



PRELIMINARY GEOTECHNICAL EXPLORATION REPORT
PROPOSED INDUSTRIAL BUILDING
16201 RAYMER STREET
VAN NUYS DISTRICT OF LOS ANGELES CALIFORNIA

Prepared For **16201 RAYMER STREET LLC**
C/O LONGPOINT REALTY PARTNERS
116 HUNTINGTON AVENUE, SUITE 1001
BOSTON, MASSACHUSETTS 02116

Prepared By **LEIGHTON CONSULTING, INC.**
2600 MICHELSON DRIVE, SUITE 400
IRVINE, CALIFORNIA 92612

Project No. 13918.001

July 27, 2023 (Revised September 1, 2023)

July 27, 2023

Revised September 1, 2023

Project No. 13918.001

16201 Raymer Street LLC
c/o Longpoint Realty Partners
116 Huntington Avenue, Suite 1001
Boston, Massachusetts 02116

Attention: Mr. Andy Torres (9th St. Partners)

**Subject: Preliminary Geotechnical Exploration Report
Proposed Industrial Warehouse Building
16201 Raymer Street
Van Nuys District of Los Angeles, California**

In accordance with our proposal, authorized on May 11, 2023, Leighton Consulting, Inc. (Leighton) has prepared this preliminary geotechnical exploration report for the subject project (site). Development of the site will include demolition of existing buildings/improvements and construction of a new on-grade one-story industrial building encompassing 123,468 square feet of space. The proposed building will be constructed within the southern and central portion of the site along with associated truck loading on the northern side of the building, Los Angeles Fire Department access, and vehicular surface parking. Ancillary improvements are expected to include utility infrastructure, pavement, flatwork, and landscaping.

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

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

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



Respectfully submitted,

LEIGHTON CONSULTING, INC.

Jeff L. Hull, CEG 2056
Associate Geologist
Extension 4265, jhull@leightongroup.com



Christian Delgadillo, PE, GE 3144
Associate Engineer
Extension: 4218, cdelgadillo@leightongroup.com

LFO/JLH/CD/lr/dlm

Distribution: (1) Addressee

TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
1.0 INTRODUCTION	1
1.1 Site Description and Proposed Development	1
1.2 Purpose and Scope	2
2.0 GEOTECHNICAL FINDINGS.....	5
2.1 Regional Geologic Setting	5
2.2 Surficial Geology.....	5
2.3 Subsurface Conditions.....	5
2.3.1 Expansive Soil Characteristics	6
2.3.2 Soil Corrosivity	6
2.3.3 Soil Compressibility.....	7
2.3.4 Shear Strength.....	7
2.3.5 Excavation Characteristics	7
2.4 Groundwater Conditions	7
2.5 Infiltration	8
2.6 Surface Fault Rupture.....	9
2.7 Strong Ground Shaking	10
2.8 Liquefaction Potential.....	11
2.9 Seismically-Induced Settlement.....	11
2.10 Lateral Spreading	11
2.11 Earthquake-Induced Landslides	12
2.12 Flooding	12
2.13 Seiches and Tsunamis.....	12
3.0 PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS	13
3.1 Site Grading.....	13
3.1.1 Site Preparation	13
3.1.2 Removals and Overexcavations.....	14
3.1.3 Excavation Bottom Preparation.....	14
3.1.4 Fill Materials	14
3.1.5 Fill Placement and Compaction.....	15
3.1.6 Reuse of Concrete and Asphalt Rubble	15
3.1.7 Shrinkage and Bulking Factors	15
3.2 Foundation Design.....	16
3.3 Slabs-on-Grade	16
3.4 Cement Type and Corrosion Protection.....	17
3.5 Retaining Walls.....	17
3.6 Paving.....	19
3.6.1 Asphalt Concrete.....	19

<u>Section</u>	<u>Page</u>
3.6.2 Portland Cement Concrete Paving.....	20
3.6.3 Base Course	21
3.7 Temporary Excavations	21
3.8 Trench Backfill	22
3.9 Drainage and Landscaping	22
3.10 Additional Geotechnical Services	22
4.0 LIMITATIONS.....	24
5.0 REFERENCES.....	25

Tables

Table 1 – Measured (Unfactored) Infiltration Rate	9
Table 2 – 2022 CBC Based Ground Motion Parameters (Mapped Values)	10
Table 3 – Retaining Wall Design Earth Pressures	18
Table 4 – Asphalt Concrete Pavement Sections	20
Table 5 – PCC Pavement Sections.....	20

Attachments

Important Information About Your Geotechnical Engineering Report	Rear of Text
Figure 1 – Site Location Map	Rear of Text
Figure 2 – Geotechnical Map	Rear of Text
Figure 3 – Regional Geology Map	Rear of Text
Figure 4 – Regional Fault Map and Historical Seismicity Map	Rear of Text
Figure 5 – Seismic Hazard Map	Rear of Text
Figure 6 – Flood Hazard Zone Map	Rear of Text
Appendix A – Exploration Logs	
Appendix B – Percolation Test Data	
Appendix C – Laboratory Test Results	
Appendix D – Earthwork and Grading Guide Specifications	

1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site address is designated as 16201 Raymer Street in the Van Nuys district of Los Angeles, California. The site location (latitude 34.218374°, longitude -118.486296°) and immediate vicinity are shown on Figure 1, *Site Location Map*.

The site is roughly rectangular in shape and encompasses approximately 5.5 acres. The site is bordered by Raymer Street to the south, existing commercial properties to the east and west, and a Southern Pacific Railroad easement to the north. The site is currently occupied by three (3) existing commercial buildings with variable surface covering of Portland Cement Concrete (PCC) and Asphalt Concrete (AC) for access and parking. Access to the site is via two main drive approaches fronting Raymer Street.

The project site is relatively flat, with surface drainage generally directed to the south. Review of the United States Geological Survey (USGS) 7.5-Minute Van Nuys Quadrangle (USGS, 1996) indicates topography at the site ranges from approximately Elevation (El.) +790 to +800 feet mean sea level (msl).

Based on a review of historical aerial photographs (NETR, 2023), the site was vacant and undeveloped until at least 1952. By approximately 1967, two of the three currently existing buildings and surrounding parking areas existed, with the third existing building constructed by approximately 1977. The site has essentially remained unchanged since that time.

Based on review of the *Concept Site Plan* for the site, prepared by Herdman Architects + Design, dated January 31, 2023, we understand the proposed development concept for the site will consist of a new one-story Class A industrial building with a total building area of 123,468 square feet. The proposed building will be situated in the southern portion of the site and constructed at grade with truck loading planned on the northern side of the building. It should be noted that the proposed building footprint is located within approximately 5 feet from the southern property boundary. Vehicular surface parking is planned on northern, eastern, and western portions of the site. Ancillary improvements are expected to consist of utility infrastructure, pavement, flatwork, and landscaping. As shallow percolation test results yield low rates of infiltration, we do not expect any site stormwater system design will incorporate subsurface infiltration elements.

Structural loading information was not yet available at the time this report was prepared.

1.2 Purpose and Scope

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

- Background Review – Review readily available in-house geotechnical reports, literature, aerial photographs, and maps relevant to the site. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Section 5.0.
- Pre-Field Exploration Activities – Site Reconnaissance to mark proposed exploration locations. Underground Service Alert (DigAlert) was notified to locate and mark existing underground utilities prior to our subsurface exploration.
- Field Exploration – Our subsurface exploration was performed on June 12, 2023, including drilling, logging, and sampling of two (2) hollow-stem auger soil borings (designated LB-3 and LB-4) to depths of approximately 21.5 and 51.5 feet below the existing ground surface (bgs), respectively. One (1) additional soil boring (designated LP-2) was drilled to an approximate depth of 10 feet bgs for percolation testing. The approximate locations of the explorations are shown on Figure 2, *Geotechnical Map*. The boring logs are presented in Appendix A, *Exploration Logs*.

Bulk and relatively undisturbed drive samples were obtained during drilling for geotechnical laboratory testing. Driven ring samples were collected from the borings using a Modified California ring-lined sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPTs) were also performed within the borings in accordance with ASTM Test Method D 1586. Samples were collected at 5-foot intervals throughout the depth of exploration, with the exception of the upper ten feet of boring LB-3, which were collected at 2-foot intervals. In both test methods, the sampler is driven below the bottom of the borehole by a 140-pound weight (hammer) free-falling 30 inches. The drilling rig was equipped with an automatic hammer to provide greater

consistency in the drop height and striking frequency. The number of blows to drive the sampler the final 12 inches of the 18-inch drive interval is termed the “blowcount” or SPT N-value. The N-values provide a measure of relative density in granular (non-cohesive) soils and comparative consistency in cohesive soils. The number of blows per 6 inches of penetration was recorded on the boring logs, see Appendix A.

The borings were logged in the field by a geologist from our firm. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory for testing. After completion of drilling, the borings were backfilled to the ground surface with bentonite grout and patched with cold-mix asphalt concrete at the surface to match existing conditions.

- *Percolation Testing* – Boring LP-2 was converted to a temporary percolation test well upon completion of drilling and sampling. The test well 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 June 13, 2023 in general accordance with the County of Los Angeles *Guidelines for Geotechnical Investigation and Reporting – Low Impact Development Stormwater Infiltration* (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 the boring and the boring was 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 D 3080),
 - Consolidation (ASTM D 2435),
 - Maximum Dry Density (ASTM D 1557),
 - Passing No. 200 Sieve (ASTM D 1140),
 - Expansion Index (ASTM D 4829),

- R-value; and
- Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

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

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

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The site is located within the Transverse Ranges geomorphic province of Southern California. The Transverse Ranges are a belt of east/west-trending folds and associated thrusts that formed in response to northeast crustal shortening that initiated in Pliocene time, approximately 4 to 5 million years before the present (Yeats, 1988). Regional north-south compression caused the units to become progressively folded and faulted, forming valleys and uplands that are generally bounded by reverse/thrust faults, which generally dip north along the southern range fronts and dip south along the northern range fronts.

More locally, the site is centrally located within the San Fernando Valley, an east/west-trending structural trough bounded by the San Gabriel Mountains to the north and the Santa Monica Mountains to the south. The valley bottom is blanketed with Quaternary-aged alluvial deposits (Yerkes and Campbell, 2005; Bedrossian and Roffers, 2012).

2.2 Surficial Geology

Geologic mapping of the project area assigns near-surface soils as Quaternary-aged (Holocene and late Pleistocene) young alluvial fan deposits. These alluvial sediments are comprised of unconsolidated to slightly consolidated boulders, cobbles, gravel, sand and silt issued from a confined valley or canyon (Yerkes and Campbell, 2005; Bedrossian and Roffers, 2012). The surficial geologic units mapped in the vicinity of the project site are presented on Figure 3, *Regional Geology Map*.

2.3 Subsurface Conditions

The locations of the borings are shown on Figure 2, *Geotechnical Map*. Based on our logging of subsurface materials during drilling, the site is underlain by a relatively thin mantle of undocumented artificial fill materials (Afu) overlying Quaternary-aged (Holocene to late Pleistocene) young alluvial fan deposits (Qyf). The undocumented artificial fill (Afu) encountered in our borings at the explored locations is generally on the order of 3 feet in thickness. The fill materials encountered in our borings generally consist of dark brown sandy clay, clayey sand and silty clay. Localized thicker accumulations of the fill may exist beneath existing buildings and/or between borings. The underlying young alluvial fan sediments

(Qyf) were encountered to the maximum depth explored (51½ feet bgs). This unit is generally described as medium stiff to stiff, clay, silt, silty clay, sandy silt, and sandy clay; and medium dense sand and silty sand.

Detailed descriptions of encountered subsurface materials are presented on the logs included in attached Appendix A. Engineering properties of the soils are described in the following sections.

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.

A bulk sample of representative near-surface soil was obtained during our subsurface exploration and tested for Expansion Index (EI) potential. The results of EI testing indicate a value of 42, categorized as a low potential for expansion. The EI laboratory test results are attached in Appendix C.

For purposes of this report and based upon visual characterization of alluvial materials at approximate foundation depth, a low expansion potential for onsite materials may be considered to support design and verified upon completion of earthwork grading. As the expansion potential of onsite soils can vary, we recommend additional testing be performed upon completion of grading to verify the suitability of parameters presented in this report.

