Appendix E

Geotechnical Report

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

February 25, 2021

LOG # 115712-01
SOILS/GEOLOGY FILE - 2

Bridge 1355 Sepulveda, LLC
11100 Santa Monica Boulevard, Suite 700
Los Angeles, CA 90025

TRACT:	PM 2004-5121 / 750 Acre Tract of Maria Machado De Rocha / 65665
LOT(S):	D / PT LT2 / 1
LOCATION:	1355 W. Sepulveda Boulevard

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Addendum Report	12809.001	01/29/2021	Leighton Consulting, Inc.

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	115712	01/12/2021	LADBS
Geology/Soils Report	12809.001	09/23/2020	Leighton Consulting, Inc.
Addendum Report	12809.002	11/20/2020	"

The Grading Division of the Department of Building and Safety has reviewed the referenced reports that provide recommendations for a proposed warehouse structure. According to the reports, the site consists of several contiguous parcels that are relatively flat. Facilities related to an existing amusement park, as well as other structures on the site, are to be removed.

The earth materials at the subsurface exploration locations consist of up to 7 feet of uncertified fill underlain by alluvium. The consultants recommend to remove and replace a minimum of 5 feet of the on-site soils and support the proposed structure on conventional foundations bearing on a blanket of properly placed compacted fill a minimum of 3 feet thick below footings.

Groundwater was not encountered during exploration to a depth of 51 feet below the existing grade and the historically highest groundwater level in the area is approximately 60 feet below the ground surface, according to the consultants.

The referenced reports are acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans that clearly indicates the geologist and soils engineer have reviewed the plans prepared by the design engineer; and, that the plans include the recommendations contained in their reports (7006.1).

2. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

3. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

4. A grading permit shall be obtained (106.1.2).

5. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

6. If import soils are used, no footings shall be poured until the soils engineer has submitted a compaction report containing in-place shear test data and settlement data to the Grading Division of the Department; and, obtained approval (7008.2).

7. Compacted fill shall extend beyond the footings a minimum distance equal to the depth of the fill below the bottom of footings or a minimum of three feet whichever is greater (7011.3).

8. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

9. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

10. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cubic yards (7007.1).

    638 S. Beacon St Suite 427, San Pedro (310) 732-4677

11. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

12. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring, as recommended. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

13. Where any excavation, not addressed in the approved reports, would remove lateral support (as defined in 3307.3.1) from a public way, adjacent property or structures, a supplemental report shall be submitted to the Grading Division of the Department containing recommendations for shoring, underpinning, and sequence of construction. Shoring recommendations shall include the maximum allowable lateral deflection of shoring system to prevent damage to adjacent structures, properties and/or public ways. Report shall include a plot plan and cross-section(s) showing the construction type, number of stories, and location of adjacent structures, and analysis incorporating all surcharge loads that demonstrate an acceptable factor of safety against failure. (7006.2 & 3307.3.2)

14. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

15. Unsurcharged temporary excavation may be cut vertical up to 4 feet. Excavations over 4 feet shall be trimmed back at a uniform gradient not exceeding 1:1, from top to bottom of excavation, or shored as recommended.

16. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

17. Prior to the issuance of the permits, the soils engineer and the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

18. Shoring shall be designed for the lateral earth pressures specified on page 4 of the 01/29/2021 report; all surcharge loads shall be included into the design.

19. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

20. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

21. All foundations shall derive entire support from a blanket of properly placed compacted fill a minimum of 3 feet thick, as recommended and approved by the geologist and soils engineer by inspection.

22. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

23. The foundation/slab design shall satisfy all requirements of the Information Bulletin P/BC 2017-116 "Foundation Design for Expansive Soils" (1803.5.3).

24. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

25. Concrete floor slabs placed on expansive soil shall be placed on a 4-inch fill of coarse aggregate or on a moisture barrier membrane. The slabs shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

26. The seismic design shall be based on a Site Class D, as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check. According to ASCE 7-16 Section 11.4.8, the long period coefficient (Fv) may be selected per Table 11.4-2 in ASCE 7-16, provided that the value of the Seismic Response Coefficient (Cs) is determined by Equation 12.8-2 for values of the fundamental period of the building (T) less than or equal to 1.5Ts, and taken as 1.5 times the value computed in accordance with either Equation 12.8-3 for T greater than 1.5Ts and less than or equal to TL or Equation 12.8-4 for T greater than TL. Alternatively, a supplemental report containing a site-specific ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 shall be submitted for review and approval.

27. Retaining walls up to 6 feet high with a level backfill shall be designed for the lateral earth pressures specified on page 5 of the 01/29/2021 report. All surcharge loads shall be included into the design.

28. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

29. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

30. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

31. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

32. The structure shall be connected to the public sewer system per P/BC 2020-027.

33. The infiltration facility design and construction shall comply with the minimum requirements specified in the Information Bulletin P/BC 2020-118.

34. The infiltration system shall be constructed at the location shown on Figure 2 of the 01/29/2021 report.

35. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

36. An overflow outlet shall be provided to conduct water to the street in the event that the infiltration system capacity is exceeded. (P/BC 2020-118)

37. Approval for the proposed infiltration system from the Bureau of Sanitation, Department of Public Works shall be secured.

38. A minimum distance of 10 feet (in any direction) shall be provided from adjacent proposed/existing footings to the discharge of the proposed infiltration system. A minimum distance of 10 feet horizontally shall be provided from private property lines to the proposed infiltration system.

39. The dry well area between the blank casing and the surround soils shall be sealed to a minimum depth of 28 feet below the existing grade or bottom of any adjacent foundation with bentonite slurry (or equivalent) to prevent unintended leakage or horizontal infiltration. (See page 6 of the 11/20/2020 report)

40. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

41. Any recommendations prepared by the geologist and/or the soils engineer for correction of geological hazards found during grading shall be submitted to the Grading Division of the Department for approval prior to use in the field (7008.2, 7008.3).

42. The geologist and soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading (7008, 1705.6 & 1705.8).

43. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

44. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; protection fences; and, dust and traffic control will be scheduled (108.9.1).

45. Installation of shoring shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

46. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also

inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

47. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

FOR

DANIEL C. SCHNEIDEREIT
Engineering Geologist II

LEILA ETAAT
Structural Engineering Associate II

DCS/LE:dcs/le
Log No. 115712-01
213-482-0480

cc: Leighton Consulting, Inc., Project Consultant
SP District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	Log No. 115712-01

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 221 N. Figueroa St., 12th Fl., Los Angeles, CA 90012
Telephone No. (213)482-0480.

B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.

C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: PARCEL D OF PARCEL MAP L.A. 2004-5121
Block: Lots: LOT 1 OF TRACT 65665

2. PROJECT ADDRESS:
1355 W. Sepulveda Boulevard, Torrance, CA

3. OWNER: Bridge 1355 Sepulveda, LLC
Address: 11100 Santa Monica Boulevard, Suite 700
City: Los Angeles Zip: 90025
Phone (Daytime): (530) 514-5325

4. APPLICANT Bridge 1355 Sepulveda, LLC
Address: 11100 Santa Monica Boulevard, Suite 700
City: Los Angeles Zip: 90025
Phone (Daytime): (530) 514-5325
E-mail address: tfitzpatrick@bridgedev.com

5. Report(s) Prepared by:
Leighton Consulting, Inc.

6. Report Date(s):
9/23/2020, 11/20/2020, 1/29/2021

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage

8. Previous site reports? ☑ YES if yes, give date(s) of report(s) and name of company who prepared report(s)
2/6/2020 - Southern California Geotechnical

9. Previous Department actions? ☑ YES if yes, provide dates and attach a copy to expedite processing.
Dates: Received Comments on initial submittal

10. Applicant Signature: Tom Fitzpatrick Position: Vice President

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☐ Soils Engineering		No. of Lots		
☐ Geology		No. of Acres		
☐ Combined Soils Engr. & Geol.		☐ Division of Land		363.00
☐ Supplemental		Other		
☐ Combined Supplemental		☑ Expedite		181.50
☐ Import-Export Route		☐ Response to Correction		
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		544.50
		Surcharge		129.80
		TOTAL FEE		674

Fee Due: 674.30
Fee Verified By: Date: 2-2-21
(Cashier Use Only)

Los Angeles Department of Building and Safety
Metro 4th Floor 02/04/2021 4:09:44 PM
User ID: jbitangcol
Receipt Ref Nbr: 2021035002-119
Transaction ID: 2021035002-119-1
PLAN APPROVAL FEE $181.50
SYSTEMS DEV SURCH $32.67
GEN PLAN MAINT SURCH $38.12
DEV SERV CENTER SURCH $16.34
CITY PLAN SURCH $32.67
GRADING REPORT $363.00
MISC OTHER $10.00
Amount Paid: $674.30
PCIS Number: NA
Job Address: 1355 W. SEPULVEDA BLVD
Owners Name: BRIDGE 1355 SEPULVEDA, LLC
Grading Section Log Number: 115712

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED

☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date

GEOTECHNICAL INVESTIGATION, PROPOSED WAREHOUSE/INDUSTRIAL DEVELOPMENT, 1355 WEST SEPULVEDA BOULEVARD, TORRANCE AREA, CITY OF LOS ANGELES, CALIFORNIA

Prepared For:

BRIDGE 1355 SEPULVEDA, LLC
11100 Santa Monica Boulevard, Suite 700
Los Angeles, California 90025

Project No. 12809.001

September 23, 2020

Leighton Consulting, Inc.

A LEIGHTON GROUP COMPANY

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

September 23, 2020

Project No. 12809.001

Bridge 1355 Sepulveda, LLC
11100 Santa Monica Boulevard, Suite 700
Los Angeles, California 90025

Attention: Mr. Tom Fitzpatrick

Subject: Geotechnical Investigation
Proposed Warehouse/Industrial Development
1355 West Sepulveda Boulevard
Torrance Area, City of Los Angeles, California

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has conducted this geotechnical investigation for the proposed warehouse/industrial development at the roughly 7.4-acre site located at 1355 West Sepulveda Boulevard in the Torrance area in the City of Los Angeles, California. The site is currently developed as the Mulligan Miniature Golf Park. The purpose of this study has been to collect subsurface data at the site, evaluate the proposed development with respect to the site conditions and provide geotechnical recommendations for design and construction of the development.

Based on this investigation, construction of the proposed warehouse/industrial development is feasible from a geotechnical standpoint. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Jason D. Hertzberg, GE 2711
Principal Engineer

Steven G. Okubo, CEG 2706
Project Geologist

Luis Perez-Milicua, PE 89389
Project Enginer

LP/JDH/SGO/rsm

Distribution: (1) Addressee

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Leighton

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Appendices

1.0 INTRODUCTION

1.1 Site Location and Description

The property is approximately 7.4-acres in area and is located at 1355 West Sepulveda Boulevard in the Torrance area in the City of Los Angeles, California. The majority of the property is developed as the Mulligan Miniature Golf amusement center, which includes parking areas and drive aisles in the west, northwest and northeast, amusement buildings in the north, former batting cages, miniature golf course and go-cart track to the south and southwest. The amusement park appears to have been constructed between 1982 and 1994. A building was present on the site before 1975, which appears to have been remodeled and expanded for use in the amusement park. The park appears to have ceased operations sometime in late 2019 to early 2020.

A mostly vacant parcel, not a part of the amusement park, is present in the south central portion of the property and fronts Sepulveda Boulevard to the south. A large metal structure is present in the central portion of the parcel. This or a similar structure has been present since the early 1960s. This structure is reportedly a ready mix tower associated with an Associated Ready Mix and Concrete Plant that was formerly onsite (1361 Sepulveda Blvd). Cement trucks can be seen in aerial photographs taken in this area in the mid to late 2000s until around 2011. End-dumped piles of soil can be observed in this area in photos taken in 2013.

Topographic maps reviewed appear to indicate oil wells were present in the southern portion of the site along future Sepulveda Blvd and were drilled after 1930. Aerial photographs also appear to show an oil well onsite in the early 1950's. Evidence of the wells is not obvious in subsequent aerial photographs.

The site is located in the United States Geological Survey (USGS) Torrance California 7.5-Minute Series Quadrangle, is generally flat with an average elevation of approximately 60 feet above sea level with slight gradient to the east.

1.2 Proposed Development

The Conceptual Grading Plan for the site prepared by WestLAND Group, Inc., dated July 2020 includes construction of a single, approximate 174,211-square-foot warehouse building with associated utility, drainage, parking hardscape and landscape improvements. The western portion of the proposed building includes dock-high truck loading docks.

Based on the earthwork exhibit dated September 15, 2015, we understand that site earthwork will generally include cuts of up to 4 feet and fills of up to 2 feet. Areas west of the proposed building will have about 3 to 4 feet of cut for the truck loading docks. A copy of this earthwork exhibit is included in Appendix F.

1.3 Previous Work

Southern California Geotechnical conducted a geotechnical investigation of the site earlier this year (SoCalGeo, 2020). Their work included excavation of 6 borings to a maximum depth of 25 feet, laboratory testing and analysis. Based on their work they concluded that development of the site is feasible. We have reviewed SoCalGeo's report and where appropriate incorporated the data from their report.

A 12,000-gallon underground storage tank was reportedly removed from the central portion of the site in 2012 with observation and testing by Roux Associates, Inc. (Roux, 2012, 2013). That tank was used to refuel cement trucks in the former ready mix concrete plant. The removal extended to a depth of about 15 feet below the existing ground surface.

1.4 Purpose of Investigation

The purpose of this study has been to evaluate the geotechnical conditions with respect to the proposed development and to provide geotechnical recommendations for design and construction of the development.

1.5 Scope of Investigation

Our geotechnical exploration included hollow-stem auger soil borings, laboratory testing and geotechnical analysis to evaluate existing geotechnical conditions and to develop the conclusions and recommendations contained in this report. The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical geologic maps and reports and aerial photographs available from our in-house library or available online or those provided by you. This included review of the geotechnical report prepared by SoCalGeo (2020).

- Utility Coordination: We contacted Underground Services Alert (USA) prior to excavating borings so that utility companies could mark utilities onsite. We also subcontracted a private utility locator to further locate any near-surface underground private utilities in the area of our proposed borings. We coordinated our work with you and the site representative.

- Field Exploration: A total of 5 hollow-stem auger borings were logged and sampled onsite to evaluate subsurface conditions. The borings were drilled by a subcontracted drill rig operation to depths ranging from 21.5 to 51.5 feet below the existing ground surface (bgs). Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained. Representative bulk soil samples were also collected at shallow depths from the borings.

  Excavations were backfilled and tamped with soil cuttings and patched with cold asphalt patch if drilled in asphalt pavement areas. Logs of the geotechnical borings are presented in Appendix B. The boring logs from SoCalGeo (2020) are also provided. Approximate boring locations are shown on the accompanying Boring Location Map, Figure 2.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Sieve analysis for grain-size distribution
  - Swell-Settlement
  - Maximum dry density and optimum moisture content
  - Expansion Index
  - Water-soluble sulfate concentration in the soil
  - Resistivity, chloride content and pH

In-situ moisture content and dry density are provide on the boring logs. Remaining tests are provided in Appendix C, Laboratory Test Results. Laboratory results from SoCalGeo (2020) are also provided.

- Engineering Analysis: Data obtained from our background review, along with data from our field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.

- Report Preparation: Results of our geotechnical exploration have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical recommendations for design and construction of the proposed development.

