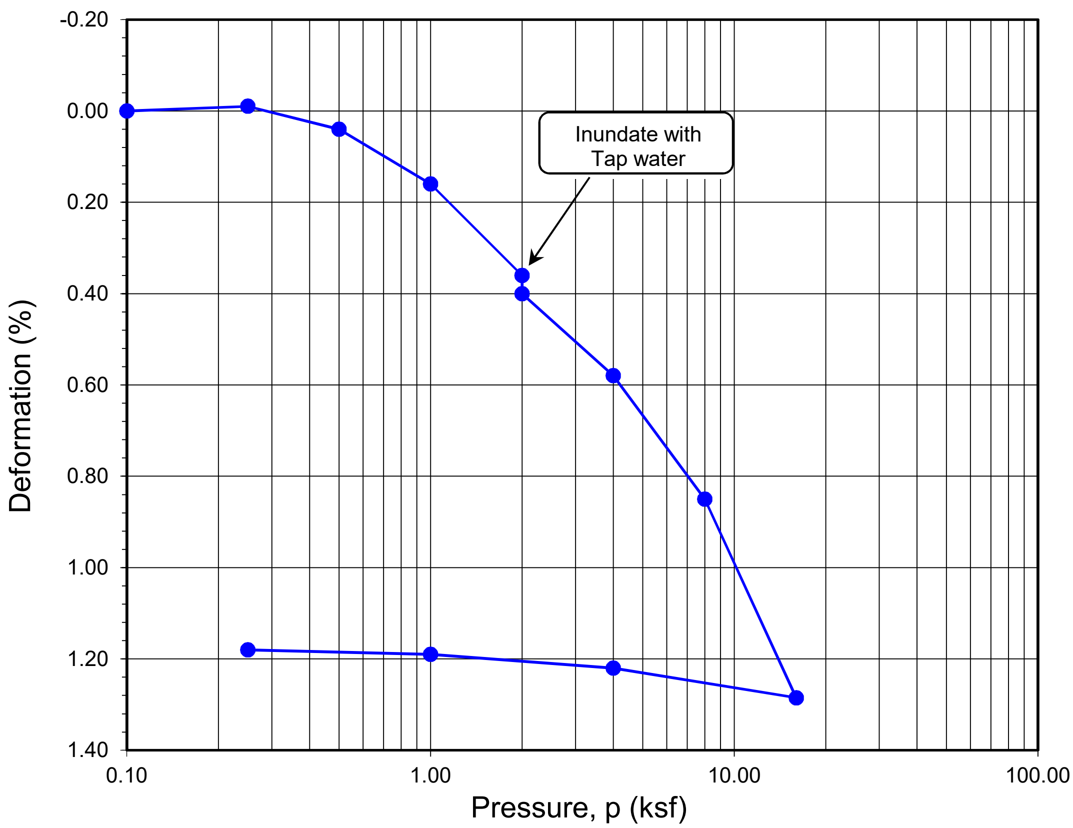
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
1/17/22	10:35:00	0.0	0.0	0.3140
1/17/22	10:35:06	0.1	0.3	0.3120
1/17/22	10:35:15	0.2	0.5	0.3118
1/17/22	10:35:30	0.5	0.7	0.3117
1/17/22	10:36:00	1.0	1.0	0.3115
1/17/22	10:37:00	2.0	1.4	0.3113
1/17/22	10:39:00	4.0	2.0	0.3112
1/17/22	10:43:00	8.0	2.8	0.3112
1/17/22	10:50:00	15.0	3.9	0.3110
1/17/22	11:05:00	30.0	5.5	0.3109
1/17/22	11:35:00	60.0	7.7	0.3108
1/17/22	12:35:00	120.0	11.0	0.3107
1/17/22	14:35:00	240.0	15.5	0.3106
1/17/22	18:35:00	480.0	21.9	0.3104
1/18/22	10:35:00	1440.0	37.9	0.3103

Time Readings @ 4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**B-1**	**0-5**	**8.9**	**12.4**	**120.2**	**121.8**	**0.402**	**0.386**	**60**	**87**

Soil Identification: Olive brown silty sand with gravel (SM)g

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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13384.001