2.3.2 Soil Corrosivity

One (1) near-surface bulk soil sample obtained during our subsurface exploration was tested for corrosivity to assess corrosion potential to buried concrete. The test result indicates a soluble sulfate concentration of 103 parts per million (ppm), chloride content of 80 ppm, pH value of 7.35, and minimum resistivity value of 1,480 ohm-cm. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C of this report.

The results of the resistivity tests indicate the underlying soil is severely 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 Soil Compressibility

Two (2) samples of site soil recovered from our borings were subjected to consolidation testing to evaluate the compressibility of these materials under assumed loads and anticipated structural bearing stresses. The results of testing indicate these soils exhibit a low to moderate compressibility potential. The results of testing performed as a part of this study are presented in Appendix C.

2.3.4 Shear Strength

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

2.3.5 Excavation Characteristics

Based on drilling difficulty/ease and our grading experience in the nearby area, we anticipate the onsite artificial fill and native earth materials can readily excavated using conventional excavation equipment in good operating condition.

2.4 **Groundwater Conditions**

Groundwater was not encountered during our subsurface exploration performed at the site to a maximum depth of 51½ feet bgs. Based on review of the California Geological Survey's (CGS) *Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle* (CGS, 1997) historically high groundwater in the area is on the order of approximately 80 feet bgs. In addition, based on review of available groundwater information from the County of Los Angeles Department of Public Works for two nearby groundwater monitoring wells located approximately 0.9 mile

southwest of the project site (State Wells # 2N15W31N01 and # 2N15W31N02), the shallowest groundwater level measured for a monitoring period between April 1974 to April 2021 was approximately 113.0 feet bgs in April 1974.

Based on these findings, groundwater is not expected to pose a constraint during or after construction. Fluctuations of the groundwater levels, localized zones of perched water, and an increase in soil moisture, should be anticipated during /following rainy seasons or periods of locally intense rainfall or storm water runoff, or from stormwater infiltration.

2.5 Infiltration

A shallow field percolation test was performed to evaluate the infiltration characteristics of subsurface soils. The test was performed within a temporary well LP-2 located in the southwest portion of the site. The test zone ranged in depth between approximately 8 to 10 feet bgs. The percolation test was conducted in general accordance with Los Angeles County guidelines (LADPW, 2021). Results of the percolation testing are presented in Appendix B. The test location is shown on Figure 2.

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

The falling-head test method was employed for testing within LP-2, in which the volume of discharge was calculated by adding the total volume of water that dropped within the PVC pipe and within the annulus, and incorporating a porosity reduction factor to account for the porosity of the annulus material. The infiltration surface area was calculated based on the average water height within the test well. The infiltration rate was calculated by dividing the rate of discharge by the infiltration surface area, or flow area.

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

Table 1 – Measured (Unfactored) Infiltration Rate

Test Well Designation	Approximate Depth of Test Zone (feet bgs)	Measured Infiltration Rate (inch per hour)	Design Infiltration Rate (inch per hour)
LP-2	8 to 10	0.15	0.08

Per the guidelines by County of Los Angeles Department of Public Works (2021), the measured infiltration rates should be reduced by the design professional with a correction factor (i.e., reduction factor) to determine design values representative of long-term infiltration system performance. Note that a reduction factor is intended to account for siltation, plugging, and maintenance, and should be applied to the design infiltration rates provided in Table 4. This reduction factor ranges from 1 to 3 and should be selected based on the specified levels of pre-treatment and maintenance requirements of the proposed infiltration system. Based on the test type, number of tests performed and site variability, we recommend a minimum correction factor of 2.0 be incorporated into system design.

In accordance with the County of Los Angeles guidelines infiltration (2021), subsurface soils with a minimum design infiltration rate of 0.3 inches per hour is required for stormwater infiltration systems. Based on the tested infiltration rate and the County guidelines, an onsite stormwater infiltration at the tested depths is not considered feasible from a geotechnical standpoint.

2.6 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 designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 2018). Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

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 Northridge Hills fault, Mission Hills fault and Verdugo fault, located approximately 1.4 miles, 4.5 miles and 5.2 miles from the site, respectively. The San Andreas fault, which is the largest active fault in California, is approximately 30.9 miles northeast of the site. Major regional faults

with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.7 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	Code-Base
Site Latitude	34.218374
Site Longitude	-118.486296
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	2.039
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.689
Short Period (0.2 sec) Site Coefficient, F_a	1
Long Period (1 sec) Site Coefficient, F_v	1.7
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	2.039
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	1.171
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.359
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.781
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.912
¹ A ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.	

2.8 Liquefaction Potential

The term liquefaction is generally referenced to loss of strength and stiffness in soils due to build-up of pore water pressure when subject to cyclic or monotonic loading. Both sandy and clayey soils are susceptible to loss of strength and stiffness. Because of the difference in strength characteristic and methods for evaluating strength loss potential for granular and clayey soils, the term liquefaction is used for granular soils while cyclic softening is used for fine-grained soils (i.e. clays and plastic silts).

In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically-induced lateral ground deformations such as lateral spreading. Depending upon the relative thickness of the liquefied strata with respect to overlying non-liquefiable soils, other potentially adverse effects such as ground oscillation and ground fissuring may occur.

As shown on the *Seismic Hazard Zones* map for the Van Nuys Quadrangle (CGS, 1998), the site is **not** located within a liquefaction hazard zone (Figure 5, *Seismic Hazard Map*). In addition, the current and historic high groundwater levels are significantly greater than 50 feet. Based on these findings, liquefaction is **not** considered a hazard at the site.

2.9 Seismically-Induced Settlement

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

Based on our evaluation of the alluvial site soils, the total seismically-induced settlement is estimated to be on the order of 1 inch. The differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

2.10 Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since the site is relatively flat and

constrained laterally, and liquefaction is not considered a hazard at the site, earthquake-induced lateral spreading is also **not** considered a hazard at the site.

2.11 Earthquake-Induced Landslides

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

2.12 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 south. As shown on Figure 6, *Flood Hazard Zone Map*, the site is **not** located within a 100-year or 500-year 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 6, *Flood Hazard Zone Map*, the site **is** mapped within a flood impact zone from the Lower San Fernando Dam. However, due to the location and distance of the site from this dam, the potential for earthquake-induced flooding to occur due to a failure of this dam is considered low. Catastrophic failure of this dam is expected to be a very unlikely event in that dam safety regulations exist and are enforced by the Division of Safety of Dams, Army Corp of Engineers and Department of Water Resources. Inspectors may require dam owners to perform work, maintenance or implement controls if issues are found with the safety of the dam.

2.13 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 PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS

The preliminary recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. Additional exploration and/or evaluation may be required in the future once more detailed development plans become available. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to determine the effect upon the recommendations subsequently presented. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the City of Los Angeles, and other governing agencies.

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

3.1 Site Grading

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

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any existing site improvements, 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 footprints prior to fill placement.

3.1.2 Removals and Overexcavations

Foundation for the proposed building should be underlain by compacted fill to provide a uniform support and reduce potential for differential settlement. The compacted fill should extend a minimum 3 feet below bottom of the foundation and a minimum 3 feet beyond outside edges of the foundation, where feasible. Pavement areas, driveway, and concrete flatwork should be underlain by a minimum 1 foot of compacted fill.

Local conditions may be encountered that could require additional overexcavation beyond the above noted minimum to obtain an acceptable subgrade, including areas where undocumented fill is exposed at the removal bottom. The actual depths and lateral extents of remedial grading will be determined by Leighton, based on subsurface conditions encountered during grading. Any deeper removals should be extended an equal distance laterally beyond the limits of the removal bottom.

3.1.3 Excavation Bottom Preparation

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

3.1.4 Fill Materials

On-site soil that is free of construction debris, organics, cobbles, boulders, rubble, or rock larger than 4 inches in largest dimension is suitable to be used as fill for support of structures. Any imported fill soil should be approved by the geotechnical engineer prior to import or use onsite.

3.1.5 Fill Placement and Compaction

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

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

3.1.6 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 rubble 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.

3.1.7 Shrinkage and Bulking Factors

The following average shrinkage and bulking factors may be assumed for calculating earthwork volumes of the onsite soils:

- Shrinkage factor of 5 to 10 percent
- Bulking factor of 15 to 20 percent.

3.2 Foundation Design

Shallow spread footings established on engineered fill may be used to support the proposed building. Footings should be embedded a minimum 18 inches below the lowest adjacent grade. An allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for footings with a minimum width of 12 inches for continuous footings and 18 inches for isolated footings. The allowable bearing capacity may be increased by 250 psf for each additional foot of embedment depth or footing width to a maximum value of 3,000 psf. The bearing capacity may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

The recommended allowable bearing capacity for the foundation is generally based on a total allowable static settlement of 1 inch, with differential settlement taken as ½ inch over 30 feet.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance, a passive pressure of 390 psf per foot of depth to a maximum of 3,900 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, so the structural engineer should apply the applicable factors of safety and/or load factors during design. 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

From a geotechnical standpoint, we recommend slab-on-grade floor slab be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 18 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.

Exterior concrete slabs that are not subject to vehicular loading, such as patio slabs and sidewalks, should be at least 4 inches thick. We suggest that the exterior concrete slabs be reinforced with W/6x6-W1.4xW1.4 welded wire mesh.

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 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 and per ASTM STP 1013, 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. The corrosion information presented in this report should be provided to your underground utility subcontractors

3.5 Retaining Walls

The following recommendations may be used for design and construction of retaining structures at the site. We recommend that any permanent earth retaining structures be backfilled with onsite or import soil with Expansion Index (EI) of not greater than 50 (per ASTM D 4829).

Table 3 – Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	37
At-Rest (braced)	57
Passive Resistance (compacted fill)	390

Walls retaining more than 6 feet of soil should consider a seismic earth pressure increment with an inverted triangular distribution of 28 psf/foot in addition to the active earth pressure provided above. The above 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.

Cantilever walls that are designed for a deflection at the top of the wall of at least 0.001H, where H is equal to the wall height, may be designed using the active earth pressure condition. Rigid walls that are not free to rotate, walls that are braced at the top, and walls that provide indirect support for foundations should be designed using the at-rest condition.

Lateral load resistance will be provided by the sliding resistance at the base of the foundation and the passive pressure developed along the front of the foundation. A frictional resistance coefficient of 0.30 may be used at the concrete and soil interface. The lateral passive resistance can be taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

Retaining walls should be provided with a drainage system behind the wall to prevent build-up of hydrostatic pressure. 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 Green Book. The subdrain outlet should be connected to a free-draining outlet or sump. Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill, particularly where horizontal

space is limited adjacent to shoring (where walls are cast against shoring). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

3.6 Paving

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

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

3.6.1 Asphalt Concrete

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Based on design procedures outlined in the current Caltrans *Highway Design Manual* and a R-value of 12 based on the laboratory test results, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) ranging 5 to 9. Assuming that the paving subgrade will consist of engineered fill with an R-value equal or greater than 12, compacted to at least 90 percent as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on one (1) near surface sample of existing onsite soils indicate a value of 12.

Table 4 – Asphalt Concrete Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5	3.5	7.5
6	4.0	10.5
7	4.5	13.0
8	5.0	15.5
9	6.0	17.5

3.6.2 Portland Cement Concrete Paving

We have assumed that the subgrade below paving will have an R-value of at least 12. Portland cement concrete (PCC) paving sections were determined in accordance with procedures developed by the Portland Cement Association.

Concrete paving sections for a range of Traffic Indices are presented in the following table. We have assumed that the Portland cement concrete will have a compressive strength of at least 4,000 pounds per square inch (psi) for truck court and drive aisle, and at least 3,000 psi for parking stalls.

Table 5 – PCC Pavement Sections

Traffic Index	PCC (inches)	Base Course (inches)
5	5½	4
6	6½	4
7	7	5
8	7½	6
9	8½	6

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

3.6.3 Base Course

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

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

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

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements, Article 6, Section 1541, effective October 1, 1995. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a $\frac{3}{4}$ H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and $1\frac{1}{2}$ H:1V for Type C soils.