2.0 FINDINGS

2.1 Regional Geologic Conditions

The site is located within the Los Angeles Basin in the northern portion of the Peninsular Range Geomorphic Province of California. Geologic units of the region consist of Pleistocene and Holocene aged colluvium/alluvium along with Miocene, Pliocene, and Pleistocene siltstones, sandstones, and conglomerates of the Puente, Fernando, and La Habra Formations. Major structural features surrounding this region include the north-northwest trending Whittier Fault and Puente Hills to the north and northeast, the Elysian Park Fold and Thrust Belt and Compton Thrust Fault to the northwest and west, and the Newport-Inglewood Fault offshore to the southwest. In addition, this is an area of large-scale crustal disturbance as the relatively northwestward-moving Peninsular Range Province collides with the Transverse Range Province (including the San Gabriel Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this collision. The Newport-Inglewood is the closest known active fault and is located approximately 5½ miles northeast of the site, transecting the southern slopes of the Puente Hills. The site is mapped as being underlain by slightly elevated, dissected alluvial soil deposits.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by alluvial soil deposits mantled with artificial fill. Artificial fill, was reported by SoCalGeo to depths up to 7 feet in the central and southern portion of the site and consisted of loose to medium dense silty sand. They reported some debris, glass and brick fragments in the fill. We observed only minor artificial fill in our borings. Based on the site topography, artificial fill may be present under the miniature golf course where we could not access.

An underground storage tank was present in the area of the former ready mix plant. The approximate location of the removed tank is shown on the Boring Location Map, Figure 2. The UST was removed in 2012 (Roux, 2012). It appears the roughly 15-foot-deep excavation was backfilled with gravel. We could find no documentation that the backfill was compacted.

The alluvial soil encountered within our borings generally consisted of combinations of sand and silt, with some clay interspersed. In general, the alluvial soil in the upper 15 feet consisted of medium dense to dense silty sand. These soils tended to be moist. SoCalGeo reported some very moist soils. Clay layers were generally encountered at depths of 15 to 25 feet. Cross-sections showing the encountered subsurface conditions are provided in Figures 3A and 3B.

The moisture content of the near surface soils ranged from 4 to 15 percent, and the dry density of the near surface soil ranged from 92 to 115 pcf. Laboratory testing performed shows the near-surface soils maximum dry density in a range from 123.5 to 132 pcf at 8.5 to 9 percent optimum moisture content.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on this study, undocumented artificial fill and the upper portion of native soils are considered slightly to moderately compressible. Complete removal of undocumented fill and partial removal of near surface alluvium is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Based on testing by Leighton and SoCalGeo, the onsite soils are anticipated to have a negligible collapse potential when inundated with water.

2.2.2 Expansive Soils

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

A near surface sample of the soil collected during our study was tested for expansion potential yielding an expansion Index of 1. SoCalGeo testing yielded an expansion Index of 19 for near-surface soils. Based on this testing the onsite near-surface soil is expected to have a very low to low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on American Concrete Institute (ACI) provisions, adopted by the 2019 CBC (CBC, 2019, Chapter 19, and ACI 318, 2014).

A near-surface soil sample was tested during this investigation for soluble sulfate content. The results of these tests indicate sulfate contents of less than 0.02 percent by weight, indicating negligible sulfate exposure. SoCalGeo (2020) also indicated negligible levels of soluble sulfates. Recommendations for concrete in contact with the soil are provided in Section 3.11.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, representative soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 1,898 ohm-cm, chloride content of 81 ppm, and pH of 7.9. SoCalGeo (2020) indicated resistivity of 1,840 ohm-cm, chloride content of 15mg/kg and a pH of 7.9 Based on these results, the onsite soil is considered corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in any of our borings drilled to a maximum depth of 51 feet bgs during our investigation. Several water wells are present within about a 1.7-mile radius of the site. Well data dating back to 1934 (CDRW, 2018, LA County DPW, 2020 and Water Replenishment District, 2020) indicates water levels in the area at depths in excess of 50 feet with most recent water levels in the range of 80 feet below the ground surface.

We reviewed the Seismic Hazard Report for the Torrance 7.5-minute quadrangle (CGS, 1998), in which ahistoric high groundwater contour of 10 feet is mapped approximately 1 mile southeast of the site on Plate 1.2 of that report. However, groundwater contours west of that area are not shown on that plate. Furthermore, the project site ground surface is about 40 feet higher in elevation to the closest mapped historic high groundwater contour. Thus, shallow groundwater contours are not mapped for the site.

Based on our review of available groundwater data, we have used a historic high groundwater elevation in excess of 50 feet below the ground surface for our liquefaction analysis.

2.3.1 Regional Subsidence

Regional ground subsidence generally occurs due to rapid and intensive removal of subterranean fluids, typically water or oil. It is generally attributed to the consolidation of sediments as the fluid in the sediment is removed. The total load of the soils in partially saturated or saturated deposits is born by their granular structure and the fluid. When the fluid is removed, the load is born by the sediment alone and it settles.

No reports of regional subsidence have been reported in the site vicinity, and lack of intense removal of significant quantities of water or oil extraction in the area makes the potential for ground subsidence very low and less than a significant impact.

2.4 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region include surface rupture along active faults and strong ground shaking. The potential for fault rupture and seismic shaking are discussed below.

2.4.1 Surface Faulting

Based on our research, no active faults appear to have been mapped on or trending toward the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Palos Verde	2.7 miles to the south
Compton Thrust	3.2 miles to the northeast
Newport-Inglewood	5.5 miles to the east

A listing of active faults within a 62-mile search radius is presented in Appendix D. Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered very low.

2.4.2 Seismic Design Parameters

The site is anticipated to experience strong ground shaking during the life of the project resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly, the project should be designed 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). 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 parameters should be considered for design under the 2019 CBC:

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude: 33.8154, -118.3018	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.760 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.639 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.700*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.760 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.086* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.173 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.724* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.777 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.854 g

* Per Table 11.4-2 of Supplement 1 of ASCE 7-16, this value of F_v may only be used to calculate T_s [that note is not included in Table 1613A.2.3(2)]; note that S_{D1} and S_{M1} are functions of F_v. In addition, per Exception 2 of 11.4.8 of ASCE 7-16, special equations for C_s are required. This is in lieu of a site-specific ground motion hazard analysis per ASCE 7-16 Chapter 21.2.

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

Based on the 2019 CBC Table 1613.2.3(2) footnote c., F_v should be determined in accordance with Section 11.4.8 of ASCE 7-16, since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D; in accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is required. However, the values provided in the table above may be utilized if design is performed in accordance with Exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_S), and F_v is only used for calculation of T_S. This exception does not apply (and the values in the table above would not be applicable) for proposed structures with seismic isolation or seismic damping systems. The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.3

(M_W) at a distance on the order of 5.0 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The site is not mapped in a zone of required investigation on the Seismic Hazard Zone Map for the Torrance Quadrangle (CGS, 2009) and shallow groundwater conditions are not expected at the site (see Section 2.3 and CGS, 1998).

Based on the dense nature of the soil and the absence of shallow groundwater, the subsurface soils are not considered susceptible to liquefaction.

2.5.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_m). The results of our analyses indicate that the onsite soils are susceptible to low seismic settlement (1.2 inch or less, with maximum differential settlement of 0.6 inch over a horizontal distance of 40 feet based on the MCE). Results of our seismic settlement analysis is presented in Appendix D.

2.5.3 <u>Lateral Displacement/Spread</u>

Depending on the site topography, modes of seismically induced lateral ground displacement associated with soil liquefaction consist of ground oscillation (typically with ground slope less than 0.3 percent), lateral spread (typically with 0.3 to 5 percent ground slope), or flow failure (typically ground slope greater than 5 percent). Because liquefaction is not considered a hazard at the site, seismically induced lateral ground displacements are also not considered to be hazards at the site.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

Although not identified during this investigation, abandoned septic tanks, seepage pits, or other buried structures, trash pits, or items related to past site uses are probably present. As such items are encountered during grading, they will require further evaluation and special consideration.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of debris, which should be disposed of offsite. Any underground obstructions should be removed. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted. Trees should be removed and grubbed out.

The parcel includes existing structures; existing foundation systems should be removed.

3.1.2 Removal of Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of the onsite alluvial soil, any clean uncontrolled artificial fill should be removed and may be used as compacted fill for the project, provided any deleterious materials are removed from the site. Across most of the site, undocumented fill is expect to be a few feet in thickness, but up to 7 feet locally. The depth of undocumented fill is expected to range to about 15 feet in the area of the former underground storage tank.

3.1.3 Overexcavation and Recompaction

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

All artificial fill should be removed, including the shallow artificial fill encountered within borings, the former tank removal backfill (see Section 2.2), and any other oreas where artificial fill is encountered. In addition, for the proposed structures, we recommend that the onsite soils be overexcavated to a minimum depth of 5 feet below the existing ground surface or 3 feet below the bottom of the proposed footings, whichever is deeper, across the building pad. Where possible, the removal bottom should extend horizontally a minimum of 5 feet from the outside edges of the building footprint and footings (including columns connected to the buildings), or a distance equal to the depth of overexcavation below the footings, whichever is farther. During overexcavation, the soil conditions should be observed by Leighton to further evaluate these recommendations based on actual field conditions encountered. A firm removal bottom should be established across the building footprint to provide uniform foundation support for the proposed structure. Leighton should observe and test the removal bottom prior to placing fill. Deeper overexcavation and recompaction may be recommended locally until a firm removal bottom is achieved.

Areas outside of the proposed structures planned for new asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), site walls and low retaining walls (taller walls should be overexcavated per the recommendations for buildings), areas to receive fill, and other improvements, should be overexcavated to a minimum depth of 18 inches below existing grade or 18 inches below proposed subgrade (including the footing subgrade for walls), whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.1.4 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should also be free of organic material debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary to at least 2 percentage points above optimum moisture content, and compacted to a minimum 90 percent relative compaction. However, the upper 36 inches of fill under the building pads should be compacted to a minimum of 95 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.5 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.6 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. This value does not factor in removal of debris or other materials. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15 +/- 3 percent
Subsidence (overexcavation bottom processing)	Approximately 0.15 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.7 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 8 inches in dimension) was not observed during our investigation. Oversized material should not be used within structural fill areas.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a very low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment per code requirements, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,250 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.35. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.2.5 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

Seismic differential settlement is assumed to be approximately 0.6 inch over a horizontal distance of 40 feet for the design-level earthquake.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a very low expansion potential and considering the potential for liquefaction and seismic settlement. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture vapor retarder, steel or concrete.

- Moisture Retarder: A minimum of 10-mil moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a capillary break should be placed under the vapor retarder and whether or not a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

- Concrete Thickness and Reinforcement in Warehouse/Industrial Areas: Warehouse/industrial slabs-on-grade should be designed by the structural engineer based on anticipated wheel, equipment, and storage loads. Considering the site conditions, we recommend a minimum slab thickness of 6 inches. Crack control joints should be provided at a maximum spacing of 15 feet on center.

  The structural engineer should consider the following parameters.

  Provided that the slab subgrade soils are compacted to a minimum of 95 percent relative compaction at 1 to 2 percentage points above optimum (as measured by ASTM D 1557), an average subgrade spring constant (modulus of subgrade reaction, k) of 200 pci (with linear deflections up to ¾ inch and a non-linear response for larger deflections) may be assumed for analysis of loading on slabs-on-grade. This value should not be used for estimation of actual settlements, but is intended to estimate shears, moments, and local distortions. An alternate check may be used by assuming an allowable bearing pressure of 1,200 psf (though the modulus of subgrade reaction method is the preferred method). If soils are allowed to dry out prior to placing concrete, the upper 9 inches should be scarified, moisture conditioned to 1 to 2 percentage points above optimum moisture content, and recompacted to a minimum of 95 percent relative compaction (based on ASTM D1557) prior to placing steel or concrete.

- Concrete Thickness--Office Areas: Slabs-on-grade for office space should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced, 4-inch-thick slabs) should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Crack control joints should be provided at a maximum spacing of 15 feet on center for office areas.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking 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. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the current CBC. The CBC seismic design parameters listed in Table 1 of Section 2.4 of this report should be considered for the seismic analysis of the subject site.

3.5 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 4 (rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	38 pcf
At-Rest	59 pcf
Passive	260 pcf (allowable) (Maximum of 3,000 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

A seismic increment load of 43 pcf should be added to the active case when checking seismic stability of walls over 6 feet tall.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

3.6 Pavement Design

Flexible Pavements: Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 40, flexible pavement sections may consist of the following for the Traffic Index indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

ASPHALT PAVEMENT SECTION THICKNESS		
Traffic Index	**Asphaltic Concrete (AC) Thickness (inches)**	**Class 2 Aggregate Base Thickness (inches)**
5 or less	3.0	4.0
6	3.5	5.5
7	4.0	7.0
8	5.0	8.0

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

***Rigid Pavements*:** For onsite Portland Cement Concrete (PCC) pavement in truck drive aisles and parking areas, we recommend a minimum of 7-inch-thick concrete with dowels at construction joints, placed on compacted fill subgrade, with the upper 8 inches compacted to a minimum of 95 percent relative compaction. In areas with car traffic only, we recommend a minimum of 5-inch-thick concrete, placed on compacted fill subgrade with the upper 8 inches compacted to a minimum of 95 percent relative compaction.

The PCC pavement sections should be provided with crack-control joints spaced no more than 15 feet on center each way. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement.

***Other Pavement Recommendations*:** Irrigation adjacent to pavements without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

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 slope, 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 structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should 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-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or in accordance with Greenbook specifications. The native backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction. The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook).

3.9 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved areas.

3.10 Sulfate Attack and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. The concrete should be designed in accordance with Table 19.3.2.1 of the American Concrete Institute ACI 318-14 provisions (ACI, 2014).

The onsite soil is considered to be corrosive to ferrous metals. It is recommended that any buried pipe be made of non-ferrous material, or that any ferrous pipe be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if metallic utilities are planned.

3.11 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our supplemental geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. 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, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton Consulting, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Bridge 1355 Sepulveda, LLC for application to the design of the proposed warehouse/industrial development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.

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, clients can benefit from a lowered exposure to the subsurface problems 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 below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs 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. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project 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.*

This Report May Not Be Reliable

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

- for a different client;
- for a different project;
- 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, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been 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 your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

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

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide 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 judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed 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 full-time 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 on hand quickly 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 observation.