Rexford Live Oak Irwindale

01-22

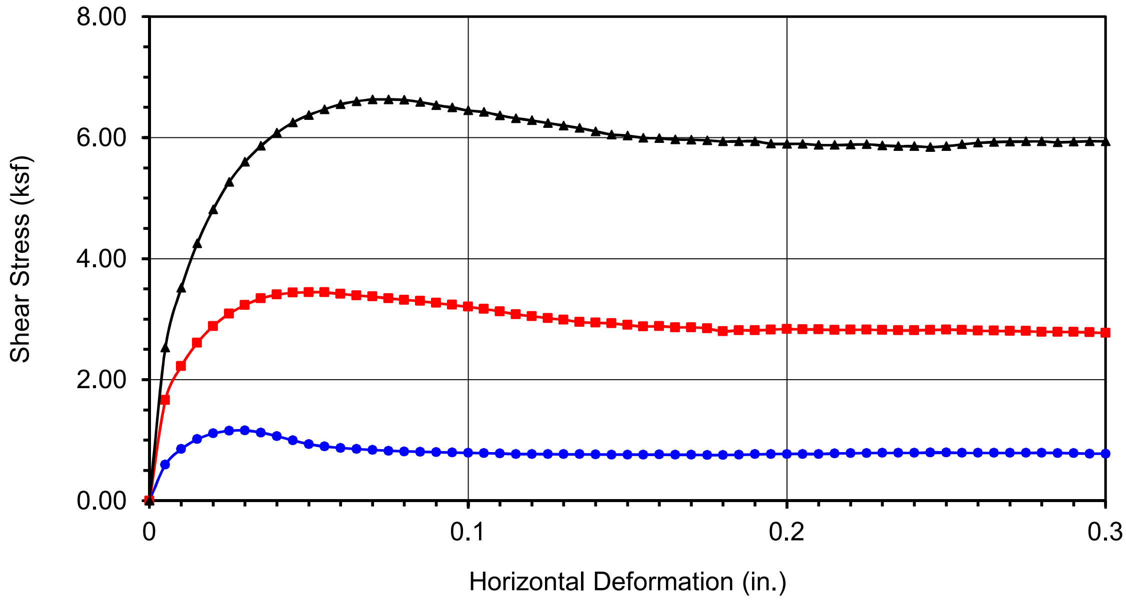
Leighton

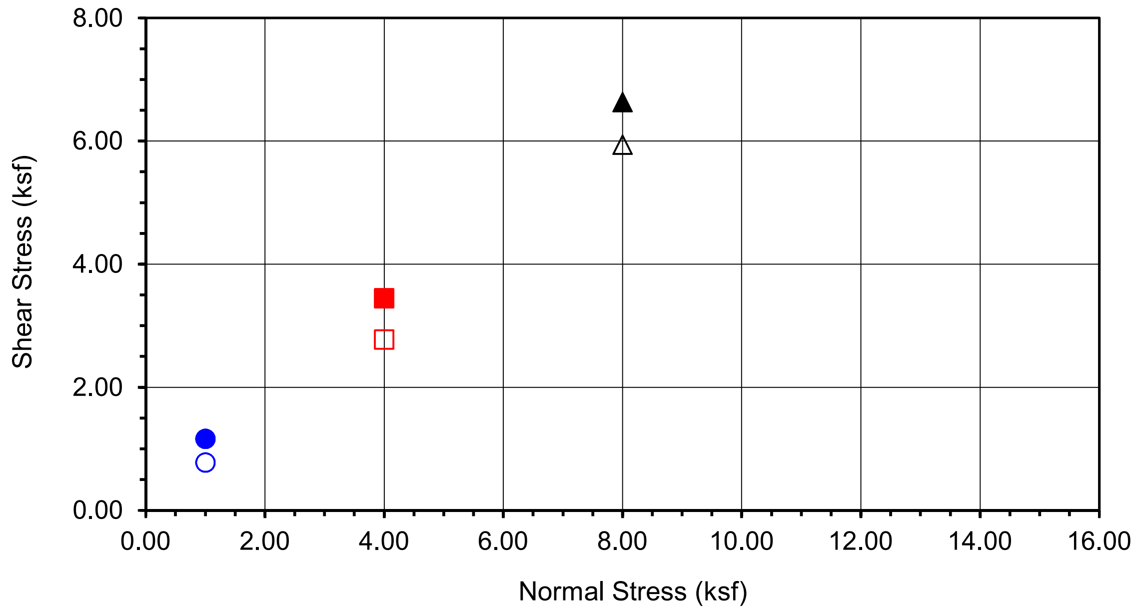
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: Rexford Live Oak Irwindale
Tested By: G. Bathala
Date: 01/19/22
Project No.: 13384.001
Checked By: A. Santos
Date: 01/24/22
Boring No.: LB-1
Sample Type: 95% Remold
Sample No.: B-1
Depth (ft.): 0-5
Soil Identification: Olive brown silty sand with gravel (SM)g

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	203.51	203.72	204.05
Weight of Ring(gm):	45.48	45.49	45.68
Before Shearing			
Weight of Wet Sample+Cont.(gm):	178.11	178.11	178.11
Weight of Dry Sample+Cont.(gm):	168.37	168.37	168.37
Weight of Container(gm):	58.96	58.96	58.96
Vertical Rdg.(in): Initial	0.2632	0.2561	0.0000
Vertical Rdg.(in): Final	0.2722	0.2709	-0.0304
After Shearing			
Weight of Wet Sample+Cont.(gm):	229.42	227.54	227.52
Weight of Dry Sample+Cont.(gm):	210.34	209.35	209.47
Weight of Container(gm):	66.93	66.85	66.08
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





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

Sample Type:

95% Remold

Soil Identification:
Olive brown silty sand with gravel (SM)g

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.160	■ 3.442	▲ 6.630
Shear Stress @ End of Test (ksf)	○ 0.777	□ 2.770	△ 5.939
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 (%)	8.90	8.90	8.90
Dry Density (pcf)	120.7	120.8	120.9
Saturation (%)	60.6	60.9	61.0
Soil Height Before Shearing (in.)	0.9910	0.9852	0.9696
Final Moisture Content (%)	13.3	12.8	12.6

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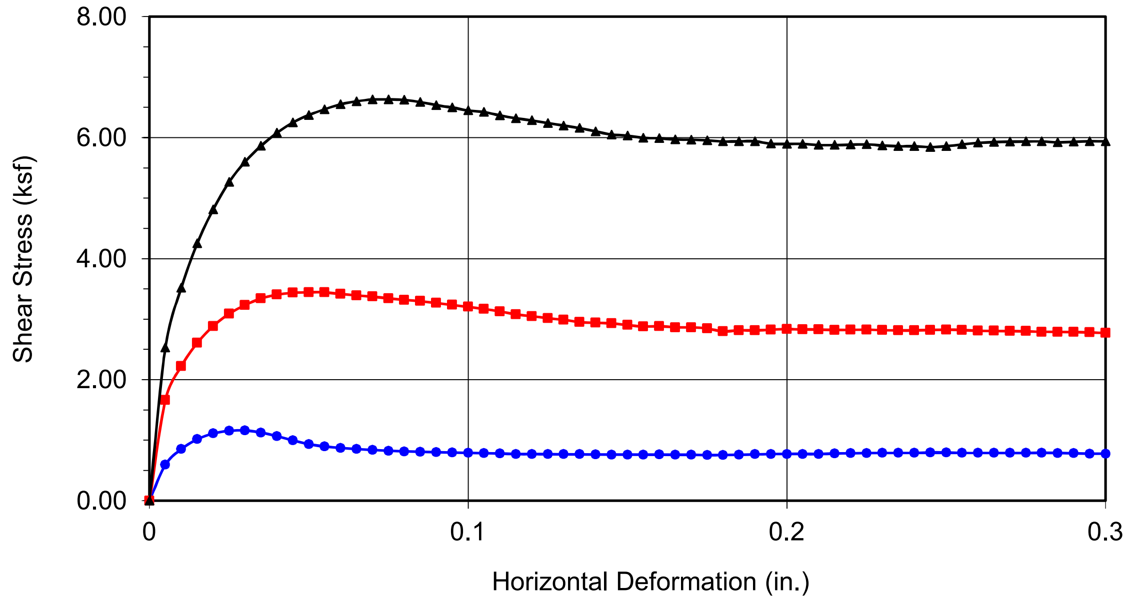
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