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

3.8 Trench Backfill

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

- (1) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater-than-or-equal-to ($\geq$) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), water densified in place, or
- (2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction*, (“Greenbook”), 2021 Edition. CLSM should not be jetted.

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

3.9 Drainage and Landscaping

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

3.10 Additional Geotechnical Services

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

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

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

4.0 LIMITATIONS

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

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

5.0 REFERENCES

- American Concrete Institute (ACI), 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, an ACI Standard, reported by ACI Committee 318.
- American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, with Supplement 1, Effective December 12, 2018.
- Bedrossian, T.L., Roffers, P., Hayhurst, C.A., Lancaster, J.T. and Short, W.R. (2012) Geologic Compilation of Quaternary Surficial Deposits in Southern California (Special Report 217). Department of Conservation, California Geological Survey, Sacramento.
- Bryant, W.A. and Hart, E.W., Interim Revision 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Geological Survey, Special Publications 42, 42p.
- California Building Standards Commission, 2022, 2022 California Building Code (CBC), California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on 2021 International Building Code, Effective January 1, 2022.
- California Department of Water Resources (DWR), 1961, Planned Utilization of the Ground Water Basins of the Coastal Plain of Los Angeles County, Bulletin No. 104, Appendix A – Ground Water Geology, June 1961.
- _____, 2022, Interactive Website, Water Data Library,
<http://wdl.water.ca.gov/waterdatalibrary/index.cfm>
- California Geological Survey (CGS; formerly California Division of Mines and Geology, CDMG), 1997, Seismic Hazard Zones Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 08.
- _____, 1998, Seismic Hazard Zones, Van Nuys, Official Map, Released February 1, 1998, map scale 1:24,000.
- _____, 2000, CD-ROM containing digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones that affect the Southern Region, DMG CD 2000-003 2000.
- _____, 2002, Note 49, Guidelines for Evaluating the Hazard of Surface Fault Rupture, dated May 2002.
- _____, 2008, Special Publication 117a, Guidelines for Evaluating and Mitigating Seismic Hazards in California.
- _____, 2010, Fault Activity Map of California, 2010.

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

County of Los Angeles Department of Public Works (LADPW) Geotechnical and Materials Engineering Division, 2013, Manual for Preparation of Geotechnical Reports, dated July 1, 2013.

_____, 2021, Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration, Geotechnical and Materials Engineering Division, GS200.1, dated June 30, 2021.

Dibblee, Jr., T.W., 1991, Geologic Map of the San Fernando and North ½ Van Nuys Quadrangles, Los Angeles County, California: Dibblee Geological Foundation Map DF-33, Santa Barbara, California, map scale 1:24,000.

Federal Emergency Management Agency, 2008, Flood Insurance Rate Map, Los Angeles County and Incorporated Areas, Map Number 06037C1280F, dated September 26, 2008.

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

Roffers, P.D., and Bedrossian, T.L., 2012, Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' x 60' Quadrangle (Revised), California Geological Survey (CGS) Special Report 217, Plate 9, Map Scale 1:100,000.

United States Geological Survey (USGS), 1996, Van Nuys Quadrangle Southern California 7.5 Minute Series Topographic Map, map scale 1:24,000.

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

_____, 2022a, Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>

_____, 2022b, Interactive Geologic Map, <http://ngmdb.usgs.gov/maps/MapView/>

Yeats, R.S., 1988, Late Quaternary slip rate on the Oak Ridge fault, Transverse Ranges, California: implications for seismic risk: Journal of Geophysical Research, v. 93, no. B10, p. 12,137-12,149.

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

Yerkes, R.F., and Campbell, R.H., 2005, Preliminary Geologic Map of the Los Angeles 30' x 60' Quadrangle, Southern California, United States Geological Survey: Open-File Report 2005-1019, Version 1.0, Map Scale 1:100,000.

Important Information about This Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

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

This Report’s Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

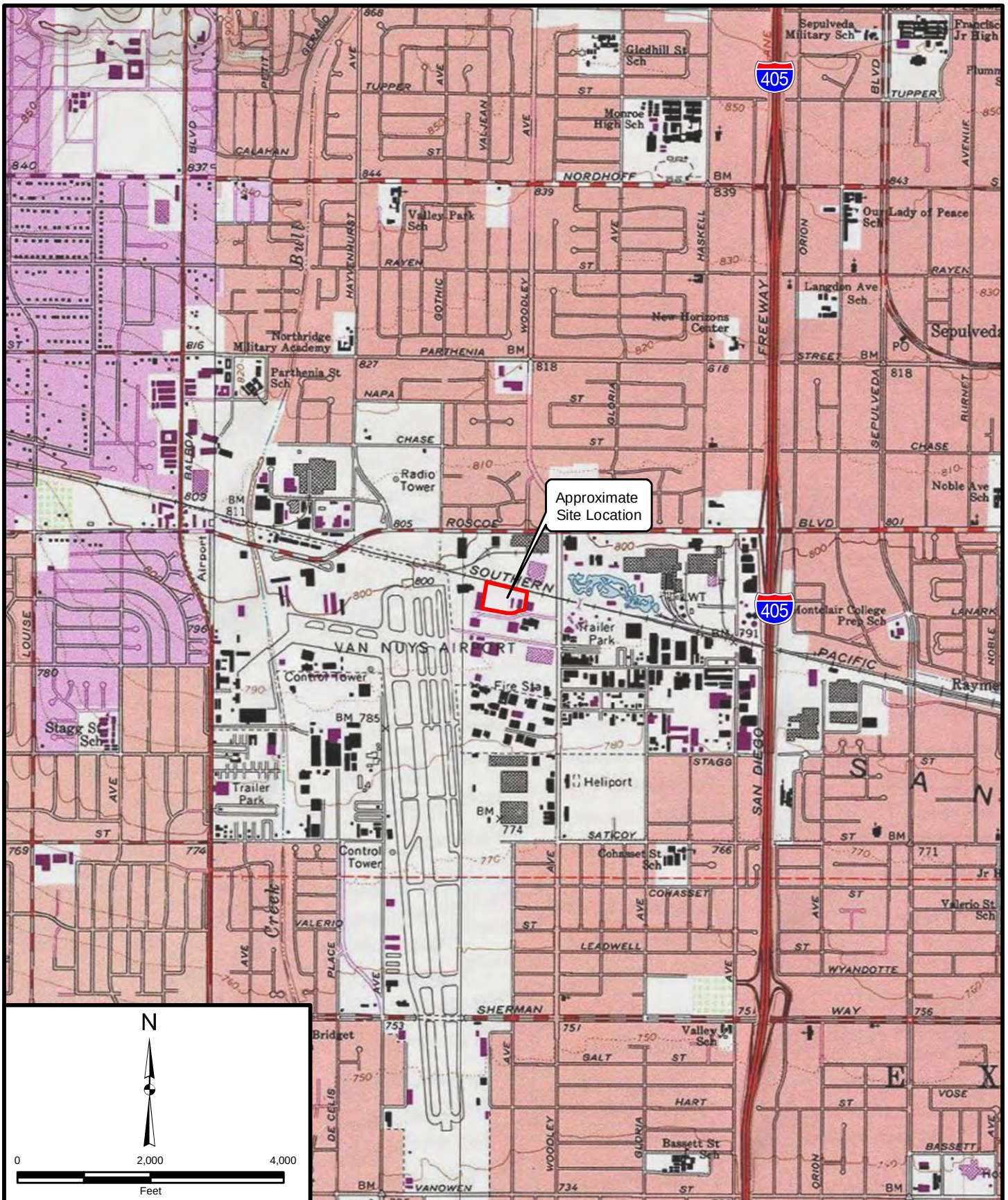


**GEOPROFESSIONAL
BUSINESS
ASSOCIATION**

Telephone: 301/565-2733

e-mail: info@geoprofessional.org www.geoprofessional.org

FIGURES



Project: 13918.001	Eng/Geol: JLH
Scale: 1" = 2,000'	Date: June 2023
Reference: Copyright:© 2013 National Geographic Society, i-cubed	

SITE LOCATION MAP

Proposed Industrial Building
16201 Raymer Street
Van Nuys District of Los Angeles, California

FIGURE 1





Project: 13918.001 Eng/Geol: JLH
Scale: 1" = 60' Date: June 2023
Reference: © 2023 Microsoft Corporation © 2023
Maxar ©CNES (2023) Distribution Airbus DS © 2023
TomTom, Rexford Industrial Realty, Conceptual Site
Plan

GEOTECHNICAL MAP

Proposed Industrial Building
16201 Raymer Street Van Nuys District of Los Angeles, California

FIGURE 2





Geologic Units

-  Af, Artificial Fill
-  Qyf, Young Alluvial Fan
-  Qof, Old Alluvial Fan Deposits

Project: 13918.001

Eng/Geol: JLH

Scale: 1" = 2,000'

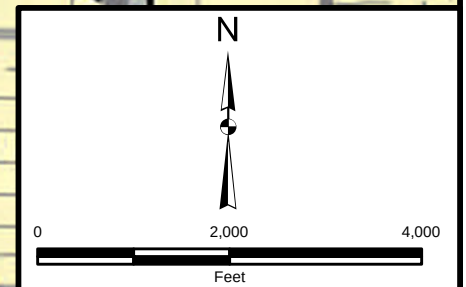
Date: June 2023

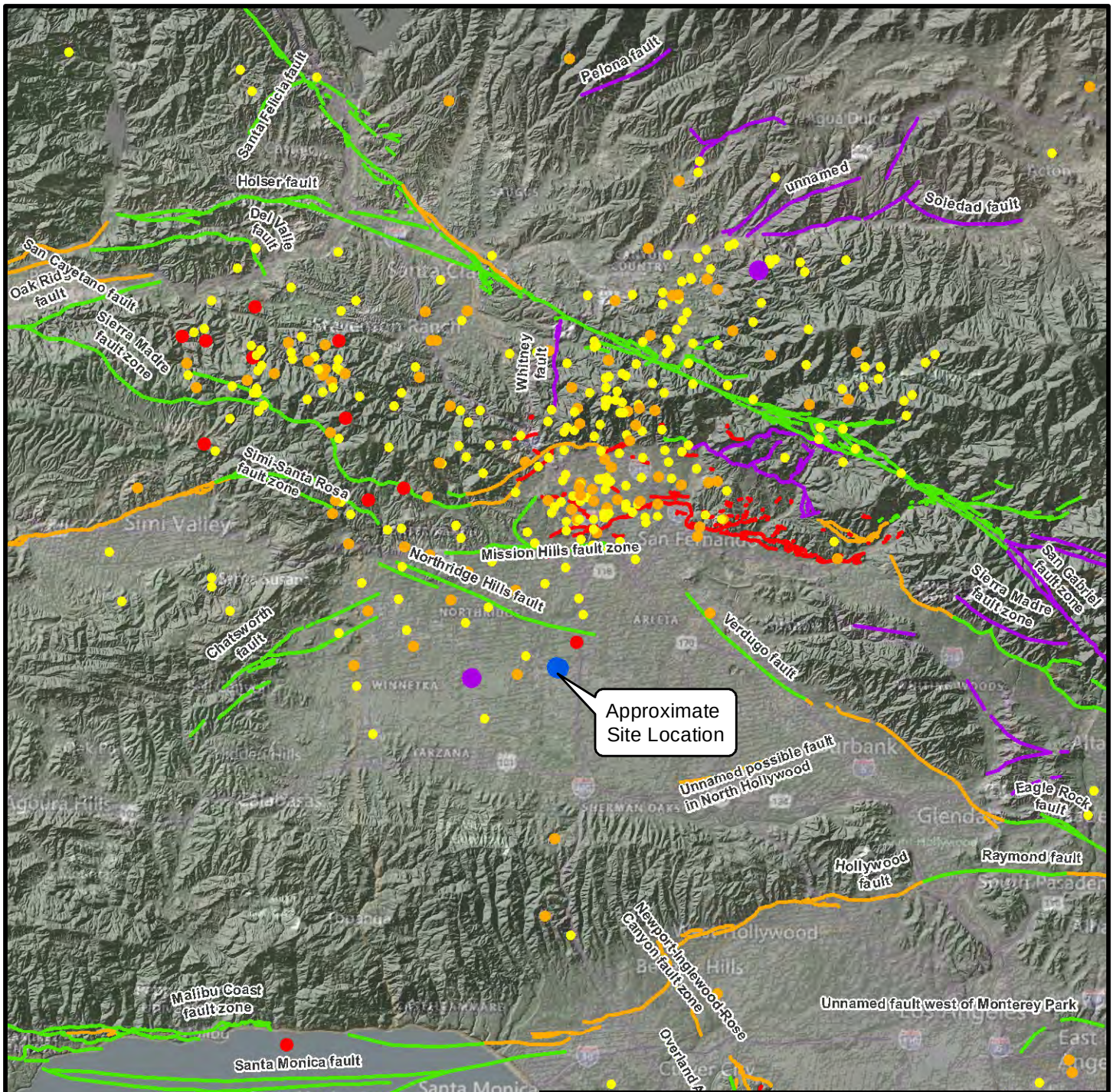
Reference: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30'x60' Quadrangle, Bedrossian and Roffers, 2012