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 informational 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, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. 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. 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 relate any 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 yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

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, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled 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.***

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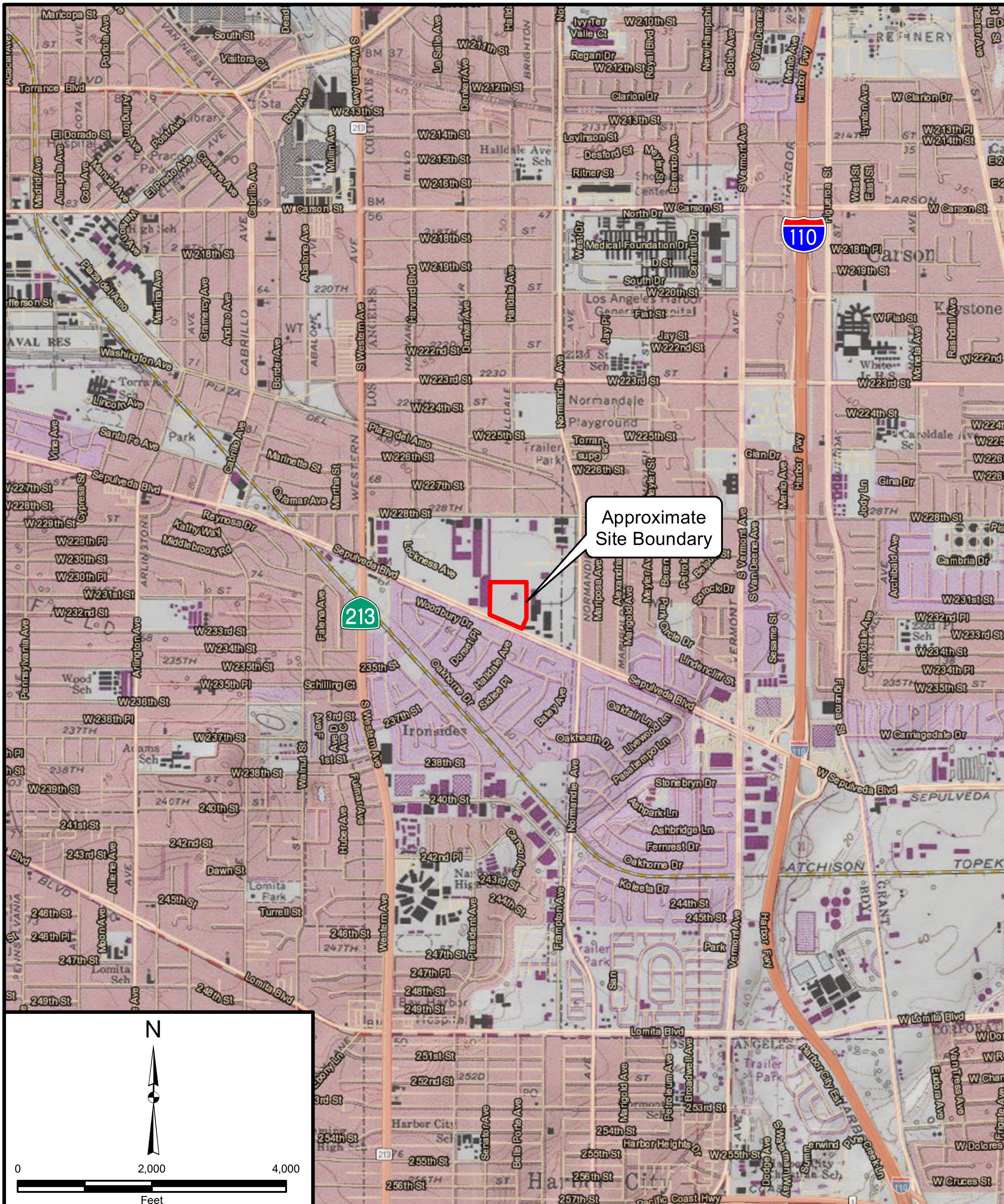
e-mail: info@geoprofessional.org   www.geoprofessional.org



Project: 12809.001	Eng/Geol: JDH/SO
Scale: 1 " = 2,000 '	Date: September 2020

Base Map: ESRI ArcGIS Online 2020

Author: (mmurphy)

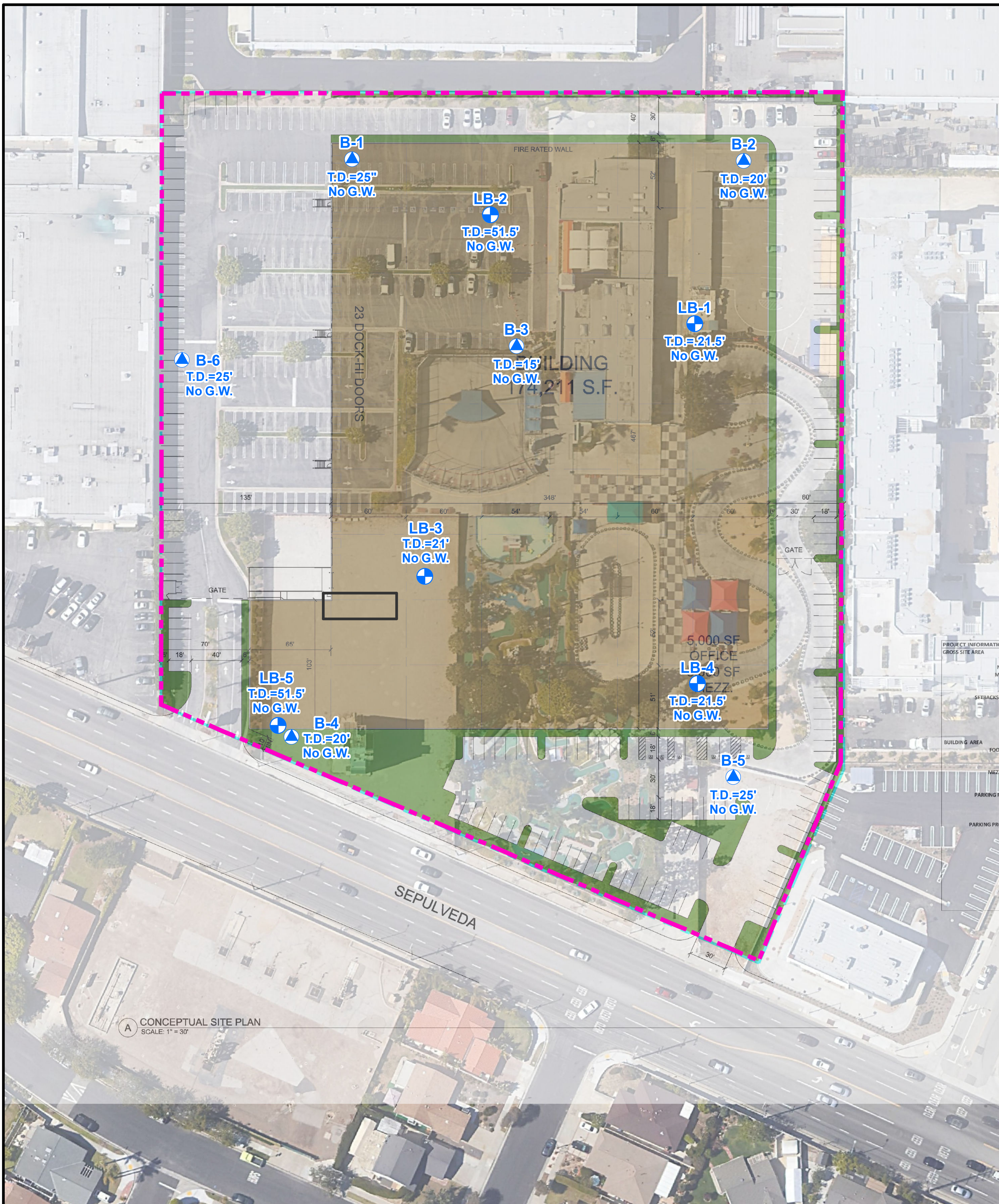
SITE LOCATION MAP

Proposed Warehouse
1355 W. Sepluveda Blvd
City of Los Angeles, California

Figure 1

Leighton

Map Saved as V:\Drafting\12809\001\Maps\12809-001_F01_SLM_2020-07-23.mxd on 7/23/2020 4:02:16 PM



Legend

LB-5
T.D.=51.5'
No G.W.

Approximate Location of Boring showing Total Depth of Boring and Groundwater (Leighton, 2020)

B-6
T.D.=25'
No G.W.

Approximate Location of Boring showing Total Depth of Boring and Groundwater (Southern California Geotechnical, 2020)

Approximate Former Location of UST (Roux, 2012)

Approximate Site Boundary

N

0 80 160

Feet

Project: 12809.001	Eng/Geol: JDH/SO
Scale: 1 " = 80 '	Date: September 2020

Base Map: Google Earth, 2020 and Conceptual Site Plan by Herdman Architecture + Design, 5/12/2020.

Author: (mmurphy)

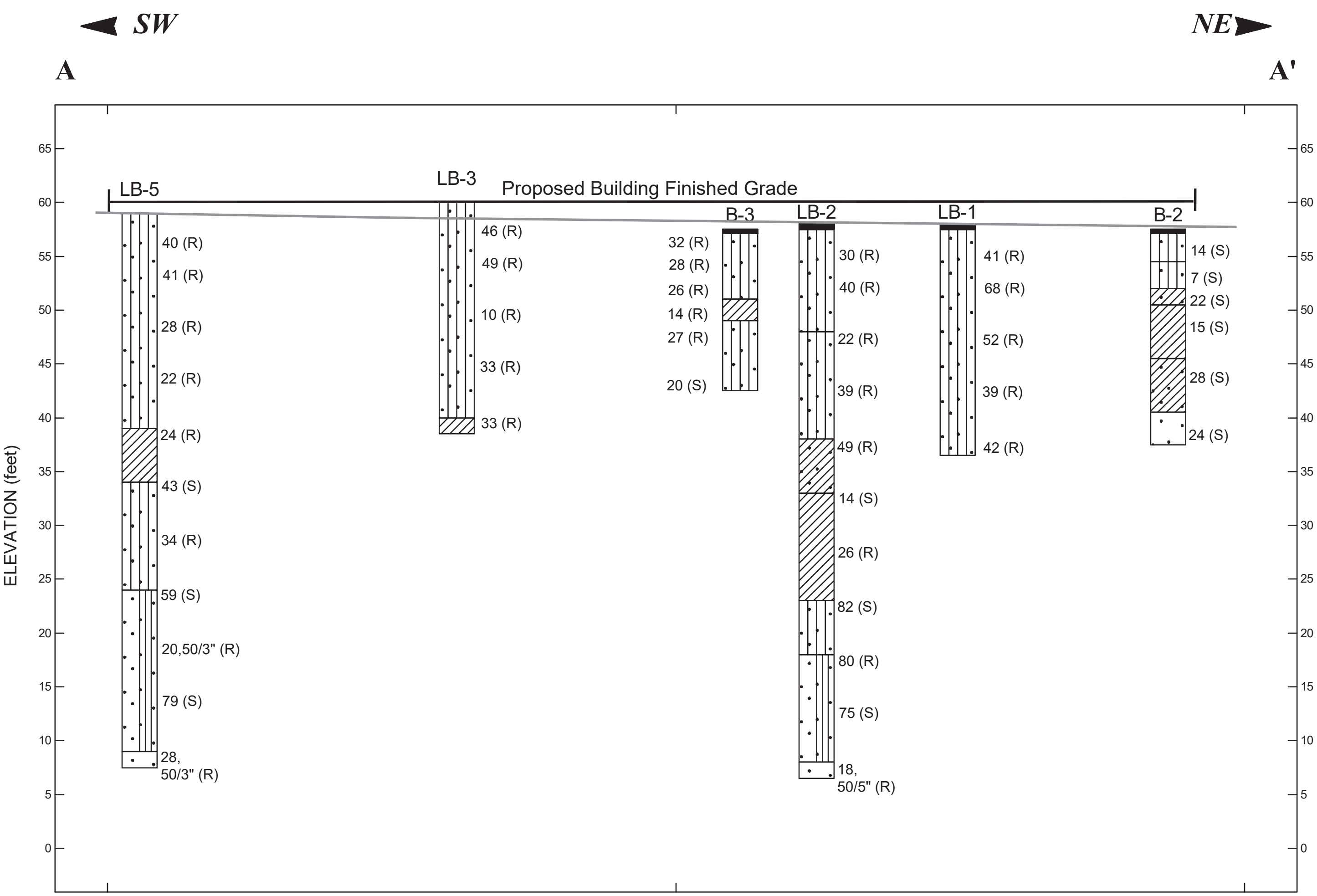
BORING LOCATION MAP

Proposed Warehouse
1355 W. Sepluveda Blvd
City of Los Angeles, California

Figure 2

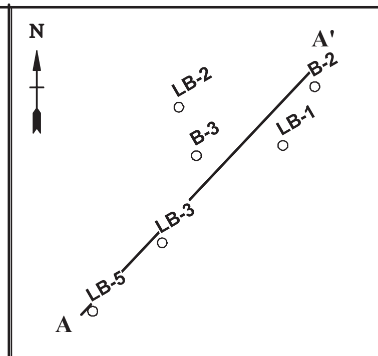
Leighton

Map Saved as V:\Drafting\12809\001\Maps\12809-001_F02_BLM_2020-07-28.mxd on 7/28/2020 11:21:33 AM



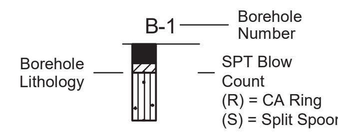
Lithology Graphics

- Asphalt
- USCS Poorly-graded Sand
- USCS Silty Sand
- USCS Poorly-graded Sand with Silt
- USCS Clayey Sand
- USCS Low Plasticity Clay



Cross-Section Location

Explanation



Water Level Reading at time of drilling (n/a)

Approx. Existing Grade

Not to Scale

Leighton Consulting, Inc.

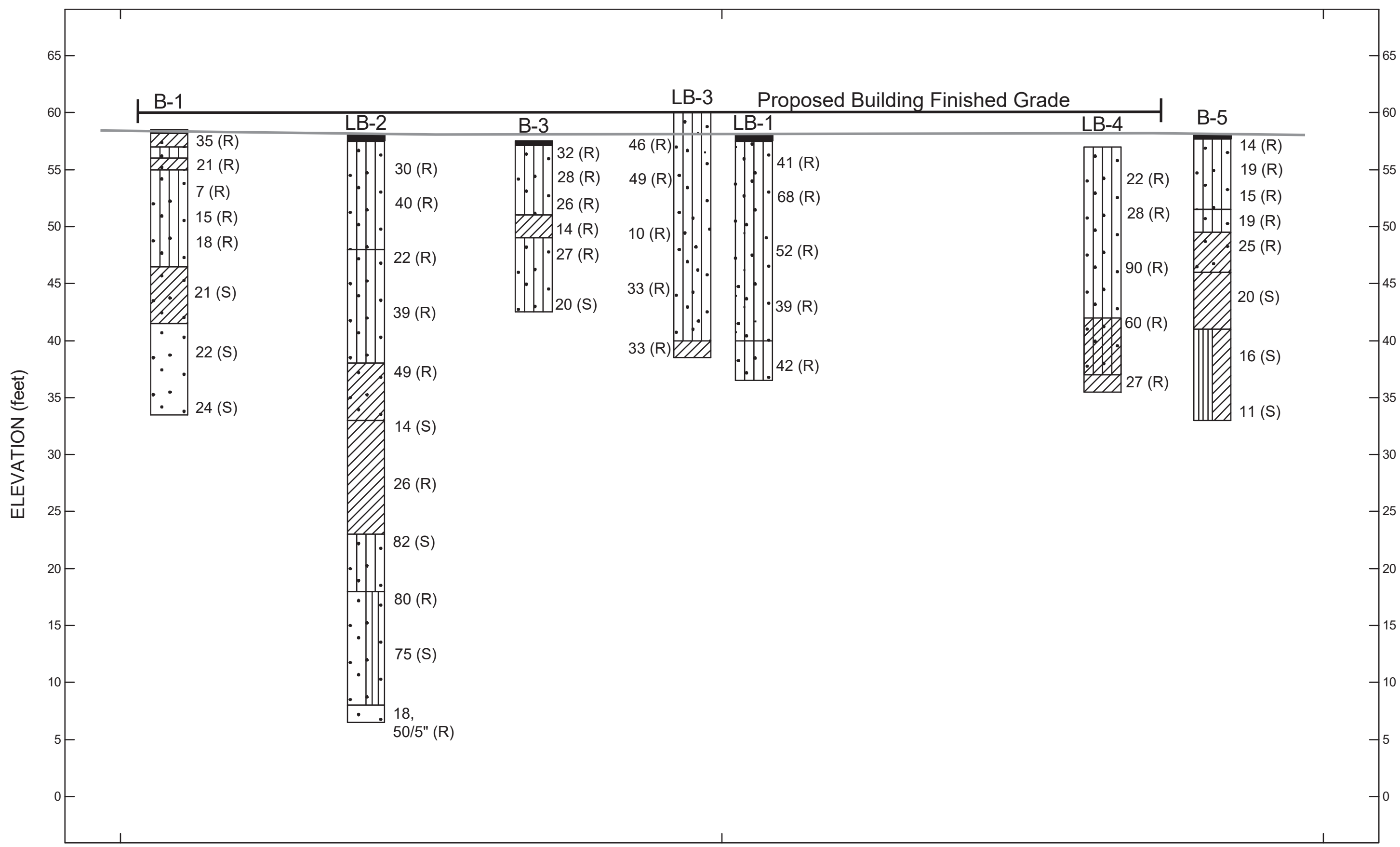
Cross Section A-A'