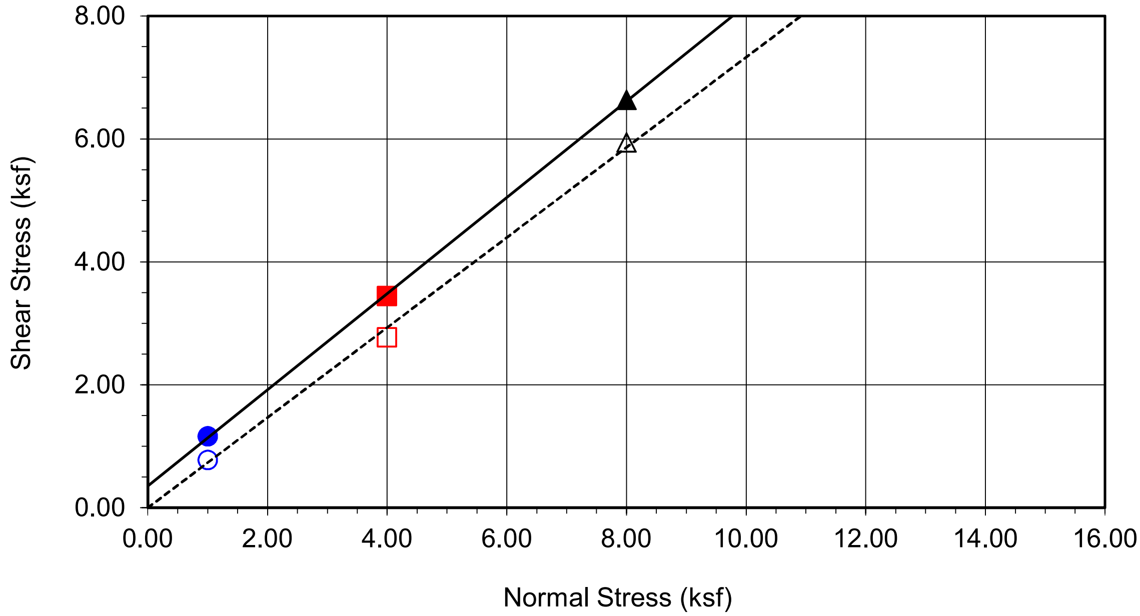
Project No.: 13384.001

Rexford Live Oak Irwindale

01-22

DS LB-1, B-1 @ 0-5





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	95% Remold

Soil Identification:
Olive brown silty sand with gravel (SM)g

Strength Parameters

	C (psf)	ϕ (°)
Peak	354	38
Ultimate	0	36

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.160	■ 3.442	▲ 6.630
Shear Stress @ End of Test (ksf)	○ 0.777	□ 2.770	△ 5.939
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 (%)	8.90	8.90	8.90
Dry Density (pcf)	120.7	120.8	120.9
Saturation (%)	60.6	60.9	61.0
Soil Height Before Shearing (in.)	0.9910	0.9852	0.9696
Final Moisture Content (%)	13.3	12.8	12.6

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13384.001

Rexford Live Oak Irwindale

01-22

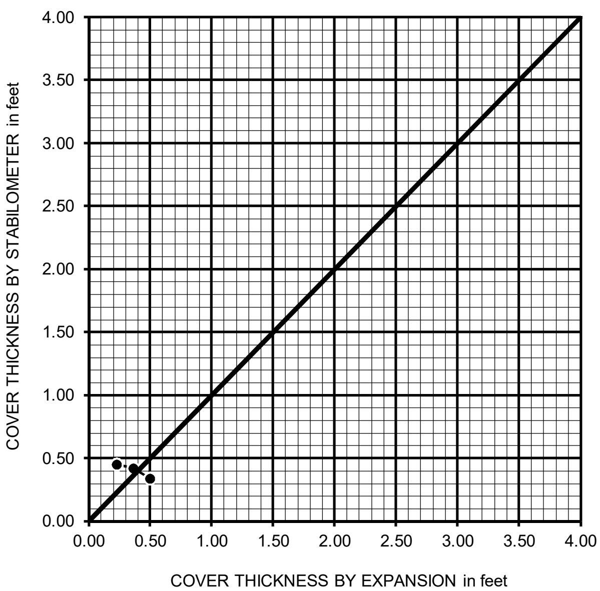
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Rexford Live Oak Irwindale	PROJECT NUMBER:	13384.001
BORING NUMBER:	LB-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	A. Santos
SAMPLE DESCRIPTION:	Olive brown silty sand with gravel (SM)g	DATE COMPLETED:	1/17/2022

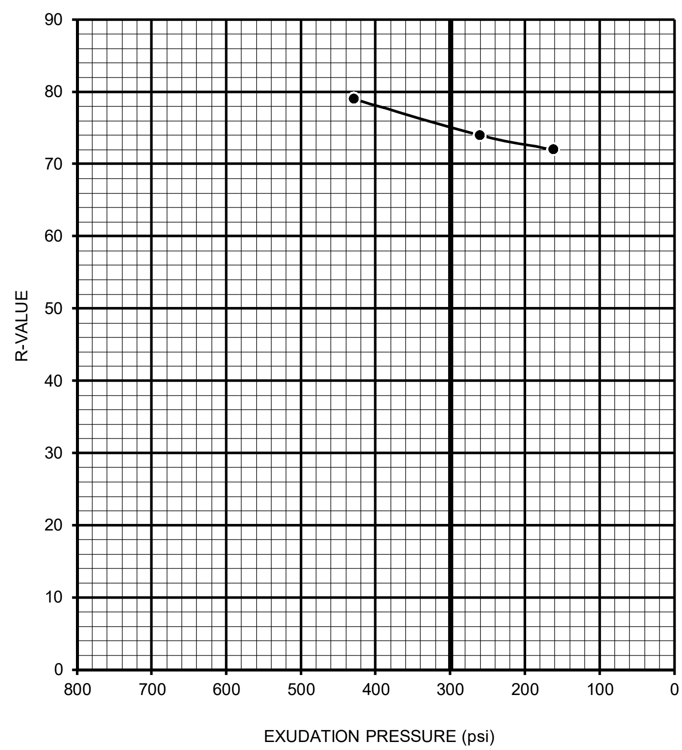
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	15.7	15.9	16.2
HEIGHT OF SAMPLE, Inches	2.48	2.50	2.54
DRY DENSITY, pcf	114.7	114.1	113.7
COMPACTOR PRESSURE, psi	250	225	200
EXUDATION PRESSURE, psi	429	261	162
EXPANSION, Inches x 10exp-4	15	11	7
STABILITY Ph 2,000 lbs (160 psi)	20	24	26
TURNS DISPLACEMENT	4.70	5.00	5.10
R-VALUE UNCORRECTED	79	74	72
R-VALUE CORRECTED	79	74	72