REGIONAL GEOLOGY MAP

Proposed Industrial Building
16201 Raymer Street
Van Nuys District of Los Angeles, California

FIGURE 3





Approximate Site Location

Legend

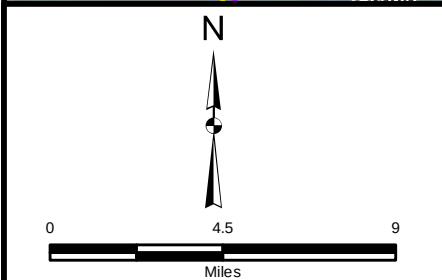
Fault activity

Recency of Movement

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

Historical Earthquakes ($\geq M3.5$)

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



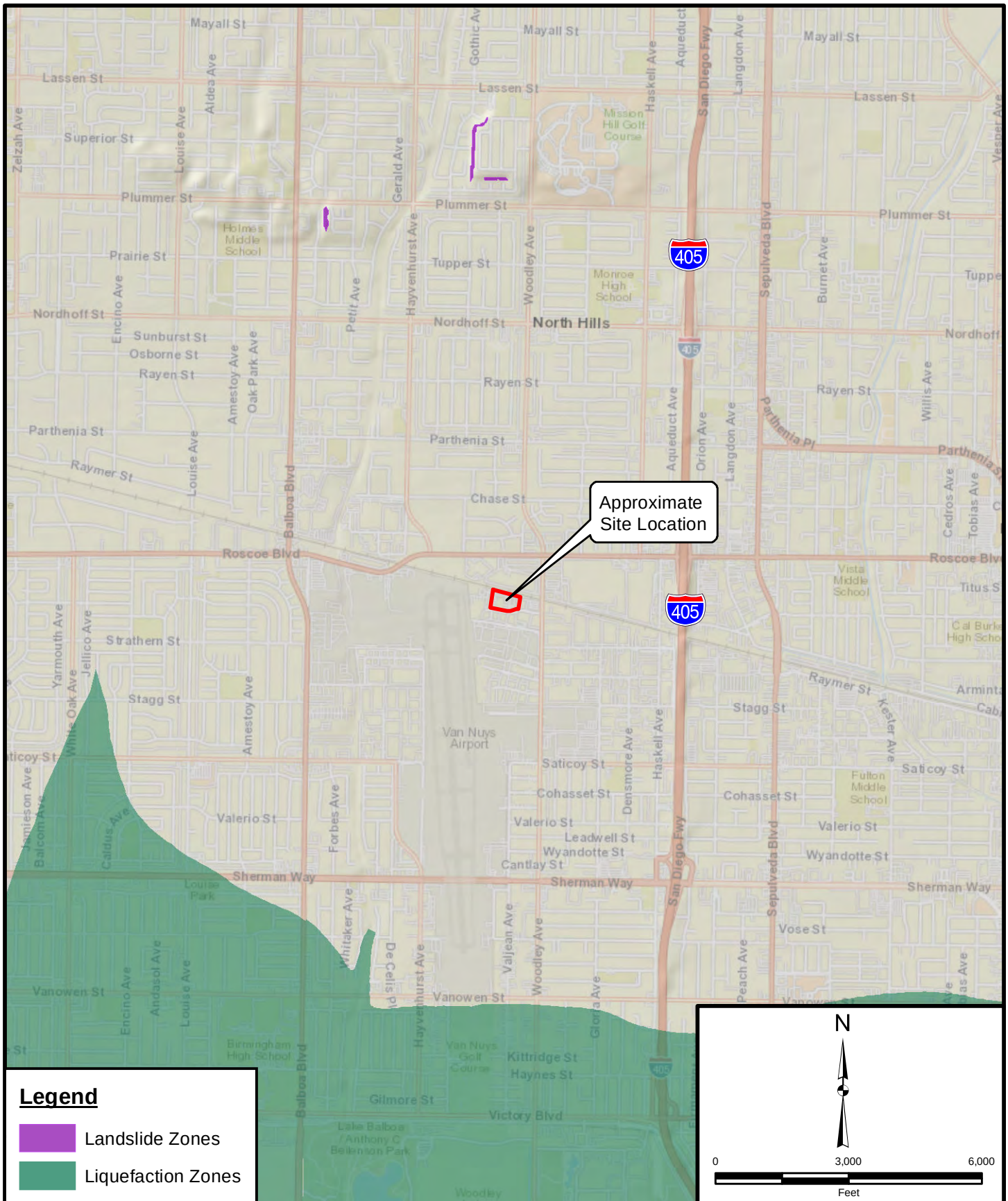
13918.001 Eng/Geol: JLH
 Scale: 1" = 5 miles Date: June 2023
 Basemap Reference: © 2023 Microsoft Corporation
 Earthstar Geographics SIO © 2023 TomTom
 Seismicity Data Reference: maps.conservation.ca.gov

REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Proposed Industrial Building
 16201 Raymer Street
 Van Nuys District of Los Angeles, California

FIGURE 4





Project: 13918.001	Eng/Geol: JLH	SEISMIC HAZARD MAP Proposed Industrial Building 16201 Raymer Street Van Nuys District of Los Angeles, California	FIGURE 5
Scale: 1 " = 3,000 '	Date: June 2023		Leighton
Service Layer Credits: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community Seismic Hazards Program, California Geological Survey, California			

Legend

100-Year Floodplains DSOD (2021)



500-Year Floodplains



Chatsworth

Encino

Lower San Fernando

Pacoima

OSM 2007

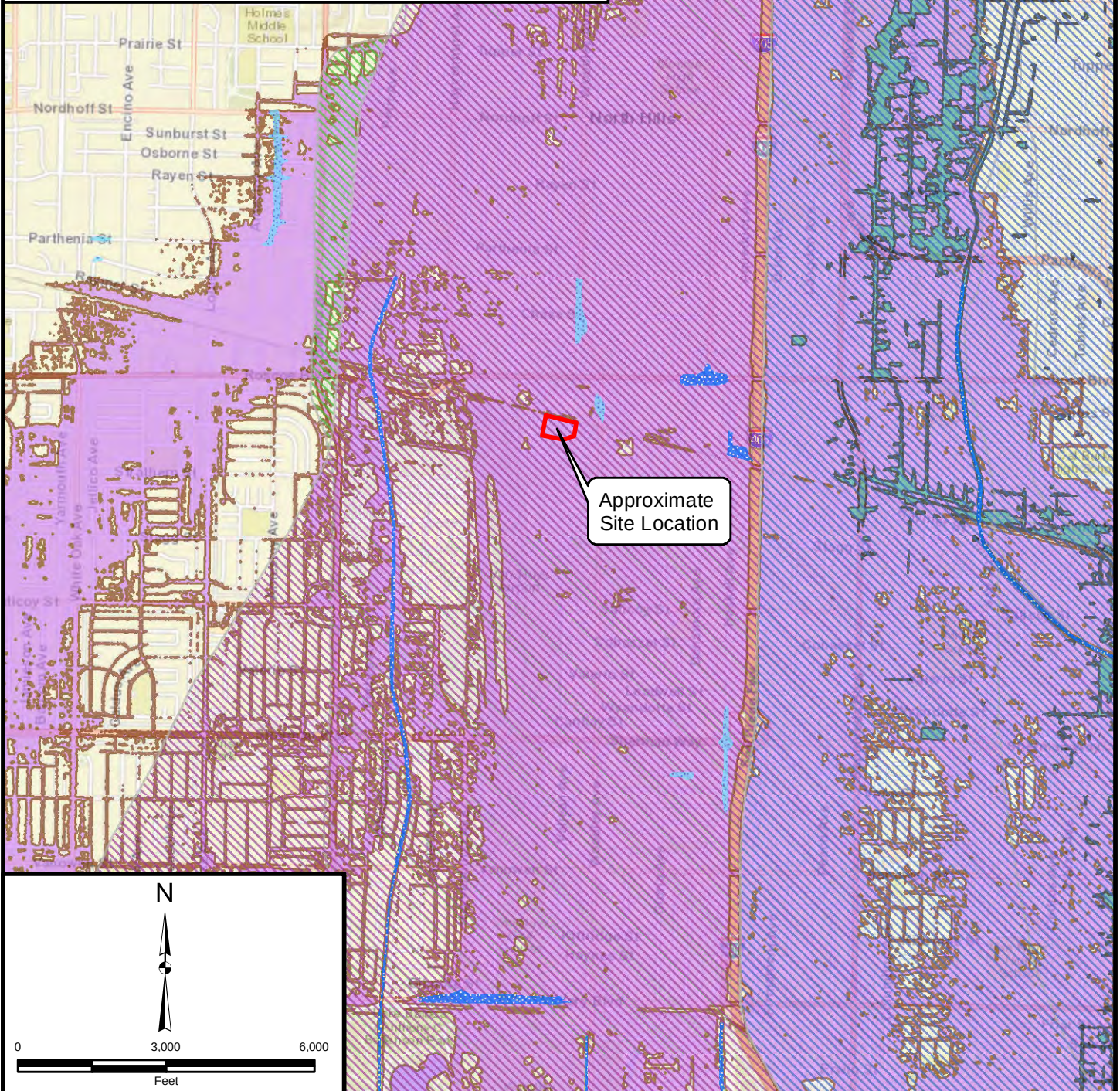
Encino

Hansen Dam

Los Angeles Res

Lower San Fernando

Pacoima



Approximate
Site Location

Project: 13918.001

Eng/Geol: JLH

Scale: 1" = 3,000'

Date: June 2023

Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
FEMA (<http://www.fema.gov/index.shtm>), DWR (<http://www.dwr.ca.gov>), Office of Emergency Services (2007), Dept of

FLOOD HAZARD ZONE MAP

Proposed Industrial Building
16201 Raymer Street
Van Nuys District of Los Angeles, California

FIGURE 6



APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-3

Project No. 13918.001
 Project LRF2 LA Van Nuys
 Drilling Co. Choice Drilling Inc.
 Drilling Method Hollow Stem Auger - 40lb - Autohammer - 30" Drop
 Location See Figure 2 - Geotechnical Map

Date Drilled 6-9-23
 Logged By LFO
 Hole Diameter 8"
 Ground Elevation 793'
 Sampled By LFO