Proposed Warehouse
Torrance Area, California

JOB NUMBER	FIGURE NUMBER
12809.001	Figure 3A

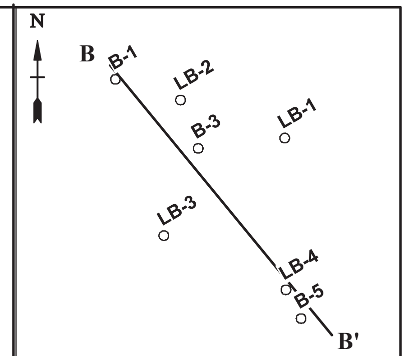
FNC AB NWNL01 - A SIZE BORING LOGS - LEIGHTON & SOCALGEO.GPJ ROCKLOG2012.GDT 9/22/20

◄ *NW* B — B' *SE* ►



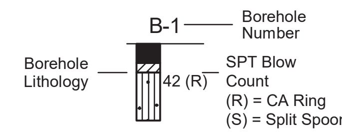
Lithology Graphics

- Asphalt
- USCS Low Plasticity Clay
- USCS Clayey Sand
- USCS Low Plasticity Silty Clay
- USCS Silty Sand
- USCS Poorly-graded Sand with Silt
- USCS Poorly-graded Sand



Cross-Section Location

Explanation



Water Level Reading at time of drilling (n/a)

Approx. Existing Grade

Not to Scale

Leighton Consulting, Inc.

Cross Section B-B'

Proposed Warehouse
Torrance Area, California

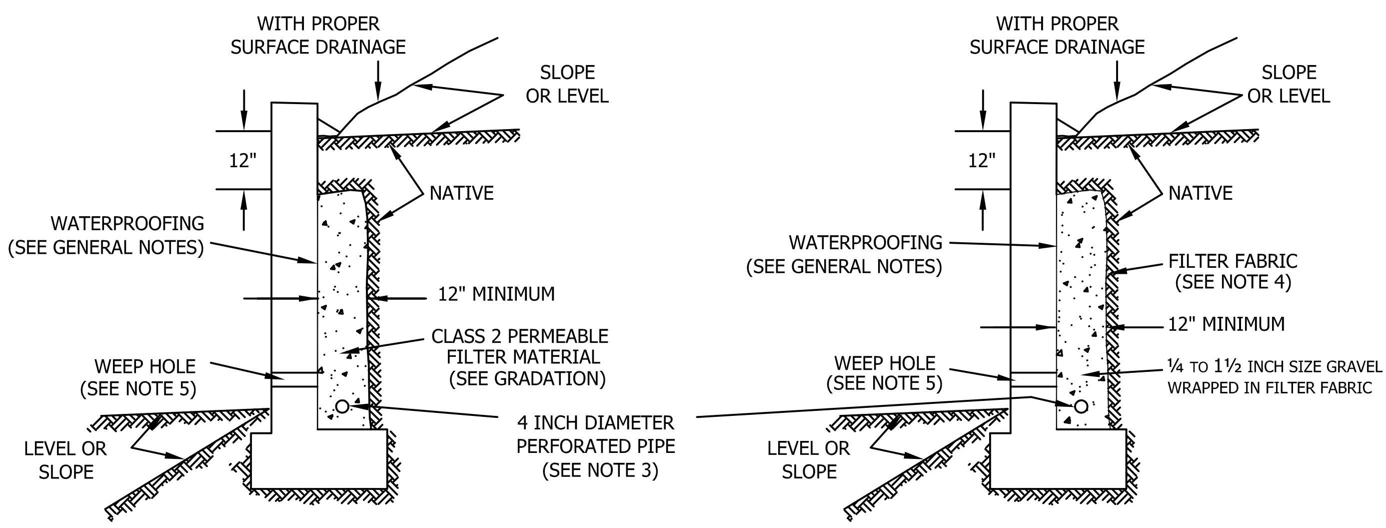
JOB NUMBER	FIGURE NUMBER
12809.001	Figure 3B

FNC AB NWNL01 - A SIZE BORING LOGS - LEIGHTON & SOCALGEO.GPJ ROCKLOG2012.GDT 9/22/20

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

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 4

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL LOGS

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APPENDIX B

FIELD EXPLORATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration. Five borings (LB-1 through LB-5) were excavated and logged to a maximum depth of approximately 51.5 feet below the existing ground surface. These boring logs are included as part of this appendix. Approximate soil boring locations are shown on Figure 2, *Boring Location Map*.

Borings: On July 10, 2020, 5 hollow-stem-auger borings were drilled, logged and sampled to depths ranging from 21.5 feet to 51 feet below the ground surface. Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using both a California ring-lined and Standard Penetration Test (SPT) split-spoon sampler. Standard Penetration Test (SPT) resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch outside diameter split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D 1586). In addition, 2.4-inch inside diameter brass ring samples were obtained using a Modified California sampler driven into the soil with the 140-pound hammer. Near surface bulk soil samples were also collected from the borings. Representative earth-material samples obtained from these subsurface explorations were transported to our geotechnical laboratory for evaluation and appropriate testing.

GEOTECHNICAL BORING LOG LB-1

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	58'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								**Alluvium (Qal)**	
				B1						
55				R1	9 19 22	98	4	SM	@2.5': SILTY SAND (SM), medium dense, brown, slightly moist, fine to medium sand, >1" gravel (field estimate), 29% fines (lab)	, SA
	5			R2	16 24 44	111	8	SM	@5': SILTY SAND (SM), dense, brown, slightly moist, fine to coarse sand, 31% fines (lab), some gravel at top of sample	, SA
50										
	10			R3	14 22 30	96	9	SM	@10': SILTY SAND with clay (SM), dense, brown to orange, moist, fine to coarse sand, subangular, >20% fines (field estimate)	
45										
	15			R4	13 18 21	95	4	SM	@15': SILTY SAND (SM), medium dense, brown to dark brown, moist, medium to coarse sand, 20-30% fines (field estimate), darker brown and siltier at top of samples, twig (1) found in darker sample, lighter at bottom, ferrous staining	
40										
	20			R5	12 16 26			SM	@20': SILTY SAND (SM), medium dense, brown, moist, fine to coarse sand, subangular, 30-35% fines (field estimate), ferrous staining	
35									**Total Depth: 21.5 feet** **Groundwater not encountered.** **Backfilled with grout, tamped with cold patch, spoils drummed**	
	25									
30										
	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-2

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	58'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								**Alluvium (Qal)**	
55				R1	12 14 16	115	10	SM	@2.5': SILTY SAND (SM), medium dense, dark brown, slightly moist, fine to coarse sand, some gravel >1" (field estimate), 26% fines (lab)	SA
	5			R2	13 20 20	96	9	SM	@5': SILTY SAND (SM), medium dense, brown, moist, fine to medium sand, less gravel than above, 40-50% fines (field estimate),	
50										
	10			R3	5 11 11	107	18	SM	@10': SILTY SAND (SM), medium dense, brown to orange, moist, 35% fines (lab), ferrous staining	-200, CN
45										
	15			R4	7 15 24	101	18	SM	@15': SILTY SAND (SM), medium dense, brown, moist, fine to coarse sand, subangular, 30-35% fines (field estimate),	
40										
	20			R5	11 21 28	96	8	SC	@20': CLAYEY SAND (SC), medium dense, brown to light brown, moist, medium to coarse sand, subangular, 40-45% fines (field estimate), ferrous staining	
35										
	25			S1	5 4 10			CL	@25': CLAY (CL), stiff, brown to orange, moist, ferrous staining, 86% fines (lab)	-200
				B1						
30										
	30									

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

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

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

GEOTECHNICAL BORING LOG LB-2

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	58'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R6	5 11 15	85	36	CL	@30': CLAY (CL), stif, brown, moist, fine to medium sand, 30-40% fines (field estimate), ferrous staining	-200
25										
	35			S2	17 32 50/6"			SM	@35': SILTY SAND (SM), very dense, light brown to brown, moist, fine to coarse sand, >20-30% fines (field estimate), laminal ferrous staining	
20										
	40			R7	25 31 49	99	5	SP-SM	@40': SAND with silt (SP-SM), dense, brown, moist, coarse sand, subangular, >10% fines (field estimate), darker and siltier in top 1/3, lighter colored at bottom	
15										
	45			S3	24 28 47			SP-SM	@45': SAND with silt (SP-SM), dense, light brown to orange, moist, coarse sand, subangular, >10% fines (field estimate), ferrous staining	
10										
	50			R8	18 50/5"			SP	@50': SAND (SP), very dense, light brown to brown, moist, medium to coarse sand, subangular	
									Total Depth: 51.5 feet **Groundwater not encountered.** **Backfilled with grout, tamped with cold patch, spoils drummed**	
5										
	55									
0										
	60									

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

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

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

GEOTECHNICAL BORING LOG LB-3

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	60'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
60	0								@surface: minor vegetation, debris, silty sand with gravel **Alluvium (Qal)**	
				R1	11 19 27	111	15	SM	@2.5': SILTY SAND (SM), medium dense, dark brown, moist, fine to coarse sand, subangular, some gravel >1" (field estimate), >20-30% fines (field estimate)	
55	5			R2	9 21 28	112	13	SM	@5': SILTY SAND with some clay (SM), medium dense, dark brown, moist, medium to coarse sand, some gravel >1" (field estimate), 40% fines (field estimate)	
50	10			R3	4 4 6			SM	@10': Failure to retrieve, loose, possibly clay	
				B1						
45	15			R4	8 11 22	116	13	SM	@15': SILTY SAND (SM), medium dense, brown to orange, moist, medium to coarse sand, 30-40% fines (field estimate),	
40	20			R5	7 13 20	94	28	CL	@20': CLAY (CL), very stiff, brown to orange, moist, ferrous staining, 85% fines (lab)	
									Total Depth: 21.5 feet **Groundwater not encountered.** **Backfilled with grout, spoils drummed**	
35	25									
30	30									

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

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

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

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

GEOTECHNICAL BORING LOG LB-4

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	57'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								@surface: minor vegetation, silty sand **<u>Alluvium (Qal)</u>**	
55										
				R1	6 11 11	102	5	SM	@2.5': SILTY SAND (SM), medium dense, brown, moist, fine to medium sand, 20-30% fines (field estimate), gravel 1-2" (field estimate)	
	5			R2	11 13 15	106	5	SM	@5': SILTY SAND (SM), medium dense, brown, moist, fine to coarse sand, >20% fines (field estimate), ferrous staining	
50										
	10			R3	18 29 50/5"	108	13	SM	@10': SILTY SAND with clay (SM), very dense, brown, moist, fine to coarse sand, >20% fines (field estimate), ferrous staining	
45										
	15			R4	12 26 34	106	18	CL	@15': SANDY CLAY (CL), hard, brown to orange, moist, fine to coarse sand, subangular, 53% fines (lab), ferrous staining	
40										
	20			R5	9 12 15	89	31	CL	@20': CLAY (CL), very stiff, brown to orange, moist, fine to medium sand, 50-60% fines (field estimate), ferrous staining clayier at bottom,	
35									**Total Depth: 21.5 feet** **Groundwater not encountered.** **Backfilled with grout, spoils drummed**	
	25									
30										
	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-5

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	59'
Location	See Figure 2	**Sampled By**	AIK

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							@surface: minor vegetation, debris, silty sand with gravel **Alluvium (Qal)**	
			B1						
			R1	7 19 21	92	9	SM	@2.5': SILTY SAND (SM), medium dense, dark brown, moist, fine to medium sand, some gravel 2" (field estimate), 20-30% fines (field estimate)	
55	5		R2	4 15 26	107	12	SM	@5': SILTY SAND (SM), medium dense, dark brown, moist, medium to coarse sand, less gravel than above sample, 20-30% fines (field estimate)	
50	10		R3	6 12 16	106	20	SM	@10': SILTY SAND with clay (SM), medium dense, brown to orange, moist, fine to coarse sand, fines 30-40% (field estimate), some ferrous staining, little to no gravel, sandier at top of sample	
45	15		R4	2 8 14	106	20	SM	@15': SILTY SAND (SM), medium dense, brown, moist, fines 40-50% (field estimate), ferrous lamination	
40	20		R5	4 9 15	88	34	CL	@20': CLAY (CL), stiff, orange to brown, moist, sandy at top of sample, ferrous stainig	
35	25		S1	6 15 28			SM	@25': SILTY SAND (SM), medium dense, light brown, moist, medium to coarse sand, clayey and siltier at top of sample	
30	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-5

Project No.	12809.001	**Date Drilled**	7-10-20
Project	Warehouse/Industrial Development	**Logged By**	AIK
Drilling Co.	Cascade	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	59'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R6	12 14 20	95	11	SM	@30': SILTY SAND (SM), medium dense, light brown to brown, moist, fine to coarse sand, subangular, sandier at middle and bottom of sample, fines >30% (field estimate), ferrous staining	
25										
	35			S2	13 28 31			SP-SM	@35':SAND with silt (SP-SM), dense, light brown to dark brown, slightly moist, medium to coarse sand, subangular, dark brown towards top, light brown towards bottom, fines >10% (field estimate), ferrous staining	
20										
	40			R7	20 50/3"	96	6	SP-SM	@40': SAND with silt (SP-SM), very dense, light brown, moist, coarse sand, subangular, siltier towards top 1/3 of sample, fines >20% (field estimate)	
				B2						
15										
	45			S3	29 50/6"			SP-SM	@45': SAND with silt (SP-SM), very dense, brown, moist, medium to coarse sand, subangular, some rock in sample possibly shale (light gray & platy-field estimate),	
10										
	50			R8	28 50/3"			SP	@50': SAND (SP), very dense, light brown, moist, medium to coarse sand, subangular	
									Total Depth: 51.5 feet **Groundwater not encountered.** **Backfilled with grout, spoils drummed**	
5										
	55									
0										
	60									

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

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
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- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
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- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G102-1
PROJECT: Proposed Industrial Building
LOCATION: Los Angeles, California