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.	0.34	0.42	0.45
EXPANSION PRESSURE THICKNESS, ft.	0.50	0.37	0.23

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	75
R-VALUE BY EXUDATION:	75
EQUILIBRIUM R-VALUE:	75

Leighton

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Rexford Live Oak Irwindale   Tested By : G. Berdy   Date: 01/13/22

Project No. : 13384.001   Checked By: A. Santos   Date: 01/21/22

Boring No.	LB-1			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SM)g			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.16			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	303			
Crucible No.	12			
Furnace Temperature (°C)	860			
Time In / Time Out	8:00/8:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.7543			
Wt. of Crucible (g)	20.7514			
Wt. of Residue (g) (A)	0.0029			
PPM of Sulfate (A) x 41150	119.34			
PPM of Sulfate, Dry Weight Basis	**119**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	120			
PPM of Chloride, Dry Wt. Basis	**120**			

pH TEST, DOT California Test 643

pH Value	**8.63**			
Temperature °C	22.0			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Rexford Live Oak Irwindale	Tested By :	J. Domingo Date: 01/20/22
Project No. :	13384.001	Checked By:	A. Santos Date: 01/21/22
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	B-1		

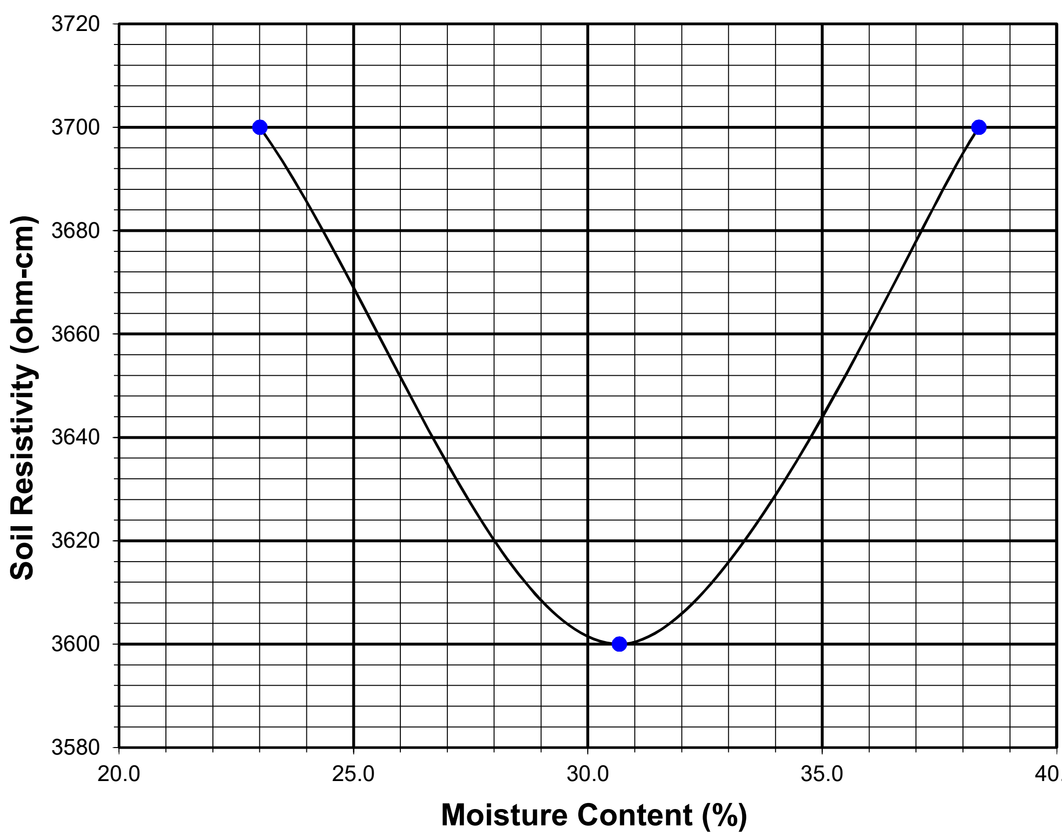
Soil Identification:* Olive brown (SM)g

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.01	3700	3700
2	40	30.67	3600	3600
3	50	38.34	3700	3700
4				
5				

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

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
3600	**30.7**	**119**	**120**	**8.63**	**22.0**



APPENDIX D

GEOPHYSICAL EVALUATION (ATLAS, 2022)

6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

February 4, 2022

Atlas No. 122006SWG
Report No. 1

MR. JEFF PFLUEGER, PG, CEG
LEIGHTON
17781 COWAN
IRVINE, CA 92614

Subject: Geophysical Evaluation
14005 Live Oak Avenue
Irwindale, California

Dear Mr. Pflueger:

In accordance with your authorization, Atlas Technical Consultants has performed a geophysical evaluation pertaining to the project located at 14005 Live Oak Avenue, Irwindale, California (Figure 1). The purpose of our study was to perform two one-dimensional (1-D) seismic surface wave refraction microtremor (ReMi) profiles to be used for design and construction at the study site. This letter report presents our methodology, equipment used, analysis, and findings. Our services were conducted on January 17, 2022.