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	0	N S							@Surface: 3" asphalt over 3" base	
				B-1				CL SM	Undocumented Artificial Fill (Afu)	
				R-1	9 10 11	108.5	17.8	CL	@6": Sandy CLAY, dark olive brown, predominantly fine to medium sand, trace coarse sand, moist @1': Silty SAND, reddish brown, fine sand, moist	
790				R-2	4 5 6	110.8	15.6	CL-ML	@2': Sandy CLAY, dark olive brown, very stiff, predominantly fine to medium sand, trace coarse sand, moist, laminated	CN, DS
	5			R-3	7 10 10	102.4	18.0	SM-CL	Quaternary Young Alluvial Fan Deposits (Qyf) @4': Silty CLAY, olive brown, stiff, some fine to medium sand, trace coarse sand, moist, laminated	
785				R-4	6 10 12	86.5	31.3	CL	@6': Silty SAND w/ Clay, yellow brown, medium dense, predominantly fine sand, some medium sand, low plasticity, moist, thinly laminated	CN, -200
	10			R-5	7 10 11	102.1	15.6	SM-SC	@8': Lean CLAY, light olive brown, very stiff, some fine sand, some silt, medium plasticity, moist	
780									@10': Silty Clayey SAND, yellow brown, medium dense, predominantly fine sand, trace medium sand, low plasticity, moist	
	15			S-6	4 5 6		11.2	SM-CL SP-SM	@15': Silty SAND w/ Clay, yellow brown, medium dense, predominantly fine sand, some medium sand, low plasticity, moist, thinly laminated	
775									@16': Poorly-graded SAND w/ Silt, orange to yellow brown, medium dense, predominantly fine sand, some medium sand, trace subrounded gravel, moist	
	20			S-7	9 9 13			SP-GP	@20': Poorly-graded SAND w/ Gravel, orange to tannish brown, dense, predominantly medium sand, some fine and coarse sand, predominantly fine to medium subrounded gravel, moist	
770									Total Depth: 21.5' bgs No groundwater encountered during drilling Backfilled with bentonite grout and surface patched with cold-mix asphalt concrete	
	25									
765										
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

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



GEOTECHNICAL BORING LOG LB-4

Project No. 13918.001
 Project LRF2 LA Van Nuys
 Drilling Co. Choice Drilling Inc.
 Drilling Method Hollow Stem Auger - 40lb - Autohammer - 30" Drop
 Location See Figure 2 - Geotechnical Map

Date Drilled 6-9-23
 Logged By LFO
 Hole Diameter 8"
 Ground Elevation 796'
 Sampled By LFO

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
795	0	N S		B-1				CL	@Surface: 3" asphalt over 3" base Undocumented Artificial Fill (Afu) @6": Sandy lean CLAY, dark brown, low plasticity, moist	CR, EI, MD, RV
								SM	@2': Silty SAND, orange brown, fine sand, moist	
									Quaternary Young Alluvial Fan Deposits (Qyf)	
790	5			R-1	9 9 10	111.7	15.6	CL	@5': Lean CLAY, dark brown, low plasticity, moist	
								SM	@6': Silty SAND, yellow brown, medium dense, fine sand, laminated	
785	10			R-2	6 7 8	104.3	13.5	SM-SC	@10': Silty Clayey SAND, yellow brown, medium dense, predominantly fine and, few medium sand, moist, some CaCO3 wisps, massive	
780	15			S-3	3 5 4		8.9	SP SC	@15': Silty Clayey SAND, yellow brown, medium dense, predominantly fine and, few medium sand, moist, some CaCO3 wisps, massive @16': Poorly-graded SAND, yellow brown, fine to medium sand, moist @16.5': Clayey SAND, yellow brown, medium dense, fine sand, some silt, moist	
775	20			S-4	4 4 6		9.5	SM	@20': Silty SAND, yellow brown, medium dense, fine sand, trace clay, moist	
770	25			S-5	5 5 6				@25': Silty SAND, yellow brown, medium dense, fine sand, trace clay, trace gravel, moist	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

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



GEOTECHNICAL BORING LOG LB-4

Project No. 13918.001
 Project LRF2 LA Van Nuys
 Drilling Co. Choice Drilling Inc.
 Drilling Method Hollow Stem Auger - 40lb - Autohammer - 30" Drop
 Location See Figure 2 - Geotechnical Map

Date Drilled 6-9-23
 Logged By LFO
 Hole Diameter 8"
 Ground Elevation 796'
 Sampled By LFO

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	
765	30			S-6	4 7 9		24.7	CL	@30': Sandy lean CLAY, olive brown, very stiff, fine sand, low plasticity, moist, massive	
760	35			S-7	5 6 8		16.2	SC	@35': Clayey SAND, olive brown, medium dense, predominantly fine sand, trace medium sand, some silt, low plasticity, moist, trace FeO staining	
755	40			S-8	5 6 9				@40': Clayey SAND, olive brown, medium dense, predominantly fine sand, trace medium sand, some silt, low plasticity, moist, trace FeO staining, interbedded with 2-3" thick lenses of yellowish brown poorly-graded fine SAND w/ Silt	
750	45			S-9	4 7 9				@45': Clayey SAND, olive brown, medium dense, predominantly fine sand, trace medium sand, some silt, low plasticity, moist, trace FeO staining	
745	50			S-10	7 7 7			SM	@50': Silty SAND, reddish brown, medium dense, predominantly fine sand, trace medium sand, trace clay, moist	
									Total Depth: 51.5' bgs No groundwater encountered during drilling Backfilled with bentonite grout and surface patched with cold-mix asphalt concrete	
	55									
740										
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

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



GEOTECHNICAL BORING LOG LP-2

Project No. 13918.001
 Project LRF2 LA Van Nuys
 Drilling Co. Choice Drilling Inc.
 Drilling Method Hollow Stem Auger - 40lb - Autohammer - 30" Drop
 Location See Figure 2 - Geotechnical Map

Date Drilled 6-9-23
 Logged By LFO
 Hole Diameter 8"
 Ground Elevation 793'
 Sampled By LFO

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	
								CL-SP	@Surface: 3" asphalt over 3" base Undocumented Artificial Fill (Afu) @6": Lean CLAY w/ Sand, dark brown, fine sand, low plasticity, slightly moist	
790									Quaternary Young Alluvial Fan Deposits (Qyf)	
	5							CL	@5': Sandy lean CLAY, olive brown, fine sand, low plasticity, moist	
785				B-1				CL	@8': Lean CLAY, yellowish brown, trace fine sand, medium plasticity, moist	-200
	10								Total Depth: 10' bgs No groundwater encountered during drilling. Converted into a percolation well with 5' of solid and 5' of 0.020" slotted PVC pipe. Boring filled up to top of screened zone with Monterey No. 3 sand. Pipe removed following percolation testing, backfilled with soil cuttings, and surface patched with cold-mix asphalt.	
780										
	15									
775										
	20									
770										
	25									
765										
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

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



APPENDIX B

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	13918.001	Test Hole Number:	LP-2
Project Name:	LRF2 LA Van Nuys	Date Excavated:	6/12/2023
Earth Description:	Alluvium	Date Tested:	6/13/2023
Liquid Description:	Tap water	Depth of boring (ft):	9.5
Tested By:	LFO/RM	Radius of boring (in):	4
<u>Time Interval Standard</u>		Radius of casing (in):	1
Start Time for Pre-Soak:	10:34 AM	Length of slotted of casing (ft):	5
Start Time for Standard:	10:34 AM	Depth to Initial Water Depth (ft):	10
Standard Time Interval	30 mins	Porosity of Annulus Material, n :	0.35
Between Readings, mins:	30	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	10:34	30	8.00	18.0	5.0	0.45
	11:04		8.42	13.0		
1	11:04	30	8.00	18.0	4.7	0.41
	11:34		8.39	13.3		
2	11:35	30	8.00	18.0	3.7	0.32
	12:05		8.31	14.3		
3	12:06	30	8.00	18.0	2.2	0.18
	12:36		8.18	15.8		
4	12:36	30	8.00	18.0	2.0	0.17
	13:06		8.17	16.0		
5	13:07	30	8.00	18.0	1.8	0.15
	13:37		8.15	16.2		
6	13:38	30	8.00	18.0	1.8	0.15
	14:08		8.15	16.2		

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

Measured Infiltration Rate, I (Average of Last 3 Readings) = 0.15 in./hr.

APPENDIX C

LABORATORY TEST RESULTS

Boring No.	LB-3	LP-2						
Sample No.	R-4	BB-1						
Depth (ft.)	8.0	8-10						
Sample Type	Ring	Bulk						
Soil Identification	Light olive brown lean clay (CL)	Yellowish brown lean clay (CL)						
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00						
Dry Weight of Soil + Container (g)	0.00	0.00						
Weight of Container (g)	1.00	1.00						
Moisture Content (%)	0.00	0.00						
Sample Dry Weight Determination								
Weight of Sample + Container (g)	437.36	805.16						
Weight of Container (g)	220.18	108.52						
Weight of Dry Sample (g)	217.18	696.64						
Container No.:								
After Wash								
Method (A or B)	B	B						
Dry Weight of Sample + Cont. (g)	246.24	200.96						
Weight of Container (g)	220.18	108.52						
Dry Weight of Sample (g)	26.06	92.44						
% Passing No. 200 Sieve	88.0	86.7						
% Retained No. 200 Sieve	12.0	13.3						
			PERCENT PASSING No. 200 SIEVE ASTM D 1140			Project Name: <u>LRF2 Van Nuys</u>		
						Project No.: <u>13918.001</u>		
						Tested By: <u>JD/GB</u> Date: <u>06/20/23</u>		

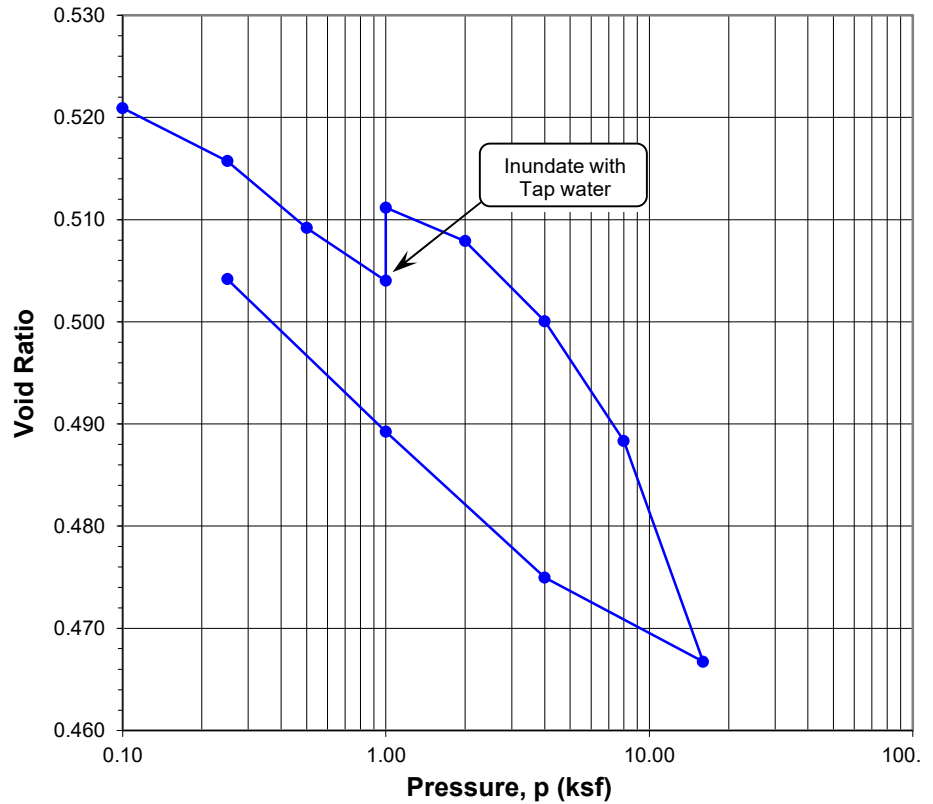


ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435

Project Name: LRF2 Van Nuys
Project No.: 13918.001
Boring No.: LB-3
Sample No.: R-2
Soil Identification: Olive brown silty clay (CL-ML)

Tested By: GB/JD Date: 06/21/23
Checked By: J. Ward Date: 07/20/23
Depth (ft.): 4.0
Sample Type: Ring