DRILLING DATE: 1/23/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					4± inches Asphaltic concrete, 4± inches Aggregate base							
		35			FILL: Brown to Dark Brown Clayey fine Sand, mottled, trace Asphaltic concrete and glass fragments, medium dense-moist to very moist	93	18					EI = 19 @ 0 to 5 feet
		21			FILL: Brown Silty fine Sand, trace fine to coarse Gravel, mottled, medium dense-moist	94	23					
5		7			FILL: Dark Brown to Black Clayey fine to medium Sand, trace coarse Sand, trace fine Gravel, mottled, trace Asphaltic concrete fragments, medium dense-very moist	94	8					
		15			ALLUVIUM: Brown Silty fine Sand, loose to medium dense-moist to very moist @ 5 feet, trace organic fibers	102	15					
10		18				107	17					
					Light Brown Clayey fine Sand, trace Iron oxide staining, medium dense-very moist							
15		21					22					
					Light Brown fine Sand, medium dense-damp to moist							
20		22					7					
25		24			@ 23½ feet, little Iron oxide staining		10					
					Boring Terminated at 25'							

TBL 20G102-1.GPJ SOCALGEO.GDT 2/6/20

BORING NO. B-2

JOB NO.: 20G102-1
PROJECT: Proposed Industrial Building
LOCATION: Los Angeles, California

DRILLING DATE: 1/23/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 15 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic concrete, 5± inches Aggregate base							
	X	14			FILL: Light Brown to Dark Brown Silty fine Sand, mottled, faint hydrocarbon odor, medium dense-moist		10					
	X	7			ALLUVIUM: Light Brown Silty fine Sand, loose-moist		9					
5	X	22			Light Brown Clayey fine Sand, little Silt, medium dense-moist		14					
	X	15	4.5		Light Brown fine Sandy Clay, abundant Iron oxide staining, very stiff-very moist		19					
10												
	X	28			Light Brown Clayey fine Sand, medium dense-moist		15					
15												
	X	24			Light Brown fine Sand, trace Iron oxide staining, medium dense-damp		7					
20												

Boring Terminated at 20'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G102-1	DRILLING DATE: 1/23/20	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 11 feet
LOCATION: Los Angeles, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic concrete, 5± inches Aggregate base							
		32			FILL: Brown Silty fine Sand, trace fine Gravel, trace Clay nodules, mottled, medium dense-moist to very moist	104	14					
		28			@ 3 feet, little medium to coarse Sand, trace coarse Gravel, trace Brick fragments	114	13					
5		26				102	17					
		14	4.0		ALLUVIUM: Light Brown fine Sandy Clay, trace porosity, trace Iron oxide staining, very stiff-moist to very moist	106	18					
		27			Light Brown Silty fine Sand, medium dense-moist to very moist	104	19					
10												
		20			@ 13½ feet, trace Iron oxide staining		11					
15												
					Boring Terminated at 15'							

TBL 20G102-1.GPJ SOCALGEO.GDT 2/6/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G102-1
PROJECT: Proposed Industrial Building
LOCATION: Los Angeles, California

DRILLING DATE: 1/23/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 15 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
			1-inch poorly graded Gravel							
	13		FILL: Brown Silty fine Sand, trace coarse Sand, trace fine Gravel, trace Clay nodules, mottled, medium dense-very moist		14					
	25		FILL: Gray Silty fine Sand, trace fine Gravel, faint hydrocarbon odor, medium dense-moist		13					
5	20		ALLUVIUM: Light Brown Clayey fine Sand, medium dense-moist to very moist		17					
	14	2.0	Light Brown fine Sandy Clay, abundant Iron oxide staining stiff-moist to very moist		18					
10										
	16	3.5			19					
15										
	14	3.0	Light Brown Clayey Silt, abundant Iron oxide staining, stiff-very moist		33					
20										

Boring Terminated at 20'

TBL 20G102-1.GPJ SOCALGEO.GDT 2/6/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 20G102-1
PROJECT: Proposed Industrial Building
LOCATION: Los Angeles, California

DRILLING DATE: 1/23/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					2± inches Asphaltic concrete, 2± inches Aggregate base							
	X	14			FILL: Brown Silty fine Sand, trace fine to coarse Gravel, trace Asphaltic concrete fragments, medium dense-moist to very moist	95	11					
	X	19			@ 3 to 6½ feet, some Clay nodules, some Iron oxide staining	105	12					
5	X	15				99	14					
	X	19			POSSIBLE FILL: Brown Silty fine Sand, mottled, hydrocarbon staining, medium dense-moist to very moist	108	14					
10	X	25			ALLUVIUM: Light Brown Clayey fine Sand, abundant Iron oxide staining, medium dense-moist to very moist	104	15					
15	X	20	4.5		Light Brown fine Sandy Clay, abundant Iron oxide staining, little Charcoal, very stiff-very moist		21					
20	X	16	4.5		Light Gray Brown Clayey Silt, abundant Iron oxide staining, stiff to very stiff-very moist		26					
25	X	11	2.0				41					
					Boring Terminated at 25'							

TBL 20G102-1.GPJ SOCALGEO.GDT 2/6/20

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 20G102-1
PROJECT: Proposed Industrial Building
LOCATION: Los Angeles, California

DRILLING DATE: 1/23/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic concrete, 3± inches Aggregate base							
	X	24			FILL: Light Brown to Dark Brown Silty fine Sand intermixed with Clayey fine Sand, trace fine to coarse Gravel, trace coarse Sand, mottled, medium dense-moist		11					
	X	13			ALLUVIUM: Light Brown to Brown Silty fine Sand, loose to medium dense-damp to very moist		8					
5	X	7					18					
	X	11					17					
10												
	X	18			@ 13½ feet, trace Clay, trace Manganese staining		21					
15												
					Light Brown fine Sand, medium dense-damp							
	X	22					9					
20												
	X	22			@ 23½ feet, trace Clay, little Iron oxide staining		8					
25												

Boring Terminated at 25'

APPENDIX C

LABORATORY TEST RESULTS

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APPENDIX C

GEOTECHNICAL LABORATORY TESTING

The geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

In-Situ Moisture and Density: The natural water content (ASTM D 2216) and in-situ dry density (ASTM D 2937) were determined for recovered relatively undisturbed ring-lined barrel drive samples, from our subsurface explorations. Results of these tests are shown on the logs at the appropriate sample depths, in Appendix B.

Sieve Analysis: Sieve analyses (ASTM D 422) were performed on selected subsurface soil samples. These tests were performed to assist in the classification of the soil. Results of these tests are presented on the "*Particle Size Analysis of Soils*" figures.

Collapse Potential: Collapse potential tests were performed on selected soil samples in general accordance with ASTM Standard Test Method D 5333. Test results are presented on the "*One Dimensional Swell or Settlement*" figure.

Modified Proctor compaction Curve: A laboratory modified Proctor compaction test (ASTM D 1557) was performed on a bulk soil sample to determine maximum laboratory dry density and optimum moisture content. Result of this test is presented on the following "*Modified Proctor Compaction Test*" plot in this appendix.

Corrosivity Tests: To evaluate the corrosion potential of the subsurface soils at the site, we tested representative bulk samples collected during our subsurface investigation for pH, resistivity and soluble sulfate and chloride content testing. Results of these tests are presented at the end of this appendix.

Boring No.	LB-2	LB-2	LB-2	LB-3	LB-5			
Sample No.	R3	S1	R6	R5	R4			
Depth (ft.)	10.0	25.0	30.0	20.0	15			
Sample Type	Ring	SPT	Ring	Ring	Ring			
Soil Identification	Yellowish brown silty, clayey sand (SC-SM)	Olive brown silt (ML)	Olive brown silt with sand (ML)s	Brown silty clay with sand (CL-ML)s	Brown sandy silt s(ML)			
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00			
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00			
Sample Dry Weight Determination								
Weight of Sample + Container (g)	532.08	625.44	773.30	811.80	921.40			
Weight of Container (g)	201.20	288.68	236.70	220.30	248.90			
Weight of Dry Sample (g)	330.88	336.76	536.60	591.50	672.50			
Container No.:								
After Wash								
Method (A or B)	B	A	A	A				
Dry Weight of Sample + Cont. (g)	416.90	335.50	318.80	360.80	566.70			
Weight of Container (g)	201.20	288.68	236.70	220.30	248.90			
Dry Weight of Sample (g)	215.70	46.82	82.10	140.50	317.80			
% Passing No. 200 Sieve	**34.8**	**86.1**	**84.7**	**76.2**	**52.7**			
% Retained No. 200 Sieve	65.2	13.9	15.3	23.8	47.3			

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name:	Bridge/Torrance
Project No.:	12809.001
Client Name:	
Tested By:	SF/GB/YN
Date:	07/21/20

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name:	Bridge/Torrance	Tested By:	S. Felter	Date:	07/21/20
Project No.:	12809.001	Checked By:	A. Santos	Date:	07/27/20
Boring No.:	LB-1	Depth (feet):	2.5		
Sample No.:	R1				
Soil Identification:	Brown silty sand (SM)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	CT	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	863.5	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	243.9	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	619.6	Moisture Content (%)	0.0

After Wet Sieve	Container No.	CT
	Wt. of Dry Soil + Container (g)	692.5
	Wt. of Container (g)	243.9
	Dry Wt. of Soil Retained on # 200 Sieve (g)	448.6

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75		
#8	2.36		
#16	1.18	0.0	100.0
#30	0.600	0.7	99.9
#50	0.300	3.0	99.5
#100	0.150	253.2	59.1
#200	0.075	437.8	29.3
PAN			

GRAVEL: **0 %**

SAND: **71 %**

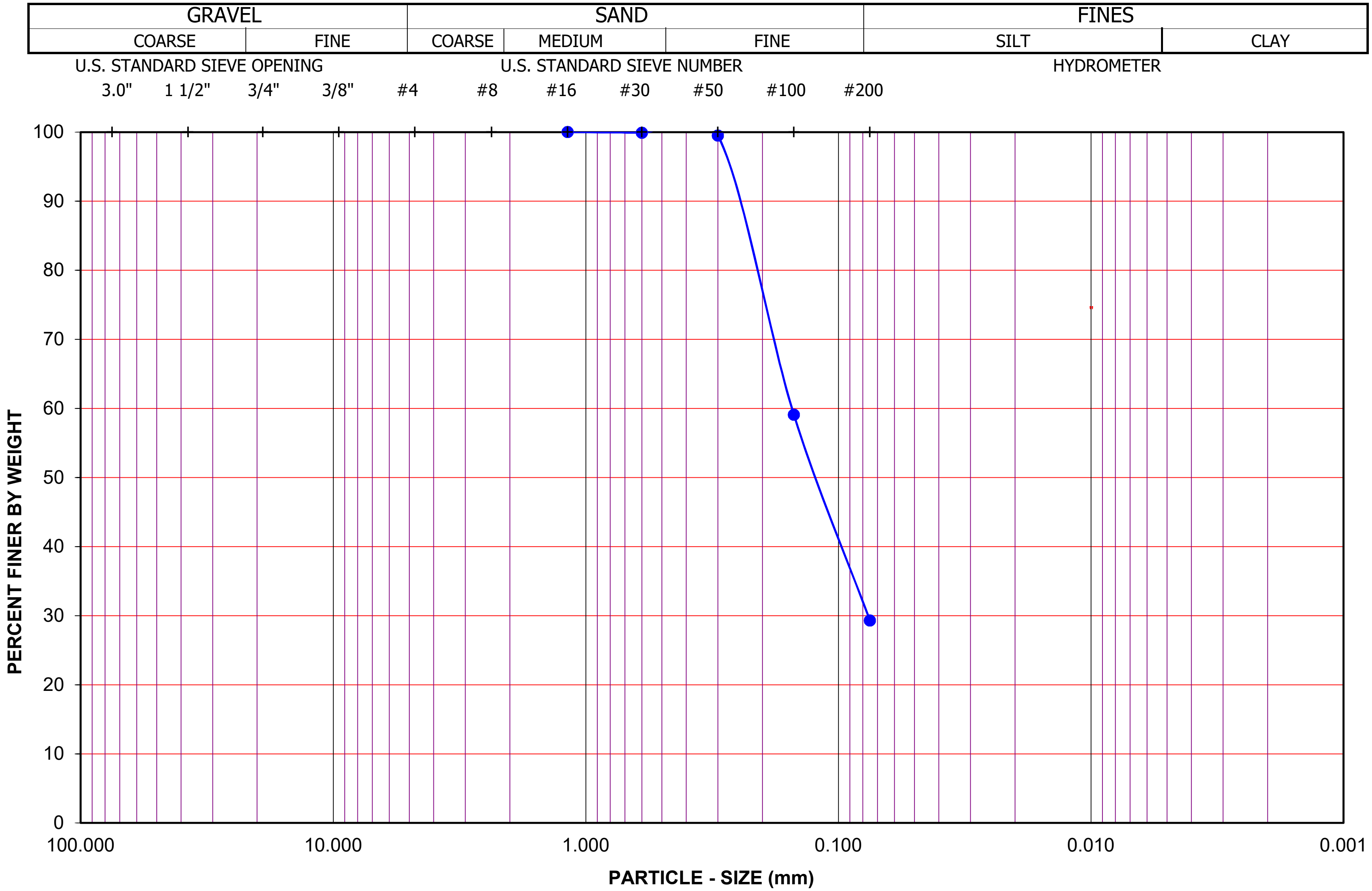
FINES: **29 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = ______

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

Remarks:



Project Name: Bridge/Torrance

Project No.: 12809.001

Boring No.: LB-1

Sample No.: R1

Depth (feet): 2.5

Soil Type : SM

Soil Identification: Brown silty sand (SM)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 71 : 29

Jul-20

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name: Bridge/Torrance
Tested By: S. Felter Date: 07/21/20
Project No.: 12809.001
Checked By: A. Santos Date: 07/27/20
Boring No.: LB-1
Depth (feet): 5.0
Sample No.: R2
Soil Identification: Brown silty sand (SM)

		Moisture Content of Total Air - Dry Soil	
Container No.:	VO	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	945.0	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	234.4	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	710.6	Moisture Content (%)	0.0

After Wet Sieve	Container No.	VO
	Wt. of Dry Soil + Container (g)	737.7
	Wt. of Container (g)	234.4
	Dry Wt. of Soil Retained on # 200 Sieve (g)	503.3

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5		
#4	4.75		
#8	2.36	0.0	100.0
#16	1.18	0.1	100.0
#30	0.600	0.7	99.9
#50	0.300	2.7	99.6
#100	0.150	320.2	54.9
#200	0.075	487.9	31.3
PAN			