Our scope of services for the project included the performance of 1-D ReMi profiles, labeled RL-1 and RL-2. (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional sounding which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low-velocity zones (velocity inversions) are detectable with ReMi. Our ReMi evaluation included the use of a 24-channel Geometrics Geode seismograph and 24, 4.5-Hz vertical component geophones. For both RL-1 and RL-2, the geophones were spaced 10 feet apart for a total line length of 230 feet. Fifteen records, each 32 seconds long, were recorded and then downloaded to a computer. The data was later processed using Surface Plus 9.1 - Advanced Surface Wave Processing Software (Geogiga Technology Corp., 2020), which uses the refraction microtremor method (Louie, 2001) and other surface wave analysis methods. The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional shear-wave velocity model of the site with roughly 85 to 95 percent accuracy.

Figure 3 depicts the general site conditions in the study area. Table 1, Figures 4a and 4b present the results from our evaluation for both seismic lines (RL-1 and RL-2).

Based on our analysis of the collected data for RL-1 and RL-2, the average characteristic site shear-wave velocities down to a depth of 100 feet are 1,648 and 1,598 feet per second (ft/s) respectively (CBC, 2019). These values correspond to a site classification of C. It should be noted the ReMi results represent the average condition across the length of each line.

Table 1 – ReMi Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1 (E-W)	0 – 6.32	834
	6.32 – 16.06	1102.7
	16.06 – 29.95	1617.7
	29.95 – 43.65	1803.9
	43.65 – 68.05	1918.2
	68.05 – 91.27	1941.8
	91.27 – 100	2552.1
RL-2 (N-S)	0 – 6.58	786.1
	6.58 – 15.94	1065
	15.94 – 30.39	1680
	30.39 – 44.39	1793.4
	44.39 – 69.77	1888.7
	69.77 – 96.37	1882
	96.37 – 100	2507.7

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluating will be performed upon request.

This document is intended to be used only in its entirety. No portions of the document, by itself, is designed to completely represent any aspect of the project described herein. Atlas should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended exclusively for use by the client. Any use of or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have questions related to this report, please call us at (858) 527-0849.

Respectfully submitted,
Atlas Technical Consultants LLC

Orion Adah
Project Geophysicist

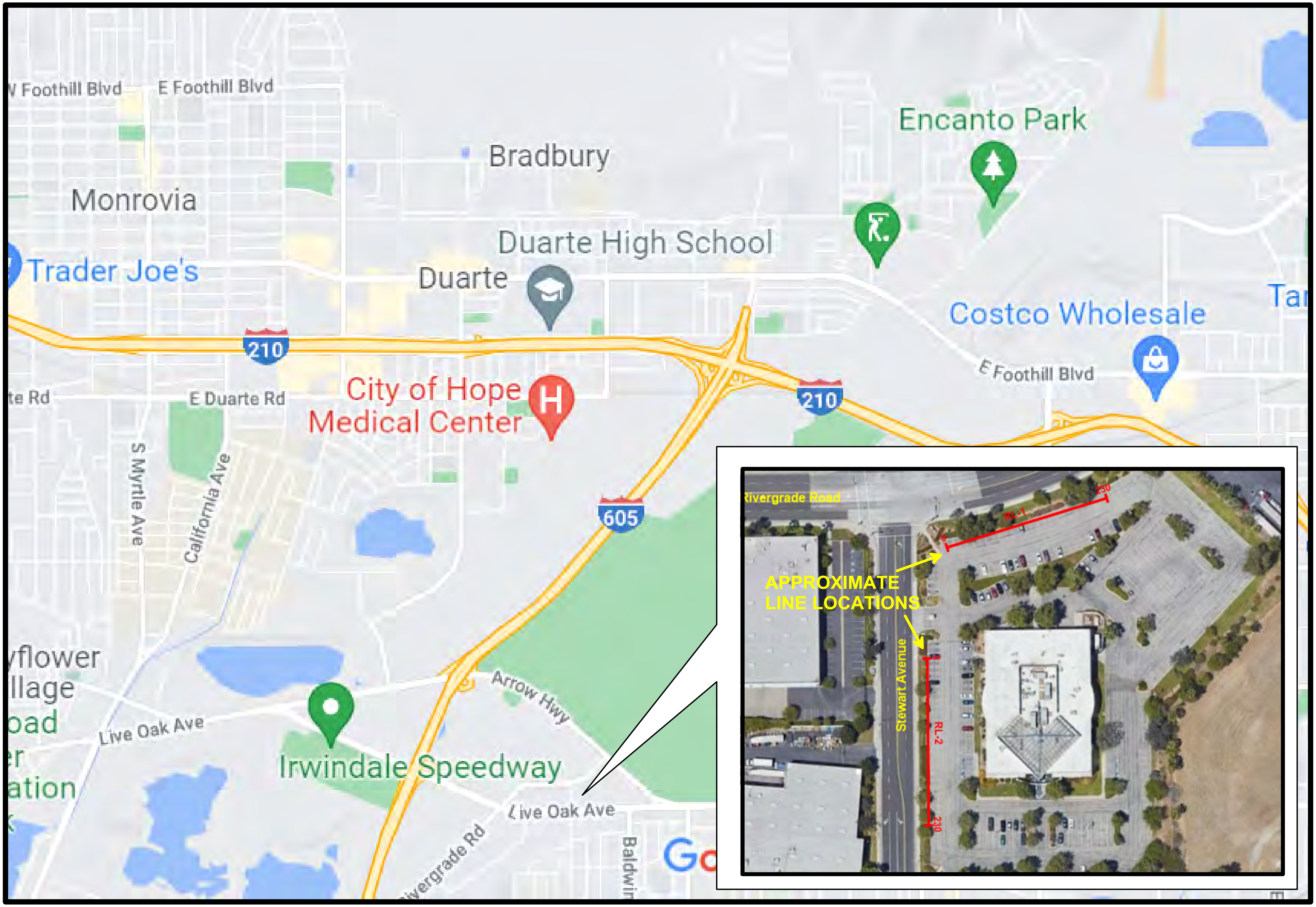
Patrick F. Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED PROFESSIONAL GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/2022
STATE OF CALIFORNIA