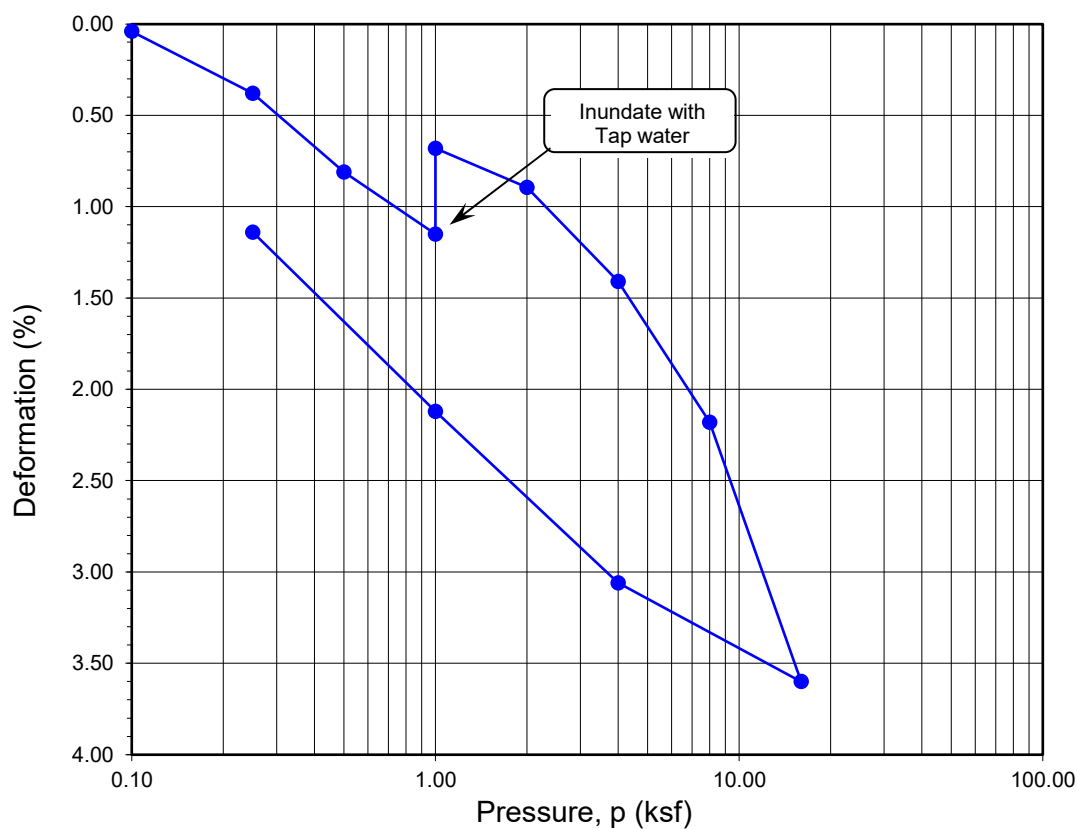
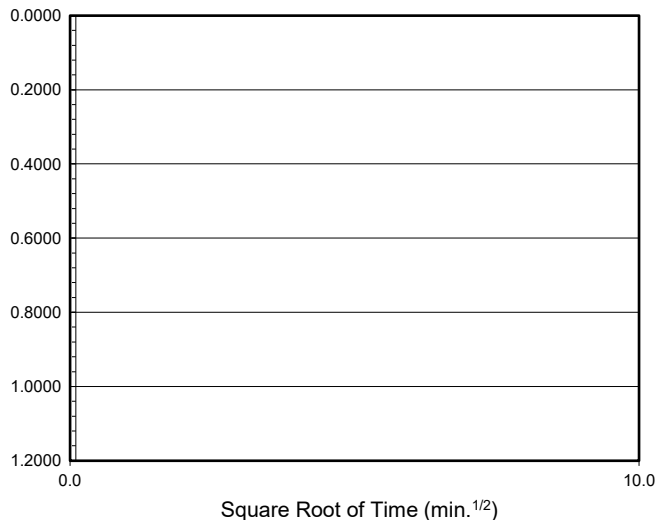
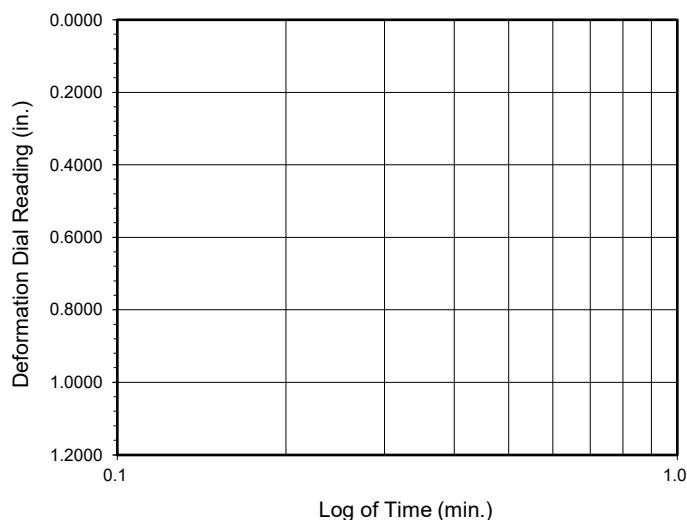
Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	199.56
Weight of Ring (g):	45.59
Height after consol. (in.):	0.9886
Before Test	
Wt. of Wet Sample+Cont. (g):	149.80
Wt. of Dry Sample+Cont. (g):	138.42
Weight of Container (g):	65.40
Initial Moisture Content (%)	15.6
Initial Dry Density (pcf)	110.8
Initial Saturation (%):	81
Initial Vertical Reading (in.)	0.0716
After Test	
Wt. of Wet Sample+Cont. (g):	257.19
Wt. of Dry Sample+Cont. (g):	232.91
Weight of Container (g):	55.47
Final Moisture Content (%)	18.41
Final Dry Density (pcf):	110.9
Final Saturation (%):	96
Final Vertical Reading (in.)	0.0864
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	0.0720	0.9996	0.00	0.04	0.521	0.04
0.25	0.0758	0.9958	0.04	0.42	0.516	0.38
0.50	0.0806	0.9910	0.09	0.90	0.509	0.81
1.00	0.0847	0.9869	0.16	1.31	0.504	1.15
1.00	0.0800	0.9916	0.16	0.84	0.511	0.68
2.00	0.0830	0.9887	0.24	1.14	0.508	0.89
4.00	0.0890	0.9826	0.33	1.74	0.500	1.41
8.00	0.0979	0.9737	0.45	2.63	0.488	2.18
16.00	0.1133	0.9583	0.57	4.17	0.467	3.60
4.00	0.1069	0.9647	0.47	3.53	0.475	3.06
1.00	0.0968	0.9748	0.40	2.52	0.489	2.12
0.25	0.0864	0.9852	0.34	1.48	0.504	1.14

Time Readings				
Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	R-2	4	15.6	18.4	110.8	110.9	0.522	0.504	81	96

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



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

Project No.: 13918.001

LRF2 Van Nuys

07-23

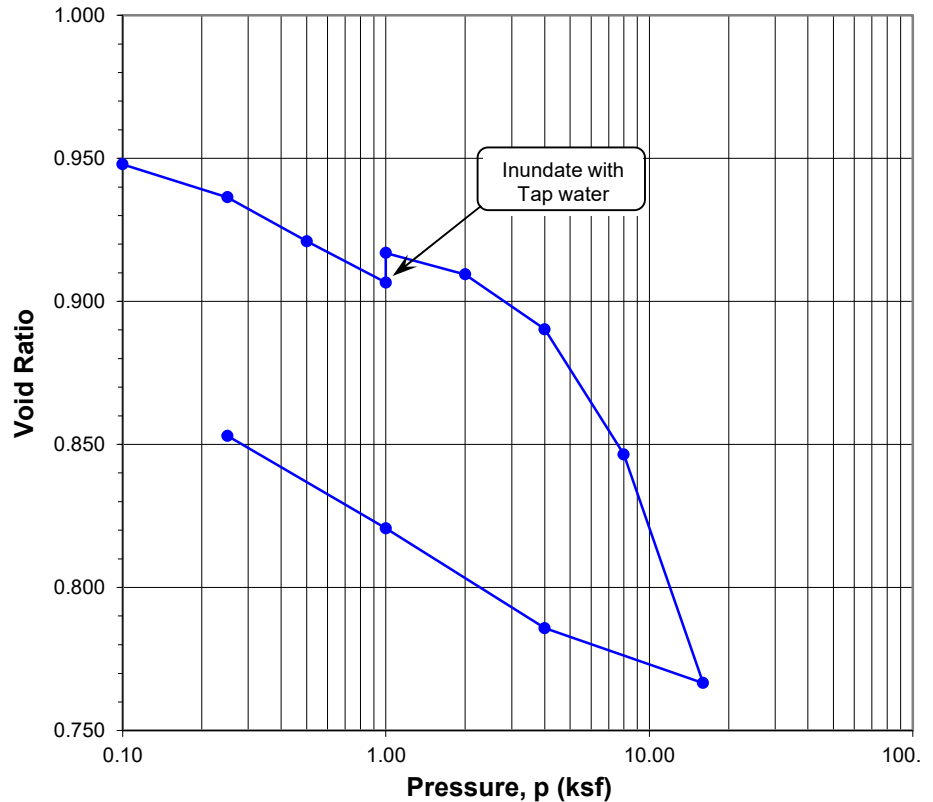


ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435

Project Name: LRF2 Van Nuys
Project No.: 13918.001
Boring No.: LB-3
Sample No.: R-4
Soil Identification: Light olive brown lean clay (CL)

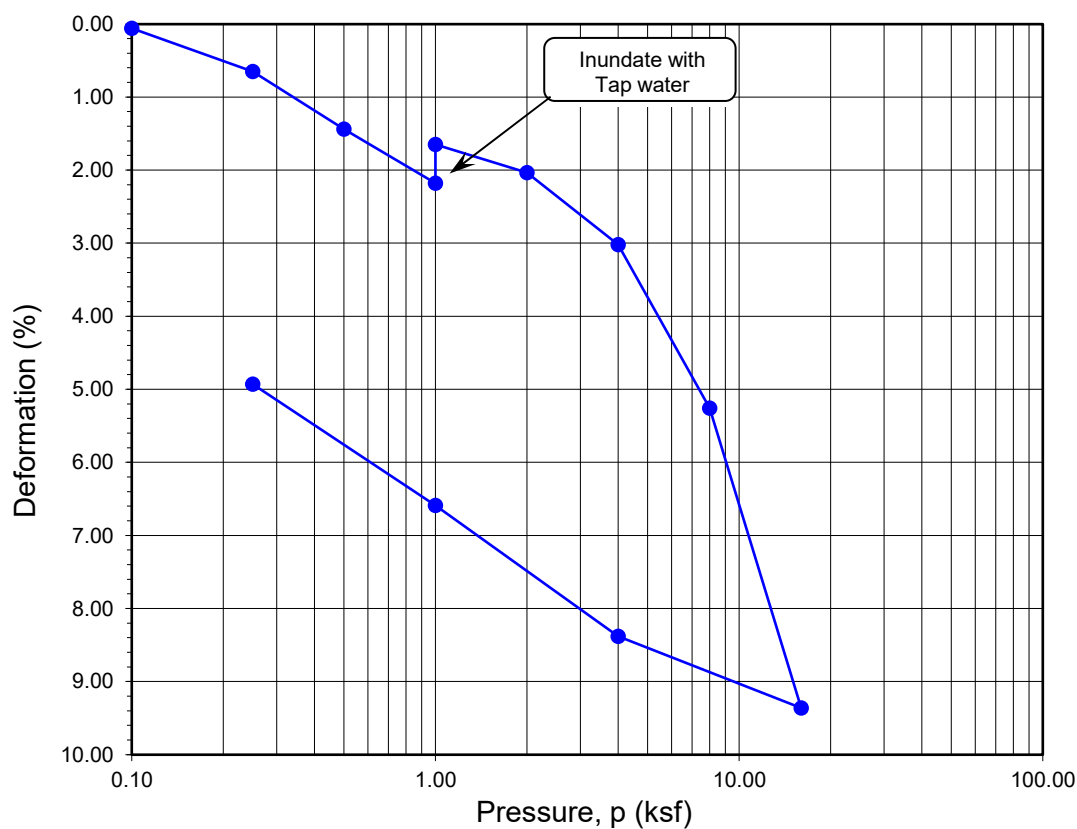
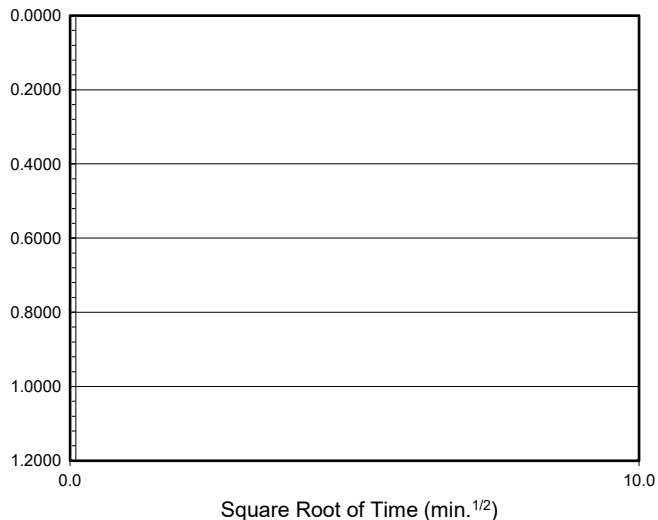
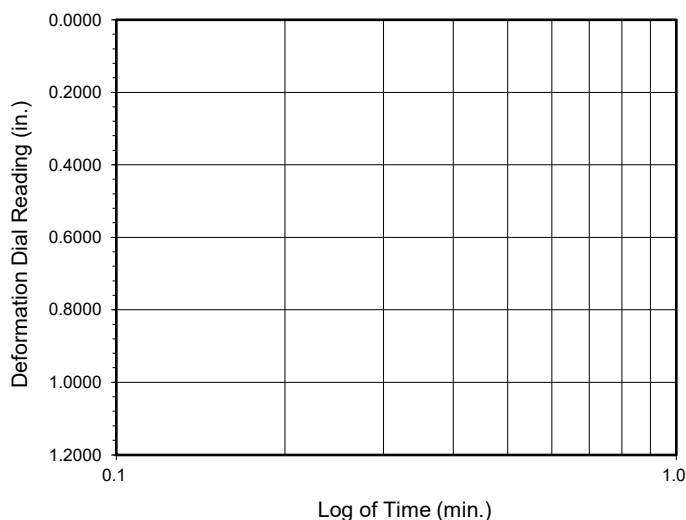
Tested By: GB/JD Date: 06/21/23
Checked By: J. Ward Date: 07/20/23
Depth (ft.): 8.0
Sample Type: Ring