GRAVEL: **0 %**

SAND: **69 %**

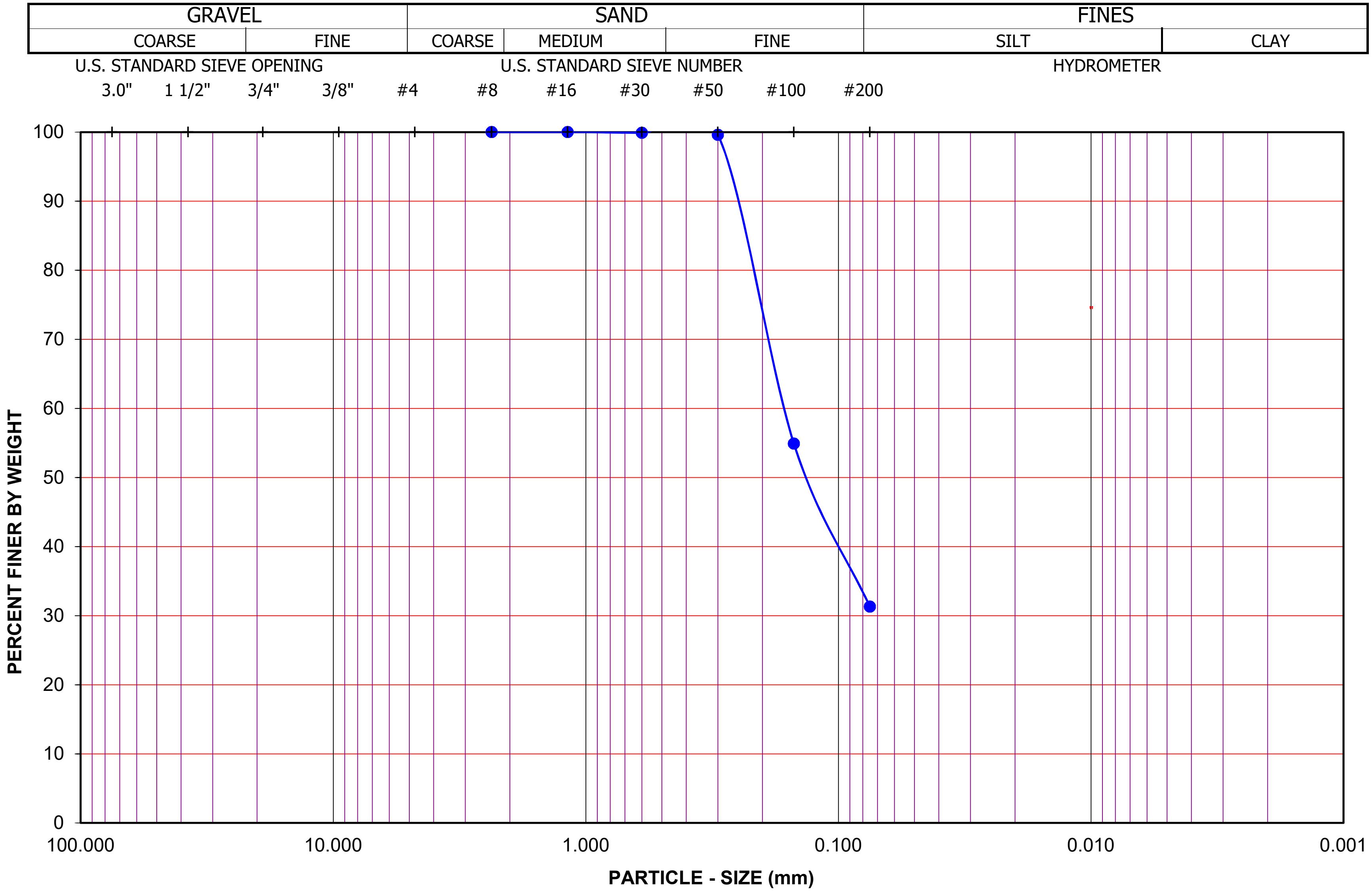
FINES: **31 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = ______

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

Remarks:



Project Name: Bridge/Torrance

Project No.: 12809.001

Boring No.: LB-1 Sample No.: R2

Depth (feet): 5.0 Soil Type : SM

Soil Identification: Brown silty sand (SM)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 69 : 31

Jul-20

Leighton

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

ASTM D 6913

Project Name: Bridge/Torrance
Project No.: 12809.001
Boring No.: LB-2
Sample No.: R1
Soil Identification: Brown silty sand with gravel (SM)g

Tested By: S. Felter Date: 07/21/20
Checked By: A. Santos Date: 07/24/20
Depth (feet): 2.5

		Moisture Content of Total Air - Dry Soil	
Container No.:	YK	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	1060.7	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	251.3	Wt. of Container No._____ (g)	1.0
Dry Wt. of Soil (g)	809.4	Moisture Content (%)	0.0

After Wet Sieve	Container No.	YK
	Wt. of Dry Soil + Container (g)	858.9
	Wt. of Container (g)	251.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	607.6

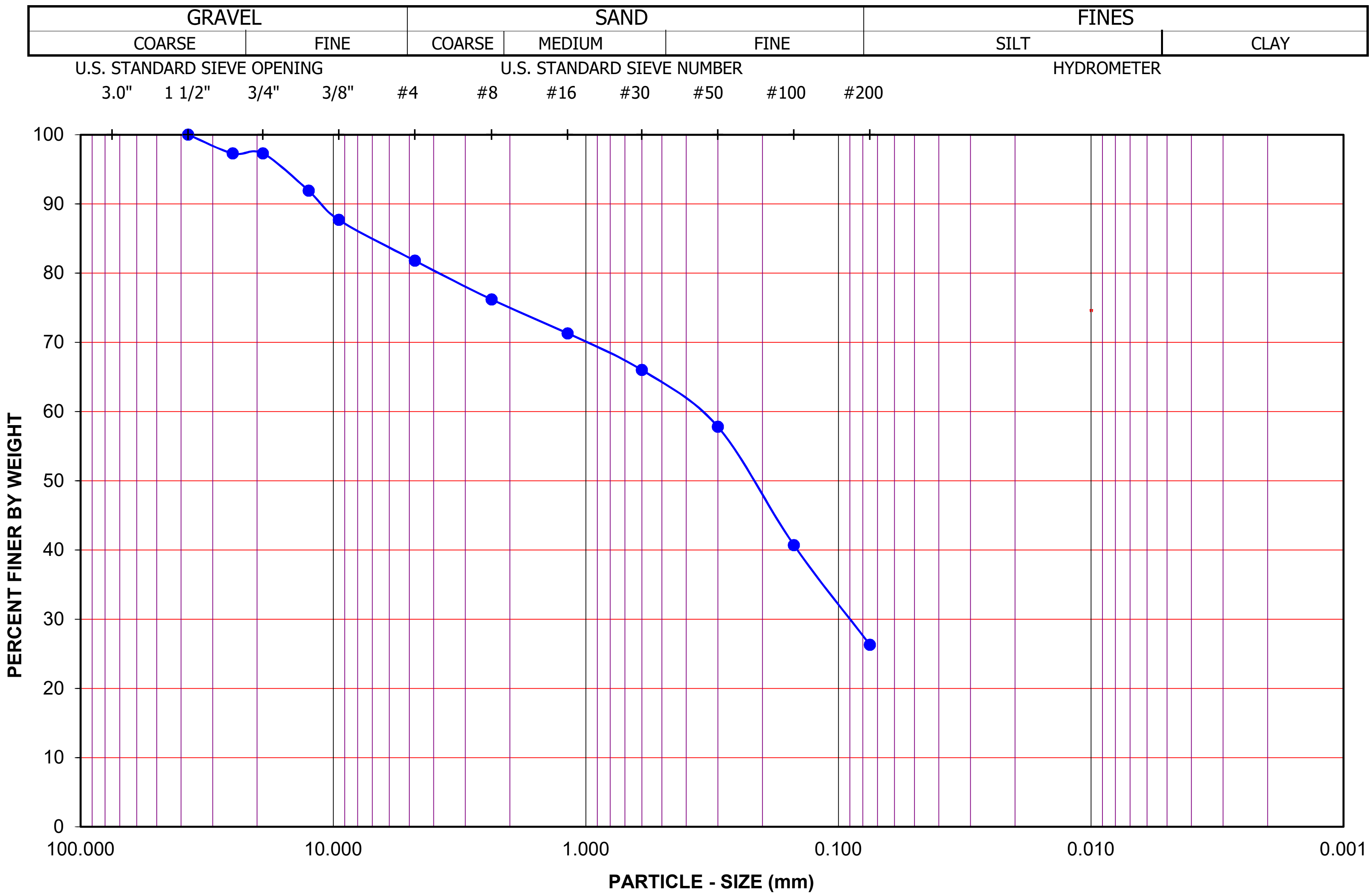
U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5	0.0	100.0
1"	25.0	22.1	97.3
3/4"	19.0	22.1	97.3
1/2"	12.5	65.7	91.9
3/8"	9.5	99.3	87.7
#4	4.75	147.6	81.8
#8	2.36	193.0	76.2
#16	1.18	232.3	71.3
#30	0.600	275.4	66.0
#50	0.300	341.4	57.8
#100	0.150	480.1	40.7
#200	0.075	596.4	26.3
PAN			

GRAVEL: **18 %**
SAND: **56 %**
FINES: **26 %**
GROUP SYMBOL: **(SM)g**

$Cu = D60/D10 =$ ______

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

Remarks:



Project Name: Bridge/Torrance

Project No.: 12809.001

Boring No.: LB-2 Sample No.: R1

Depth (feet): 2.5 Soil Type : (SM)g

Soil Identification: Brown silty sand with gravel (SM)g

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 18 : 56 : 26

Jul-20

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Leighton

Project Name:	Bridge/Torrance	Tested By:	G. Bathala	Date:	07/23/20
Project No.:	12809.001	Checked By:	A. Santos	Date:	07/27/20
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R3	Depth (ft.)	10.0		

Sample Description: Yellowish brown silty, clayey sand (SC-SM)

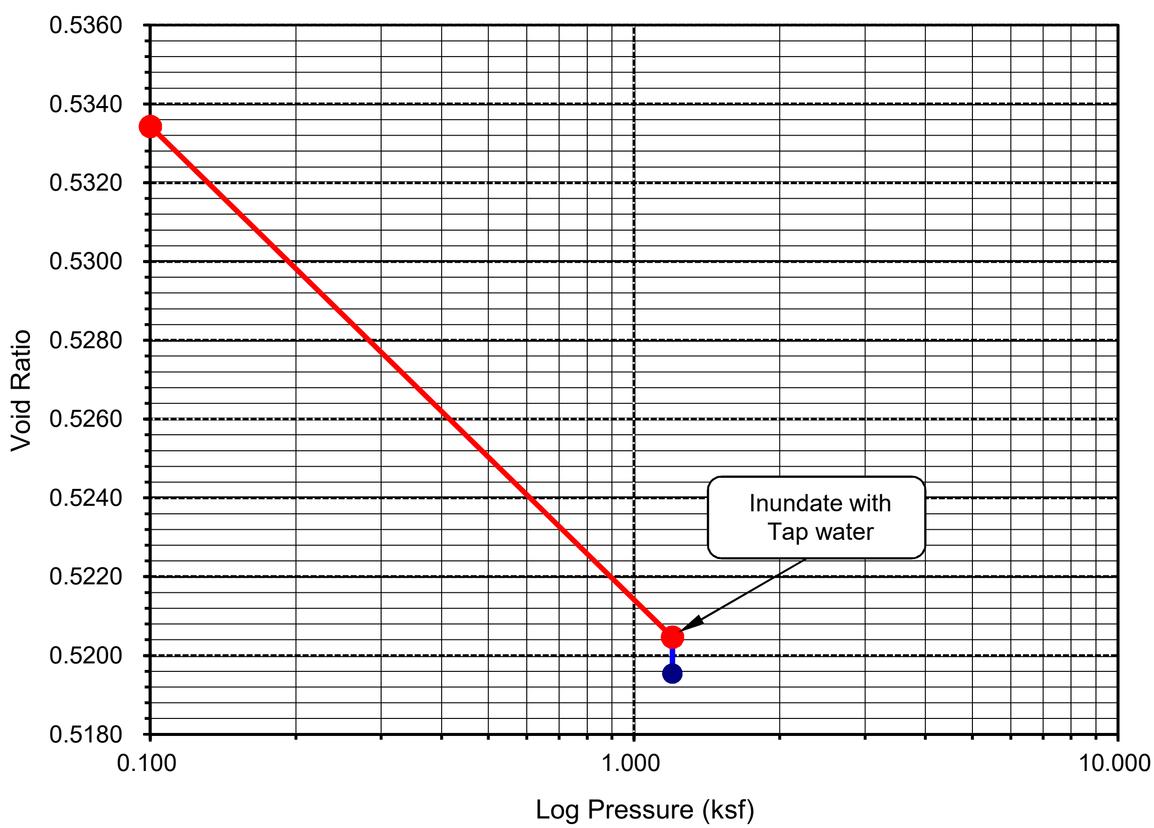
Initial Dry Density (pcf):	109.8
Initial Moisture (%):	18.46
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3295
Diameter(in):	2.415

Final Dry Density (pcf):	110.9
Final Moisture (%) :	19.2
Initial Void Ratio:	0.5347
Specific Gravity(assumed):	2.70
Initial Saturation (%)	93.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3287	0.9992	0.00	-0.09	0.5334	-0.09
1.200	0.3189	0.9894	0.13	-1.06	0.5205	-0.93
H2O	0.3183	0.9888	0.13	-1.12	0.5195	-0.99

Percent Swell (+) / Settlement (-) After Inundation = -0.06

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Leighton

Project Name: Bridge/Torrance
Project No.: 12809.001
Boring No.: LB-5
Sample No.: R5
Sample Description: Light olive brown lean clay (CL)

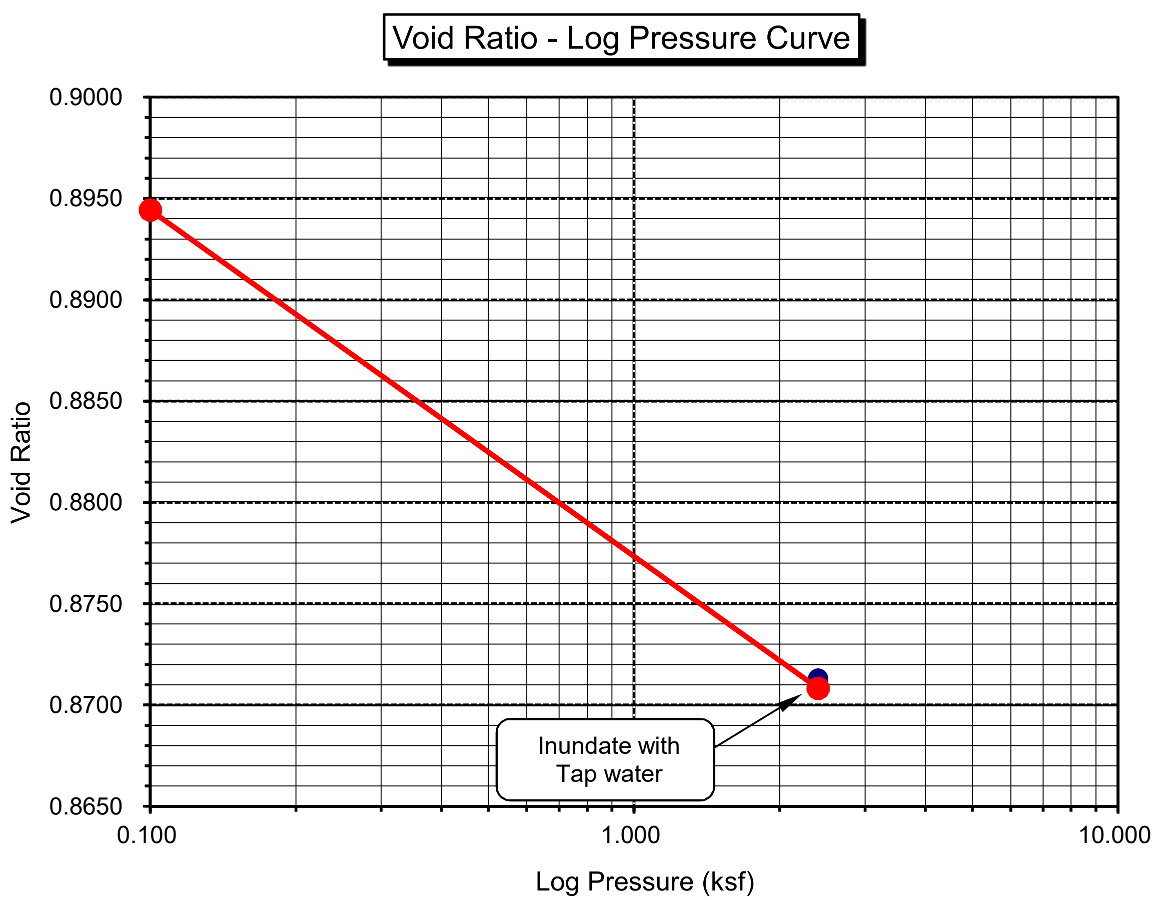
Tested By: G. Bathala Date: 07/23/20
Checked By: A. Santos Date: 07/27/20
Sample Type: Ring
Depth (ft.) 20.0