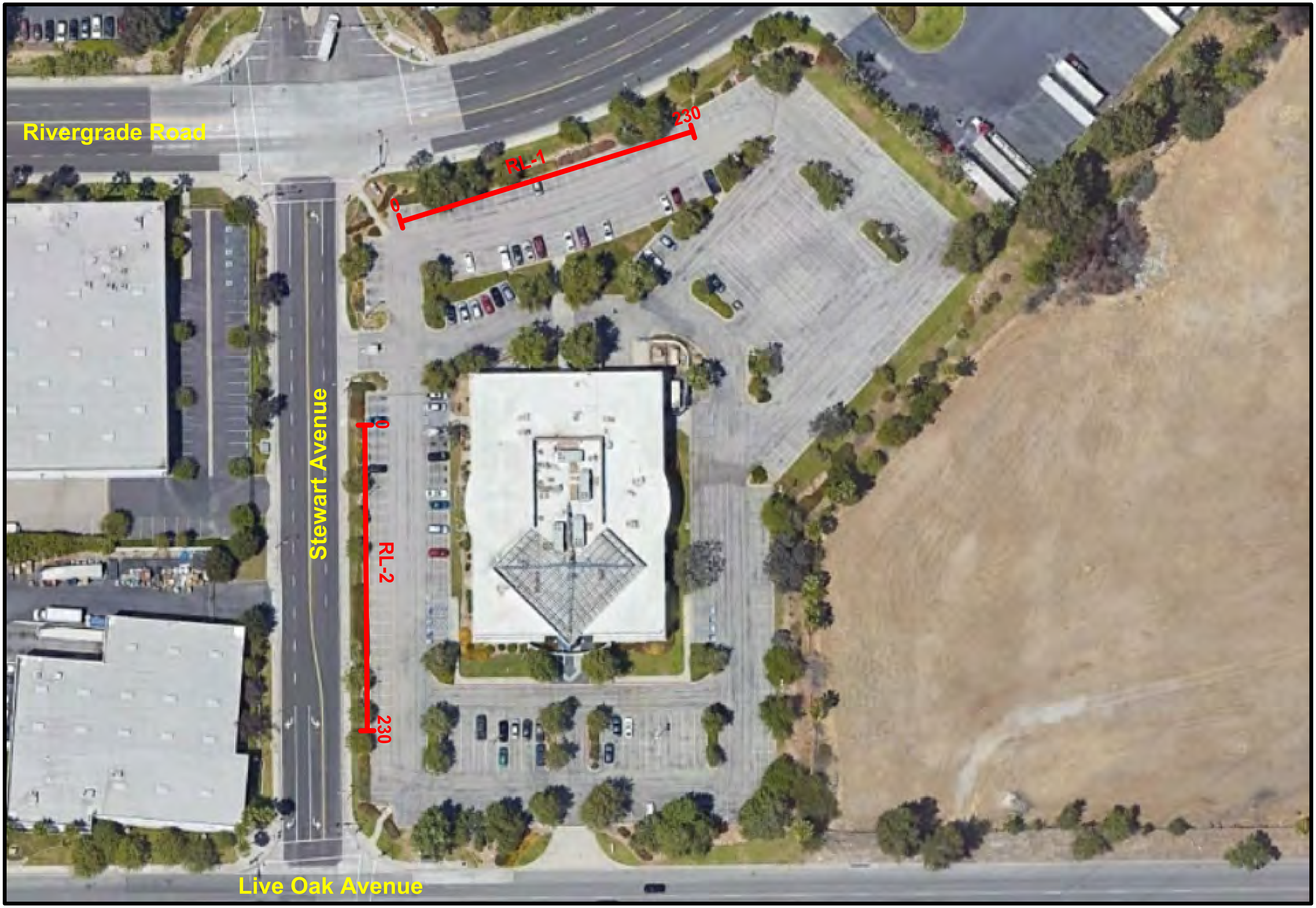
OAA:PFL:pfl:ds

Attachments: Figure 1 – Site Location Map
Figure 2 – Seismic Line Location Map
Figure 3 – Site Photographs
Figure 4a – ReMi Results RL-1
Figure 4b – ReMi Results RL-2

Distribution: jpflueger@leightongroup.com



SITE LOCATION MAP	N	14005 Live Oak Avenue Irwindale, California Project No.: 122006SWG Date: 02/22	ATLAS Figure 1