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	181.31
Weight of Ring (g):	44.74
Height after consol. (in.):	0.9507
Before Test	
Wt. of Wet Sample+Cont. (g):	505.41
Wt. of Dry Sample+Cont. (g):	437.36
Weight of Container (g):	220.18
Initial Moisture Content (%)	31.3
Initial Dry Density (pcf)	86.5
Initial Saturation (%):	89
Initial Vertical Reading (in.)	0.0387
After Test	
Wt. of Wet Sample+Cont. (g):	232.80
Wt. of Dry Sample+Cont. (g):	199.19
Weight of Container (g):	52.57
Final Moisture Content (%)	32.99
Final Dry Density (pcf):	89.1
Final Saturation (%):	100
Final Vertical Reading (in.)	0.0908
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 (%)	Time Readings				
							Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
0.10	0.0393	0.9994	0.00	0.06	0.948	0.06					
0.25	0.0455	0.9932	0.03	0.68	0.936	0.65					
0.50	0.0538	0.9849	0.07	1.51	0.921	1.44					
1.00	0.0619	0.9768	0.14	2.32	0.907	2.18					
1.00	0.0566	0.9821	0.14	1.79	0.917	1.65					
2.00	0.0613	0.9775	0.22	2.26	0.909	2.04					
4.00	0.0720	0.9667	0.31	3.33	0.890	3.02					
8.00	0.0953	0.9434	0.40	5.66	0.847	5.26					
16.00	0.1374	0.9013	0.51	9.87	0.767	9.36					
4.00	0.1269	0.9118	0.44	8.82	0.786	8.38					
1.00	0.1081	0.9306	0.35	6.94	0.821	6.59					
0.25	0.0908	0.9479	0.28	5.21	0.853	4.93					

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	R-4	8	31.3	33.0	86.5	89.1	0.949	0.853	89	100

Soil Identification: Light olive brown lean clay (CL)

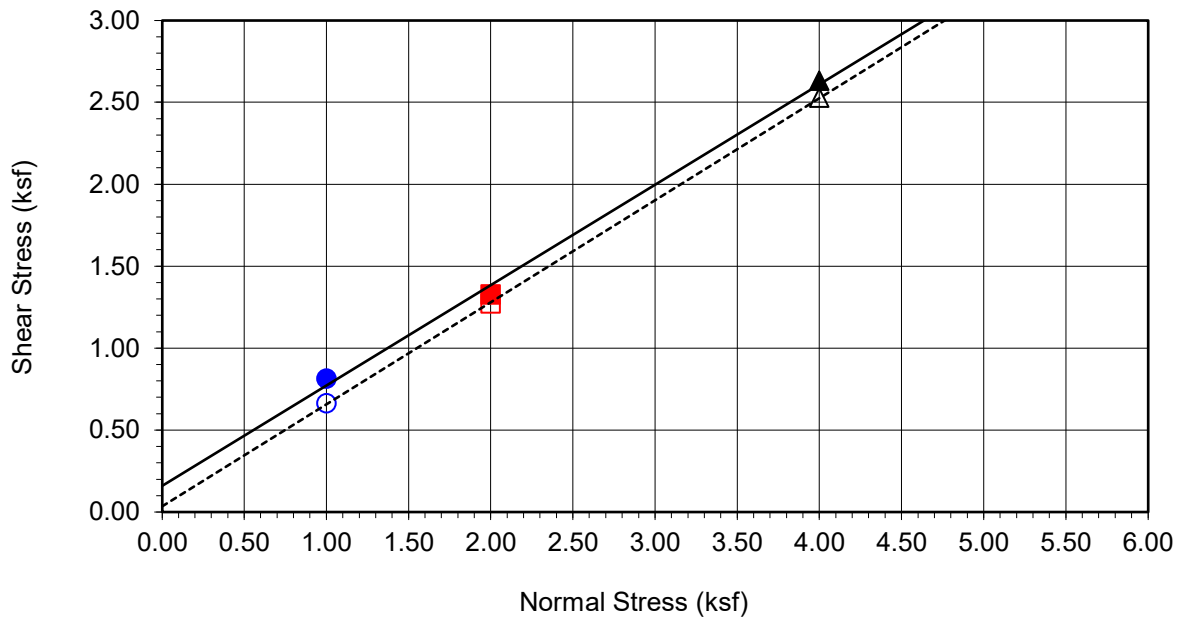
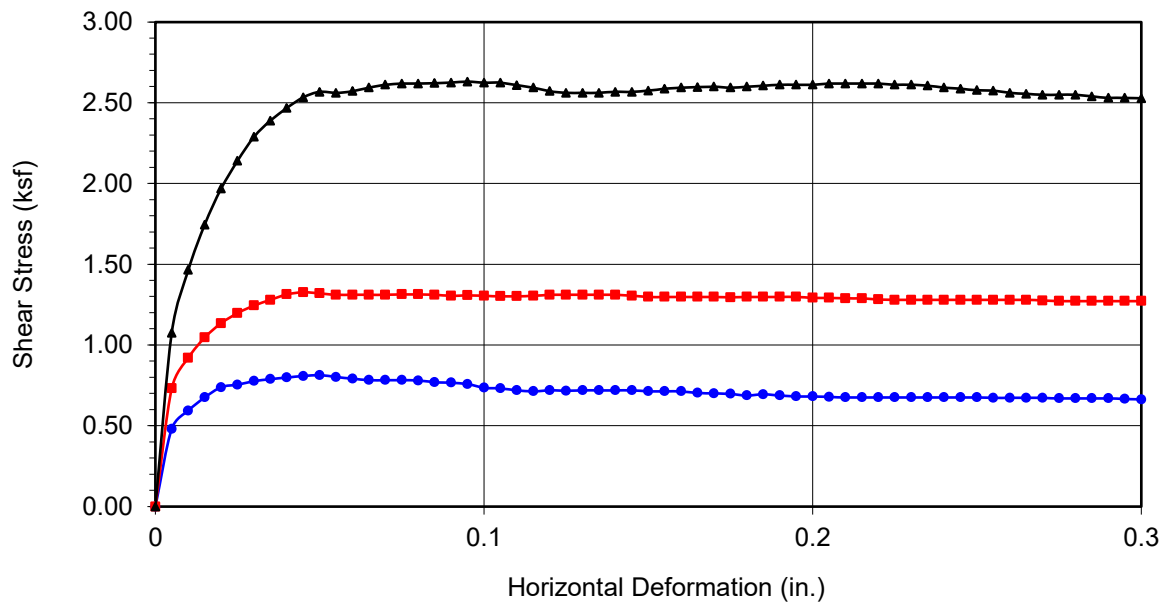


ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project No.: 13918.001

LRF2 Van Nuys

07-23



Boring No.	LB-3	
Sample No.	R-2	
Depth (ft)	4	
<u>Sample Type:</u>		Ring
<u>Soil Identification:</u> Olive brown silty clay (CL-ML)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	162	31
Ultimate	35	32

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.814	■ 1.327	▲ 2.631
Shear Stress @ End of Test (ksf)	○ 0.663	□ 1.273	△ 2.528
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	15.58	15.58	15.58
Dry Density (pcf)	104.7	102.6	101.7
Saturation (%)	69.0	65.4	64.0
Soil Height Before Shearing (in.)	0.9931	0.9828	0.9787
Final Moisture Content (%)	22.0	21.2	19.9



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13918.001

LRF2 Van Nuys

06-23



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: LRF2 Van Nuys Tested By: G. Berdy Date: 06/23/23
Project No.: 13918.001 Checked By: J. Ward Date: 07/24/23
Boring No.: LB-4 Depth (ft.): 0-5
Sample No.: BB-1
Soil Identification: Dark brown sandy lean clay s(CL)

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	1.0400
Wt. Comp. Soil + Mold (g)	572.80	430.35
Wt. of Mold (g)	180.50	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	782.00	610.85
Dry Wt. of Soil + Cont. (g)	705.80	534.58
Wt. of Container (g)	0.00	180.50
Moisture Content (%)	10.80	21.54
Wet Density (pcf)	118.3	124.8
Dry Density (pcf)	106.8	102.7
Void Ratio	0.578	0.642
Total Porosity	0.366	0.391
Pore Volume (cc)	75.9	84.1
Degree of Saturation (%) [S _{meas}]	50.4	90.7

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.)
06/23/23	10:56	1.0	0	0.4620
06/23/23	11:06	1.0	10	0.4605
Add Distilled Water to the Specimen				
06/23/23	12:00	1.0	54	0.4900
06/24/23	9:57	1.0	1371	0.5000
06/26/23	6:08	1.0	4022	0.5020

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	42
---	-----------



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: LRF2 Van Nuys
Project No. : 13918.001
Boring No.: LB-4
Sample No. : BB-1

Tested By : J. Domingo Date: 06/27/23
Checked By: J. Ward Date: 07/24/23
Depth (ft.) : 0-5