Initial Dry Density (pcf):	88.9
Initial Moisture (%):	33.83
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2817
Diameter(in):	2.415

Final Dry Density (pcf):	90.1
Final Moisture (%) :	33.6
Initial Void Ratio:	0.8960
Specific Gravity(assumed):	2.70
Initial Saturation (%)	102.0

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2809	0.9992	0.00	-0.08	0.8944	-0.08
2.400	0.2649	0.9832	0.36	-1.69	0.8708	-1.33
H2O	0.2651	0.9834	0.36	-1.66	0.8713	-1.30

Percent Swell (+) / Settlement (-) After Inundation = 0.03



Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Bridge/Torrance	Tested By :	G. Berdy	Date:	07/22/20
Project No. :	12809.001	Checked By:	A. Santos	Date:	07/27/20

Boring No.	LB-5			
Sample No.	B1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SM)			
Wet Weight of Soil + Container (g)	194.66			
Dry Weight of Soil + Container (g)	194.08			
Weight of Container (g)	154.33			
Moisture Content (%)	1.46			
Weight of Soaked Soil (g)	100.10			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	15			
Crucible No.	16			
Furnace Temperature (°C)	860			
Time In / Time Out	11:00/11:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.4688			
Wt. of Crucible (g)	18.4651			
Wt. of Residue (g) (A)	0.0037			
PPM of Sulfate (A) x 41150	152.26			
PPM of Sulfate, Dry Weight Basis	**155**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	80			
PPM of Chloride, Dry Wt. Basis	**81**			

pH TEST, DOT California Test 643

pH Value	**7.89**			
Temperature °C	20.9			

SOIL RESISTIVITY TEST
DOT CA TEST 643

Leighton

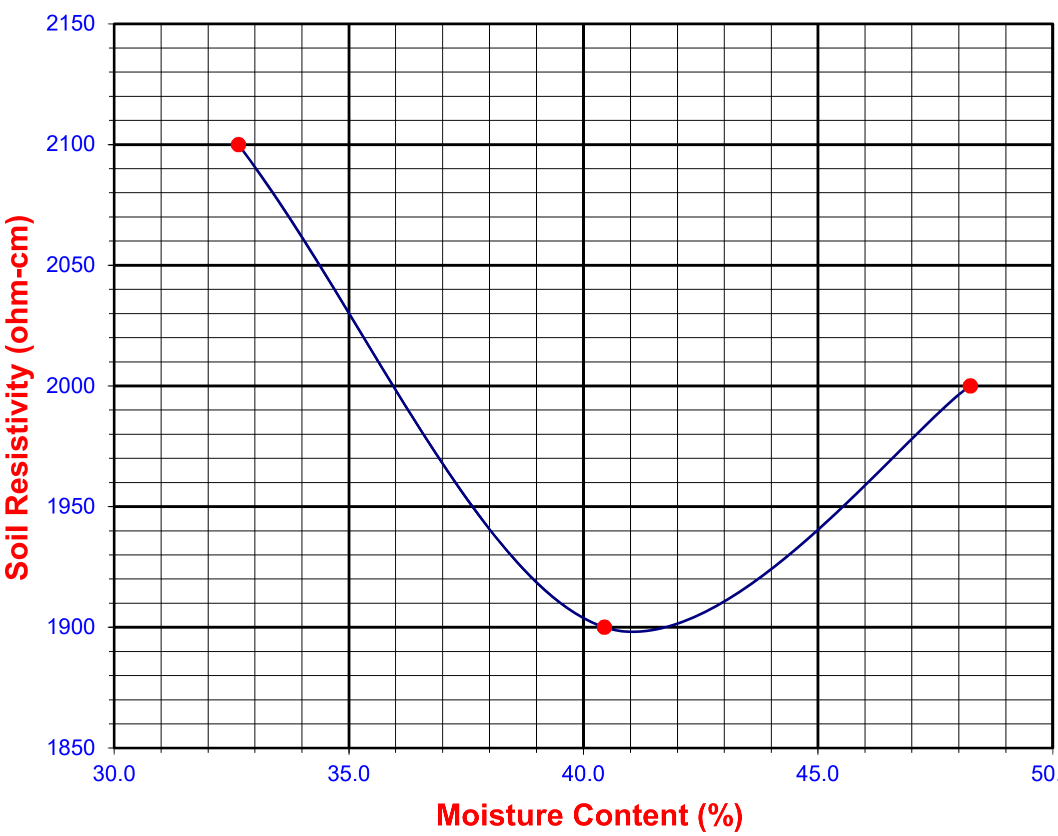
Project Name:	Bridge/Torrance		Tested By :	G. Berdy	Date: 07/23/20
Project No. :	12809.001	###	Checked By:	A. Santos	Date: 07/27/20
Boring No.:	LB-5		Depth (ft.) :	0-5	
Sample No. :	B1				
Soil Identification:*	Olive brown (SM)				

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	40	32.65	2100	2100
2	50	40.45	1900	1900
3	60	48.24	2000	2000
4				
5				

Moisture Content (%) (MCi)	1.46
Wet Wt. of Soil + Cont. (g)	194.66
Dry Wt. of Soil + Cont. (g)	194.08
Wt. of Container (g)	154.33
Container No.	
Initial Soil Wt. (g) (Wt)	130.12
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1898	**41.1**	**155**	**81**	**7.89**	**20.9**



EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	Bridge/Torrance	Tested By:	A. Santos	Date:	07/27/20
Project No.:	12809.001	Checked By:	G. Bathala	Date:	07/28/20
Boring No.:	LB-5	Depth (in.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand (SM)				

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

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0010
Wt. Comp. Soil + Mold	(g)	615.60	429.20
Wt. of Mold	(g)	201.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	819.60	630.50
Dry Wt. of Soil + Cont.	(g)	751.90	581.04
Wt. of Container	(g)	0.00	201.30
Moisture Content	(%)	9.00	13.02
Wet Density	(pcf)	125.0	129.3
Dry Density	(pcf)	114.6	114.4
Void Ratio		0.470	0.473
Total Porosity		0.320	0.321
Pore Volume	(cc)	66.2	66.6
Degree of Saturation (%) [S_{meas}]		**51.7**	74.3

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.)
07/27/20	14:00	1.0	0	0.6060
07/27/20	14:10	1.0	10	0.6060
Add Distilled Water to the Specimen				
07/27/20	14:15	1.0	5	0.6060
07/28/20	6:30	1.0	980	0.6070
07/28/20	8:30	1.0	1100	0.6070

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**1**

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Bridge/Torrance	Tested By:	O. Figueroa	Date:	07/23/20
Project No.:	12809.001	Input By:	A. Santos	Date:	07/27/20
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B1				
Soil Identification:	Dark brown silty sand (SM)				

Preparation Method: [X] Moist [ ] Dry — [X] 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)	3801	3895	3844			
Weight of Mold (g)	1862	1862	1862			
Net Weight of Soil (g)	1939	2033	1982			
Wet Weight of Soil + Cont. (g)	452.9	449.6	470.9			
Dry Weight of Soil + Cont. (g)	426.2	414.5	425.1			
Weight of Container (g)	39.1	40.4	41.0			
Moisture Content (%)	6.90	9.38	11.92			
Wet Density (pcf)	128.8	135.0	131.6			
Dry Density (pcf)	120.4	123.4	117.6			

Maximum Dry Density (pcf) **123.5** **Optimum Moisture Content (%)** **9.0**

PROCEDURE USED

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

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

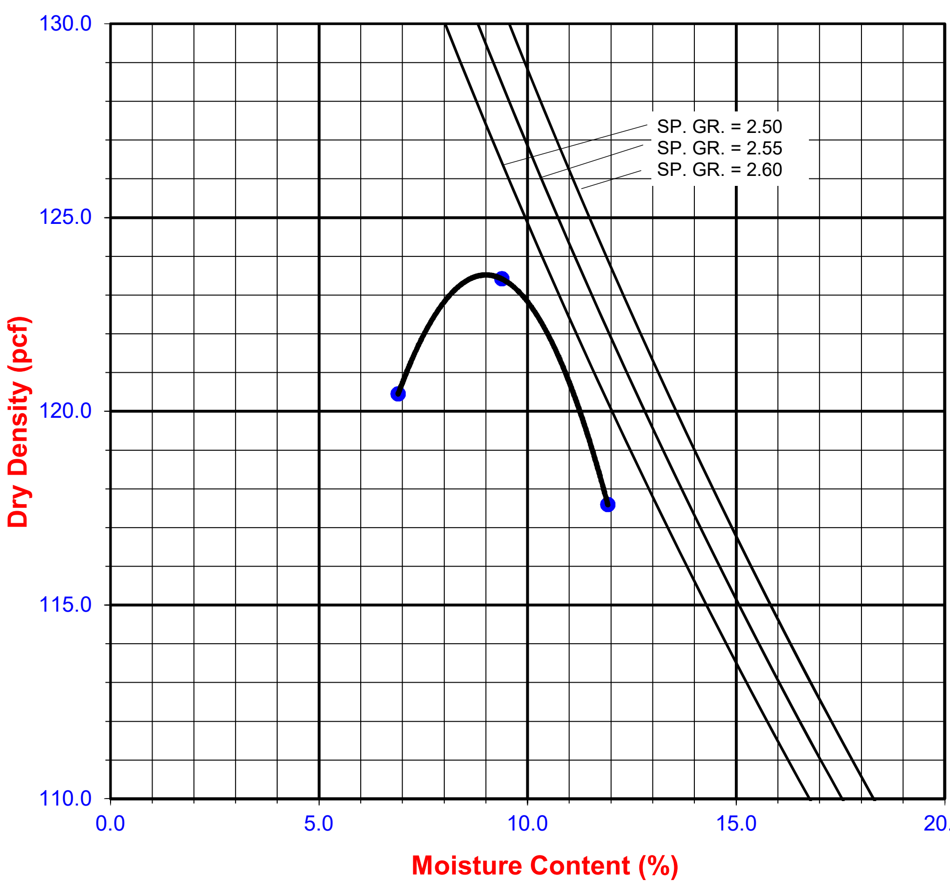
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

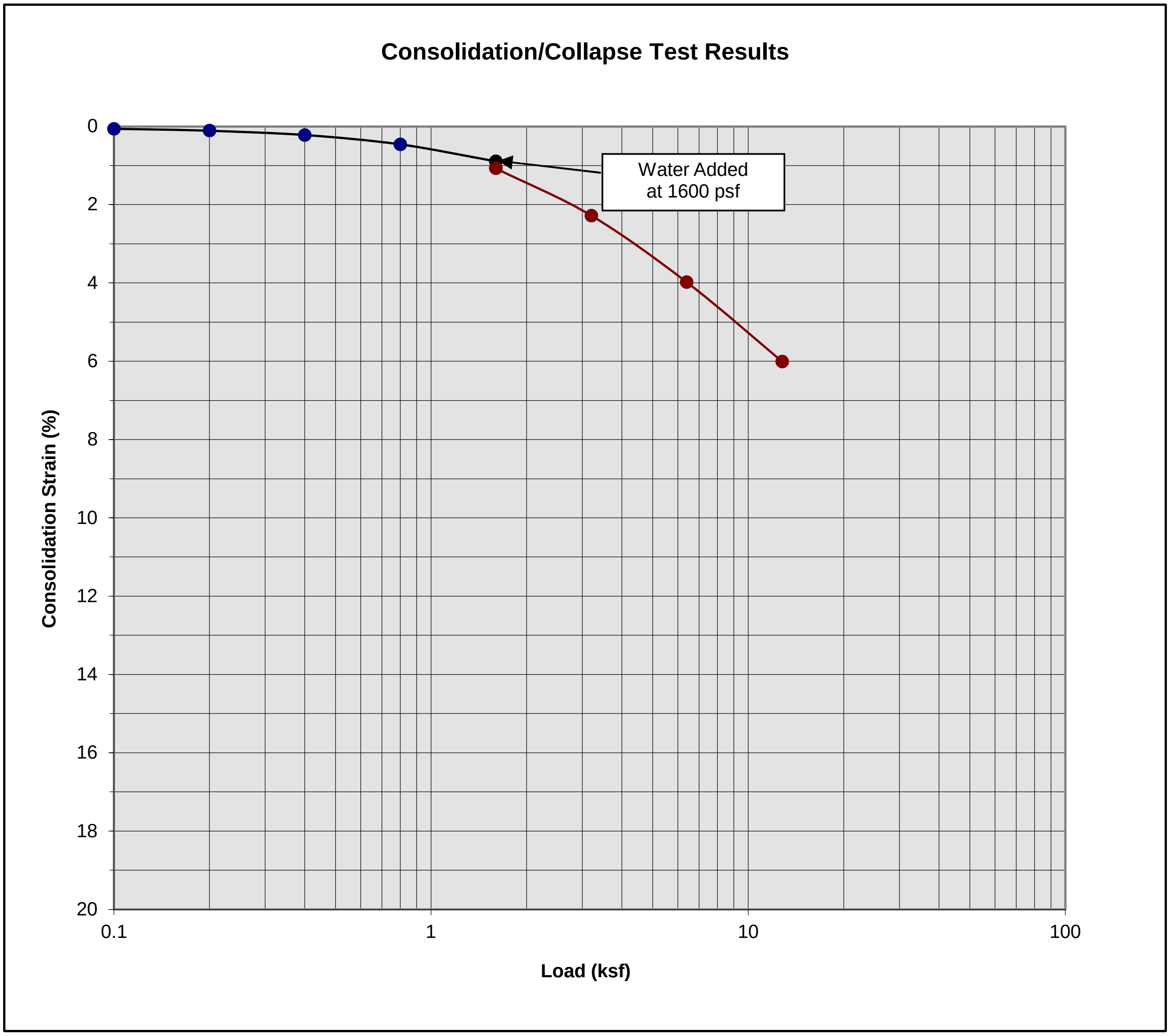
Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI





Classification: FILL: Dark Brown to Black Clayey fine to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	24
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	3 to 4	Initial Dry Density (pcf)	96.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.18