SEISMIC LINE LOCATION MAP

N

14005 Live Oak Avenue
Irwindale, California

Project No.: 122006SWG

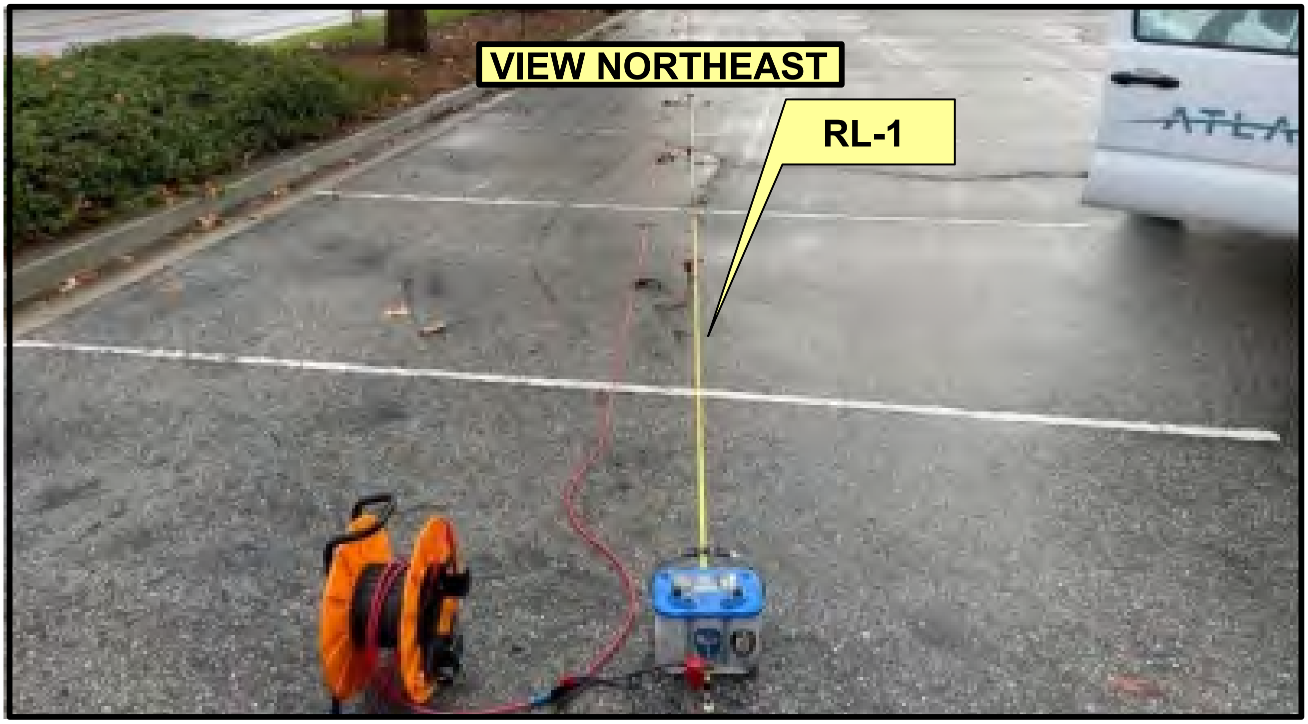
Date: 02/22

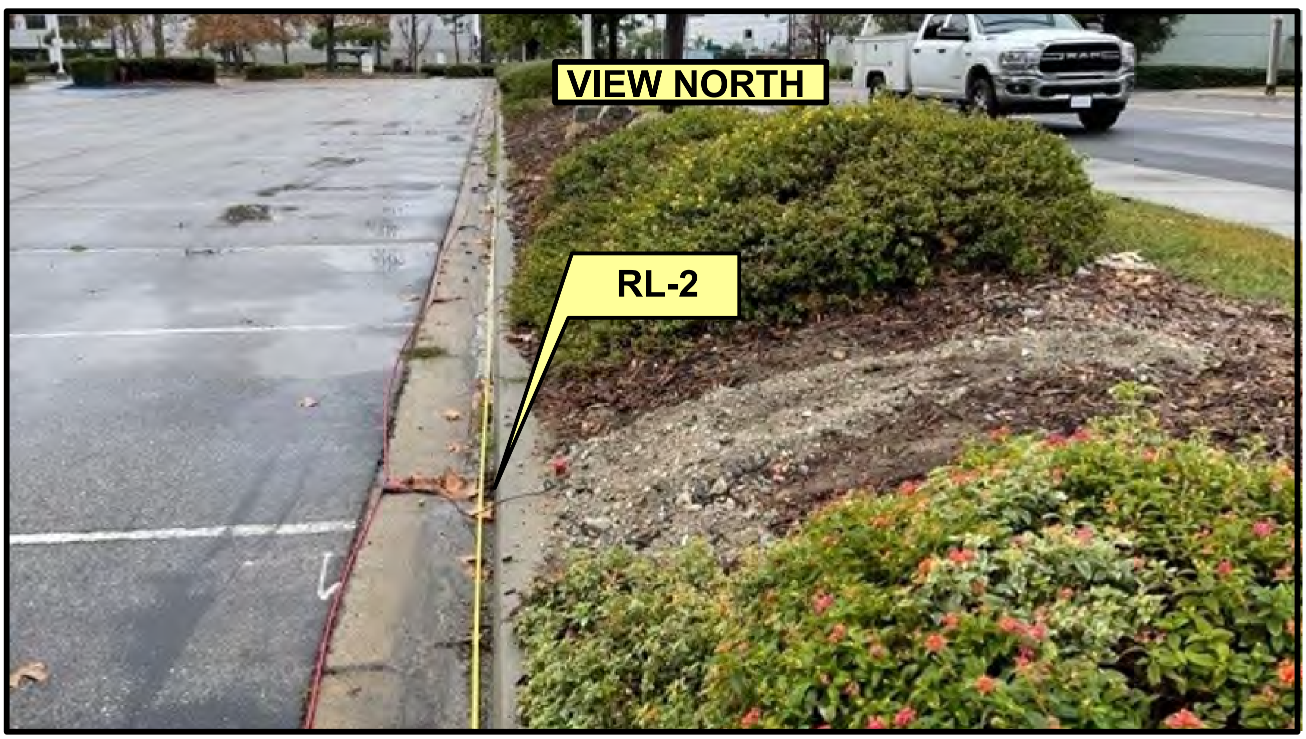
ATLAS

Figure 2

0 125 250

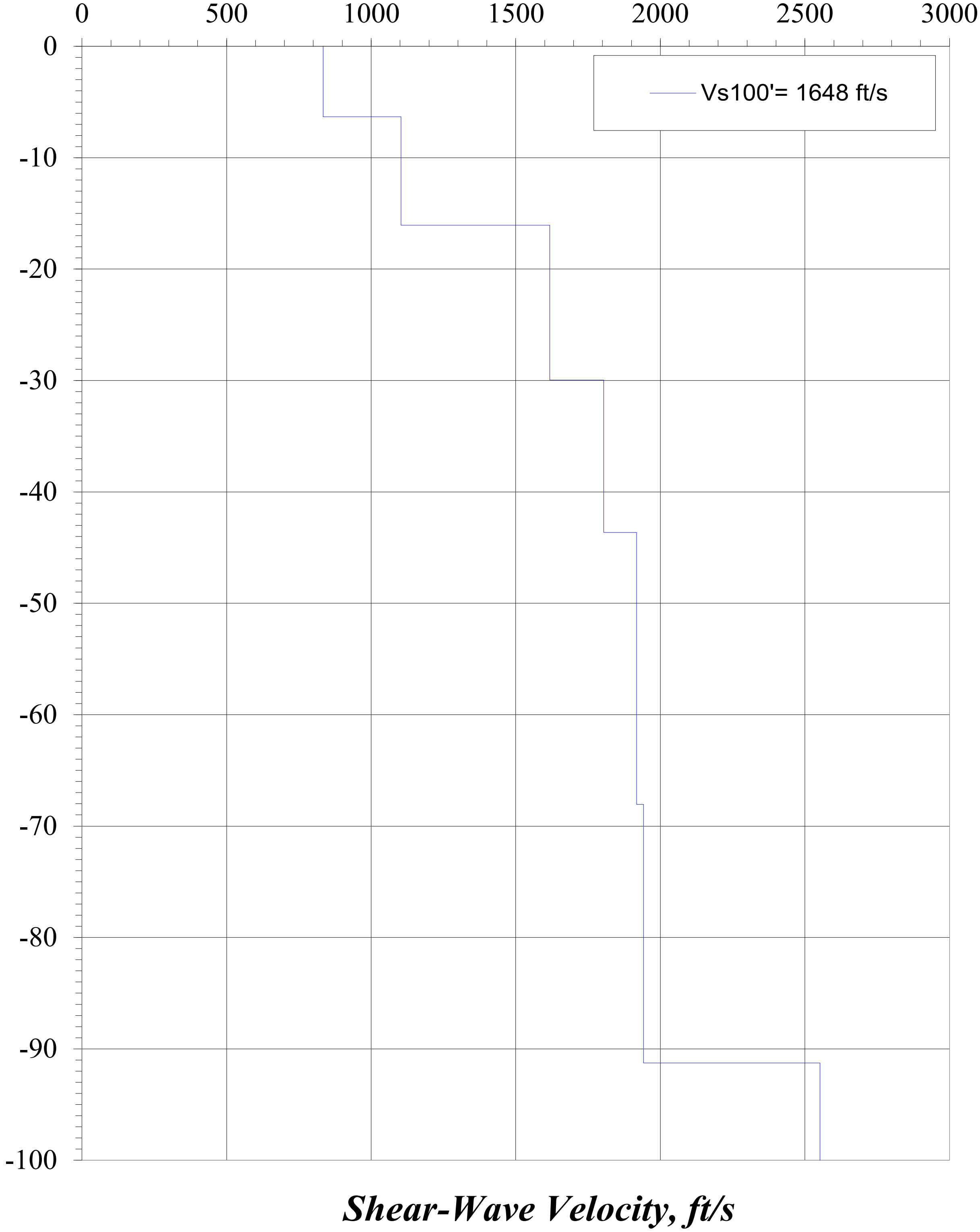
approximate scale in feet



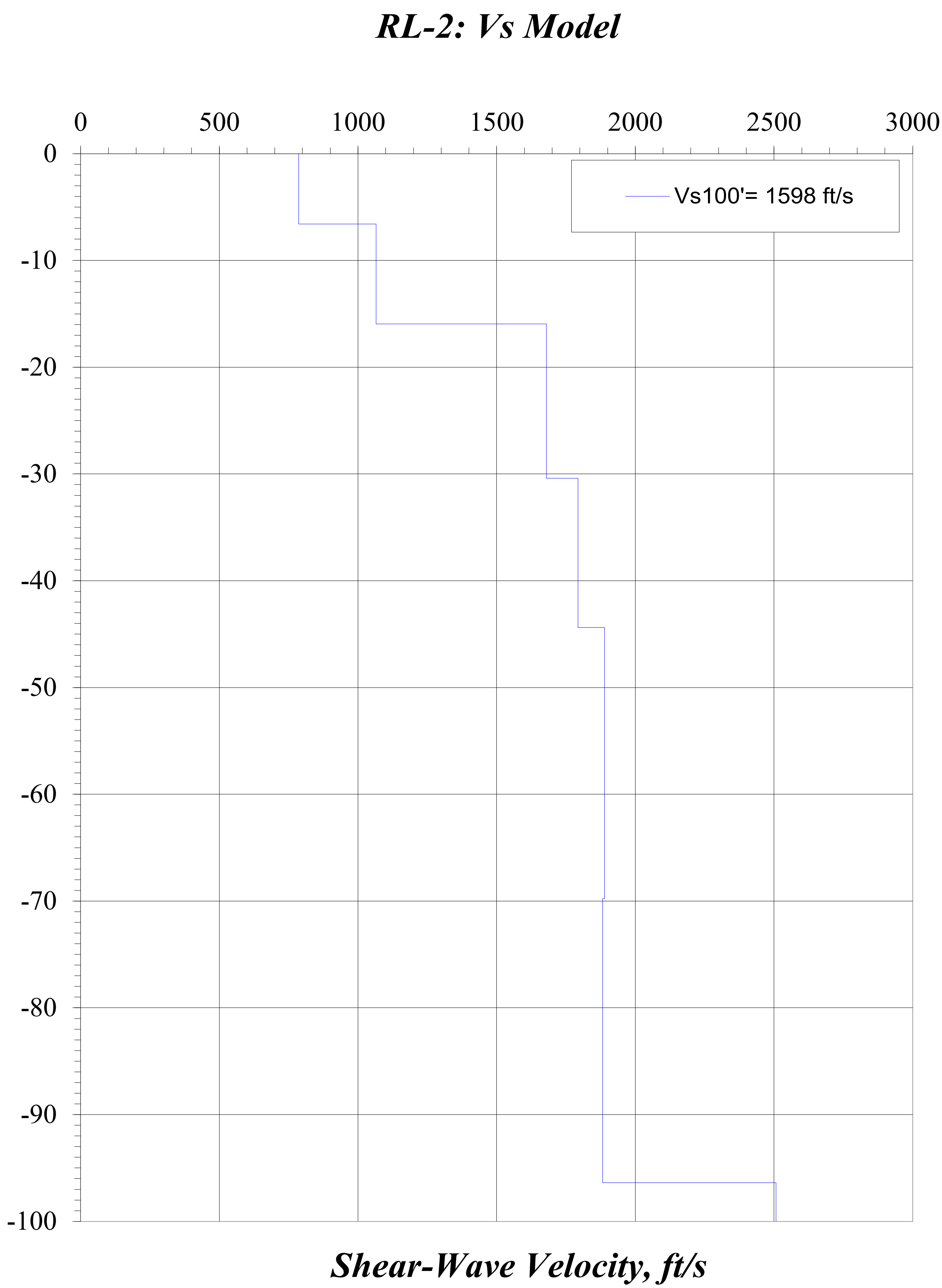


SITE PHOTOGRAPHS	14005 Live Oak Avenue Irwindale, California	ATLAS Figure 3
	Project No.: 122006SWG — Date: 02/22	

RL-1: Vs Model



ReMi RESULTS **RL-1**	14005 Live Oak Avenue Irwindale, California	ATLAS
	Project No.: 122006SWG · Date: 02/22	Figure 4a



ReMi RESULTS
RL-2

14005 Live Oak Avenue
Irwindale, California

Project No.: 122006SWG

Date: 02/22

ATLAS

Figure 4b