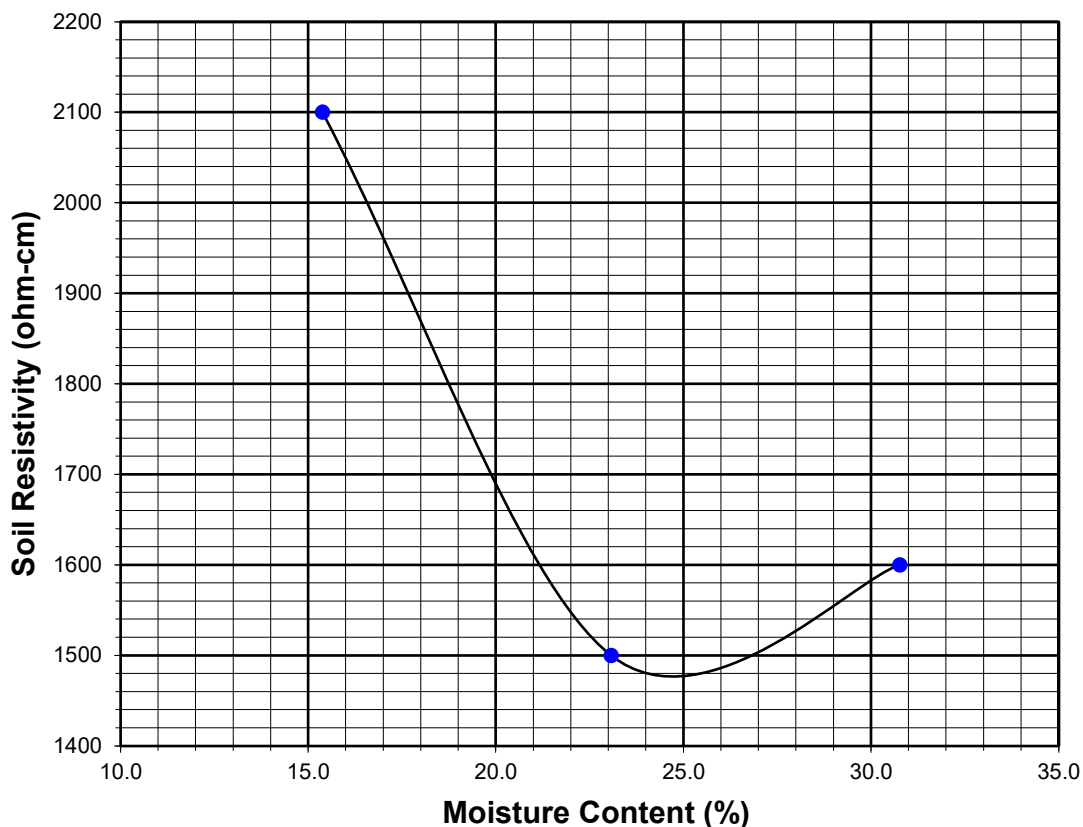
Soil Identification:* Dark brown s(CL)

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.38	2100	2100
2	30	23.08	1500	1500
3	40	30.77	1600	1600
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)	130.01
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1480	24.7	103	80	7.35	23.3





MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: LRF2 Van Nuys Tested By: O. Figueroa Date: 07/03/23
Project No.: 13918.001 Checked By: J. Ward Date: 07/24/23
Boring No.: LB-4 Depth (ft.): 0-5
Sample No.: BB-1
Soil Identification: Dark brown sandy lean clay s(CL)

Preparation Method:

☒ Moist
☐ Dry

☒ Mechanical Ram
☐ Manual Ram

Mold Volume (ft³)

0.03320

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3709	3795	3844	3802		
Weight of Mold (g)	1808	1808	1808	1808		
Net Weight of Soil (g)	1901	1987	2036	1994		
Wet Weight of Soil + Cont. (g)	738.7	746.0	792.4	775.8		
Dry Weight of Soil + Cont. (g)	689.6	682.2	711.0	681.6		
Weight of Container (g)	78.8	78.5	88.9	76.1		
Moisture Content (%)	8.04	10.57	13.08	15.56		
Wet Density (pcf)	126.2	131.9	135.2	132.4		
Dry Density (pcf)	116.8	119.3	119.6	114.6		

Maximum Dry Density (pcf)

119.9

Optimum Moisture Content (%)

12.0

PROCEDURE USED

☒ Procedure A

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

☐ Procedure B

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

☐ Procedure C

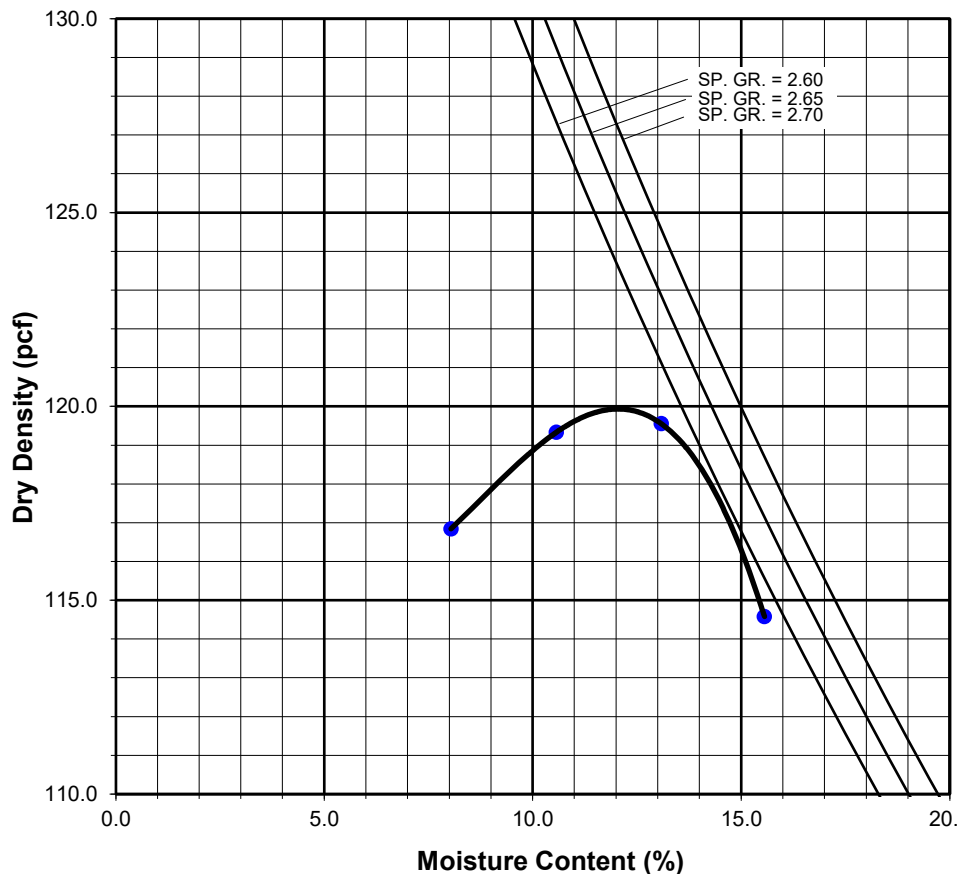
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL, PL, PI





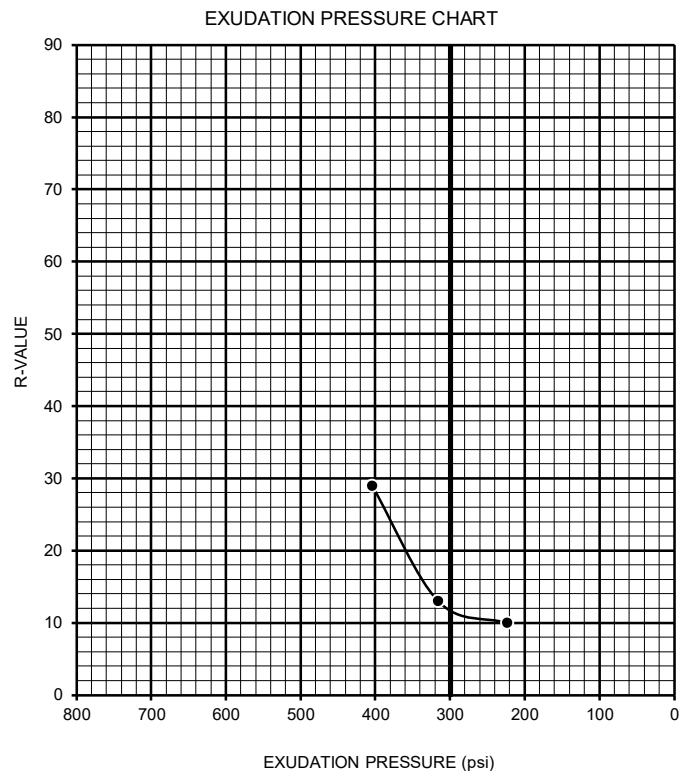
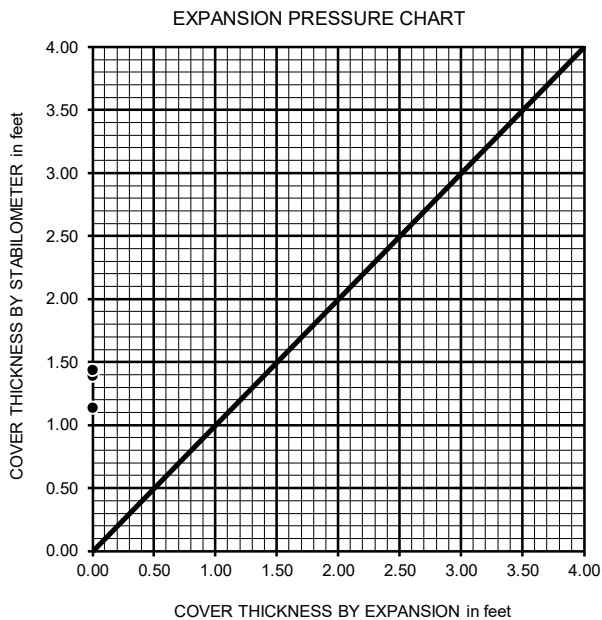
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME: LRF2 Van Nuys PROJECT NUMBER: 13918.001
BORING NUMBER: LB-4 DEPTH (FT.): 0-5
SAMPLE NUMBER: BB-1 TECHNICIAN: O. Figueroa
SAMPLE DESCRIPTION: Dark brown sandy lean clay s(CL) DATE COMPLETED: 7/6/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	16.4	17.5	18.8
HEIGHT OF SAMPLE, Inches	2.51	2.50	2.53
DRY DENSITY, pcf	114.5	112.6	110.5
COMPACTOR PRESSURE, psi	120	90	70
EXUDATION PRESSURE, psi	404	316	224
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	99	128	133
TURNS DISPLACEMENT	3.85	4.15	4.35
R-VALUE UNCORRECTED	29	13	10
R-VALUE CORRECTED	29	13	10

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.14	1.39	1.44
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00



R-VALUE BY EXPANSION: N/A
R-VALUE BY EXUDATION: 12
EQUILIBRIUM R-VALUE: 12

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D

LEIGHTON CONSULTING, INC. EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

<u>Section</u>	<u>Appendix D Page</u>
D-1.0 GENERAL.....	1
D-1.1 Intent	1
D-1.2 Role of Leighton Consulting, Inc.....	1
D-1.3 The Earthwork Contractor	1
D-2.0 PREPARATION OF AREAS TO BE FILLED	2
D-2.1 Clearing and Grubbing	2
D-2.2 Processing.....	3
D-2.3 Overexcavation	3
D-2.4 Benching	3
D-2.5 Evaluation/Acceptance of Fill Areas	3
D-3.0 FILL MATERIAL	4
D-3.1 Fill Quality.....	4
D-3.2 Oversize	4
D-3.3 Import	4
D-4.0 FILL PLACEMENT AND COMPACTION	4
D-4.1 Fill Layers	4
D-4.2 Fill Moisture Conditioning	5
D-4.3 Compaction of Fill.....	5
D-4.4 Compaction of Fill Slopes.....	5
D-4.5 Compaction Testing	5
D-4.6 Compaction Test Locations.....	5
D-5.0 EXCAVATION.....	6
D-6.0 TRENCH BACKFILLS	6
D-6.1 Safety	6
D-6.2 Bedding and Backfill	6
D-6.3 Lift Thickness	7

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

D-1.1 Intent

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

D-1.2 Role of Leighton Consulting, Inc.

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

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

D-1.3 The Earthwork Contractor

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

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

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

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

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

D-2.1 Clearing and Grubbing

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

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

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage

of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

D-2.2 Processing

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

D-2.3 Overexcavation

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

D-2.4 Benching

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

D-2.5 Evaluation/Acceptance of Fill Areas

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

D - 3 . 0 F I L L M A T E R I A L

D-3.1 Fill Quality

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

D-3.2 Oversize

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

D-3.3 Import

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

D - 4 . 0 F I L L P L A C E M E N T A N D C O M P A C T I O N

D-4.1 Fill Layers

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

D-4.2 Fill Moisture Conditioning

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

D-4.3 Compaction of Fill

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

D-4.4 Compaction of Fill Slopes

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

D-4.5 Compaction Testing

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

D-4.6 Compaction Test Locations

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

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

D - 5 . 0 E X C A V A T I O N

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

D - 6 . 0 T R E N C H B A C K F I L L S

D-6.1 **Safety**

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

D-6.2 **Bedding and Backfill**

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

D-6.3 Lift Thickness

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