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 1

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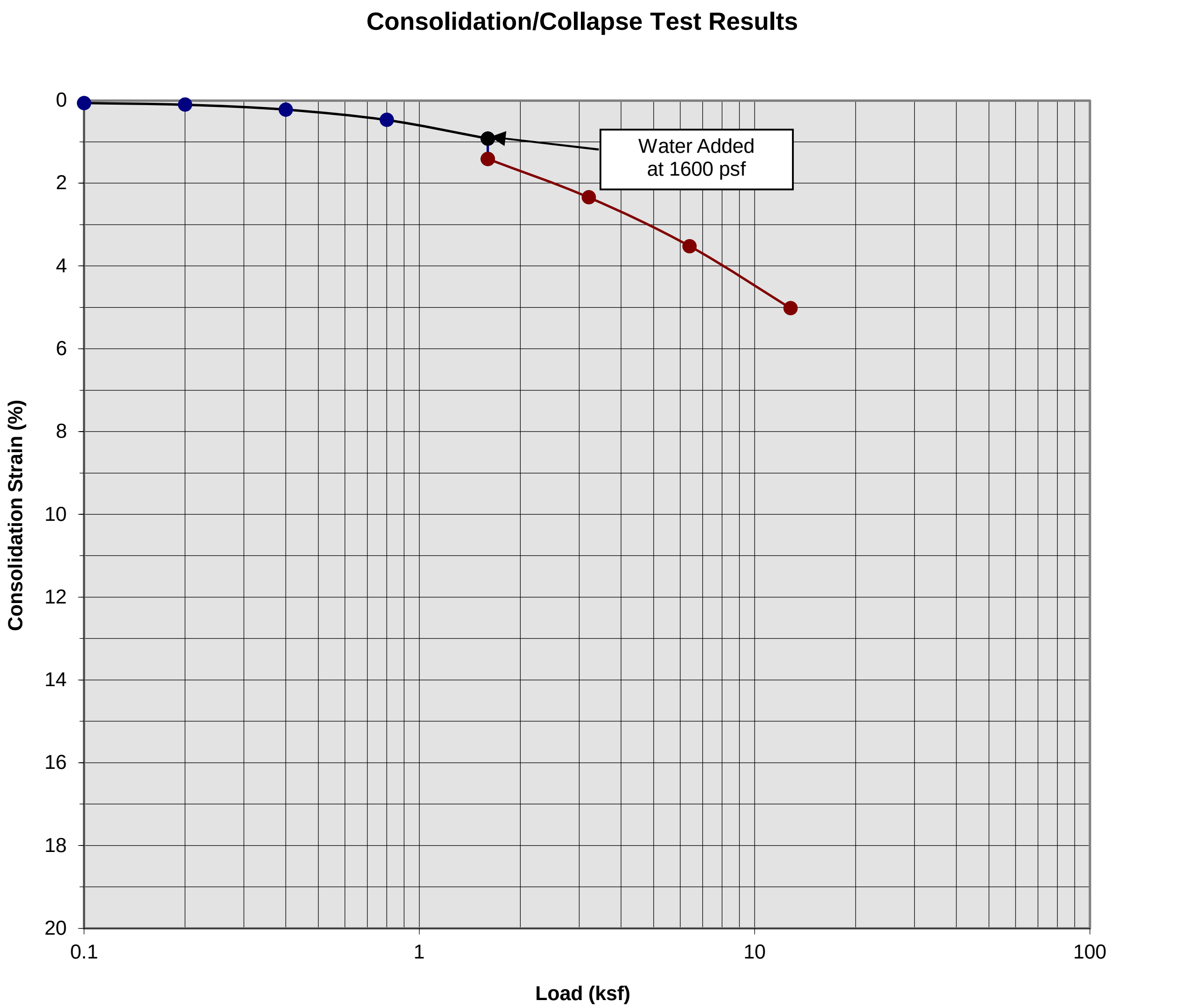

Classification: ALLUVIUM: Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	93.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.49

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 2

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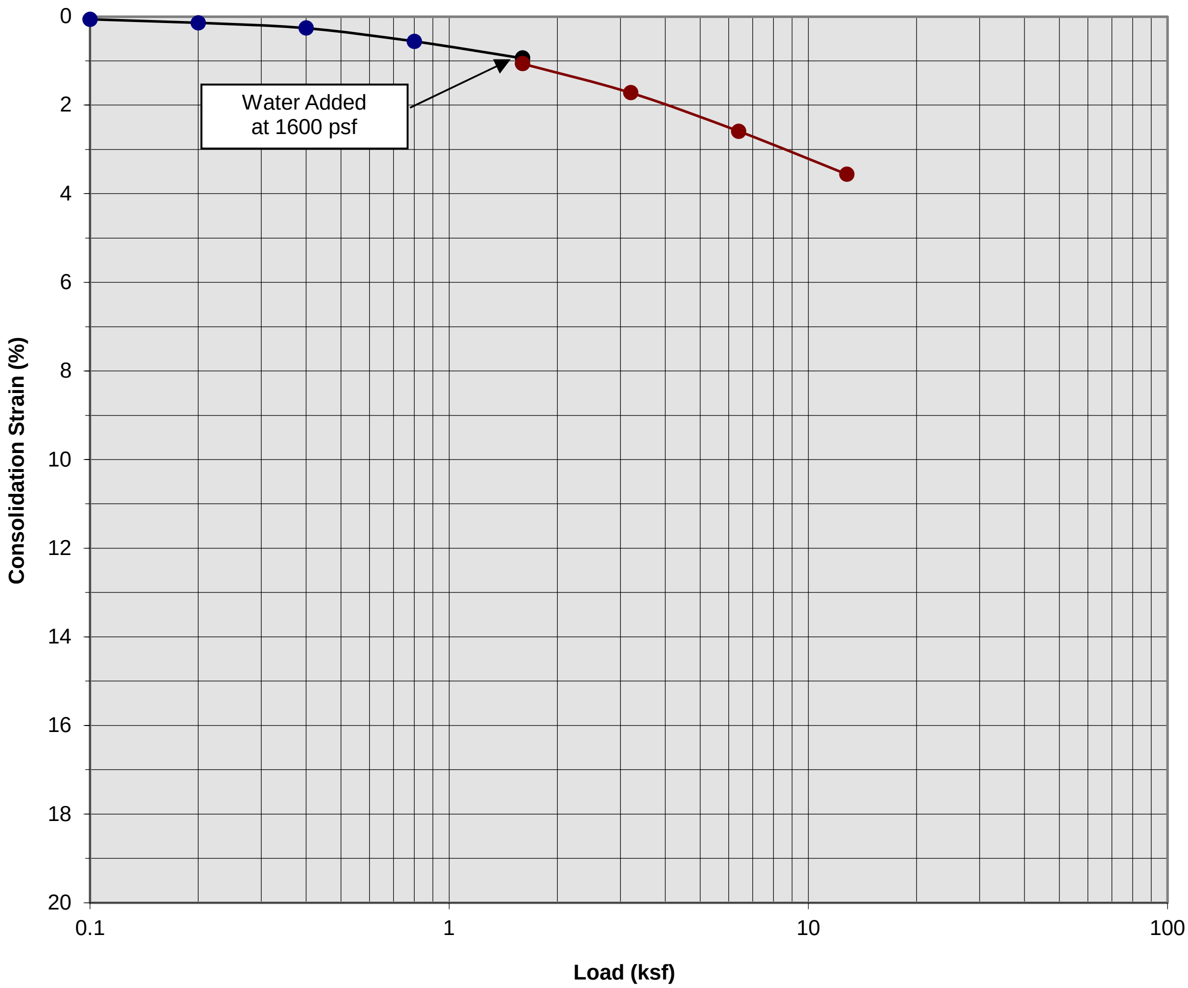
Consolidation/Collapse Test Results



Classification: ALLUVIUM: Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	7 to 8	Initial Dry Density (pcf)	103.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.12

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 3

SoCalGeo
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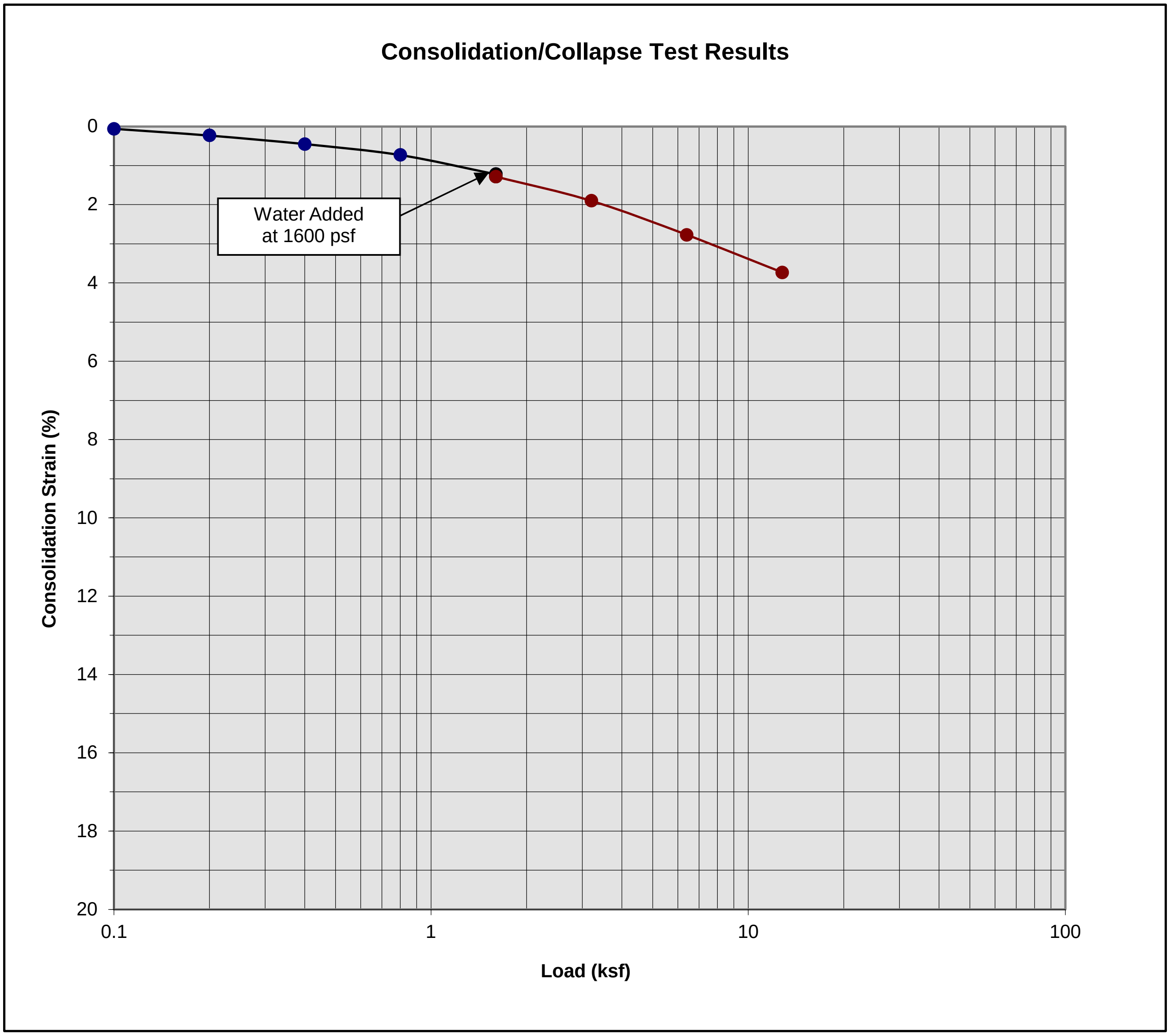

Classification: ALLUVIUM: Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	9 to 10	Initial Dry Density (pcf)	107.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.06

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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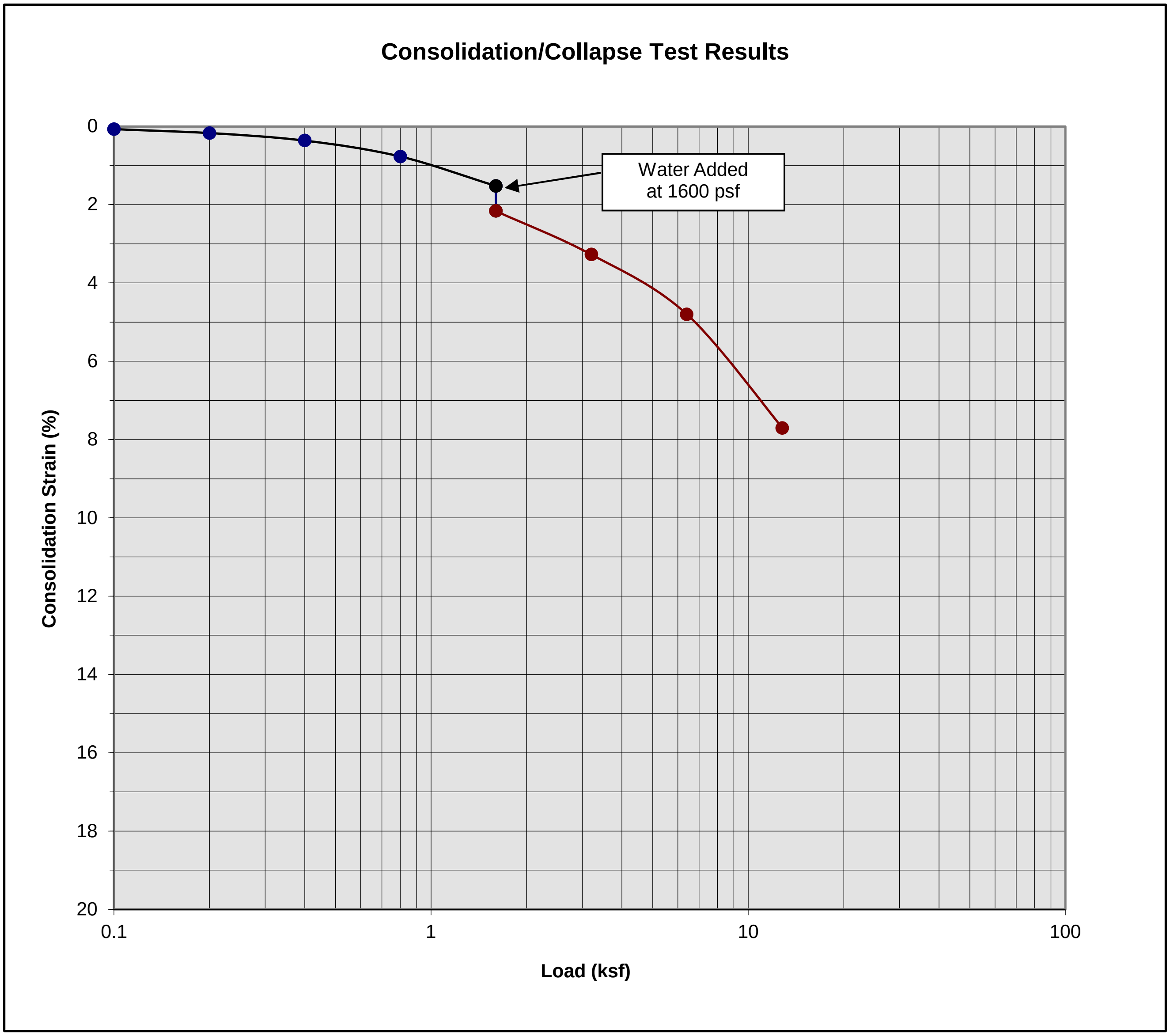

Classification: FILL: Brown Silty fine Sand, some Clay nodules

Boring Number:	B-5	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.64

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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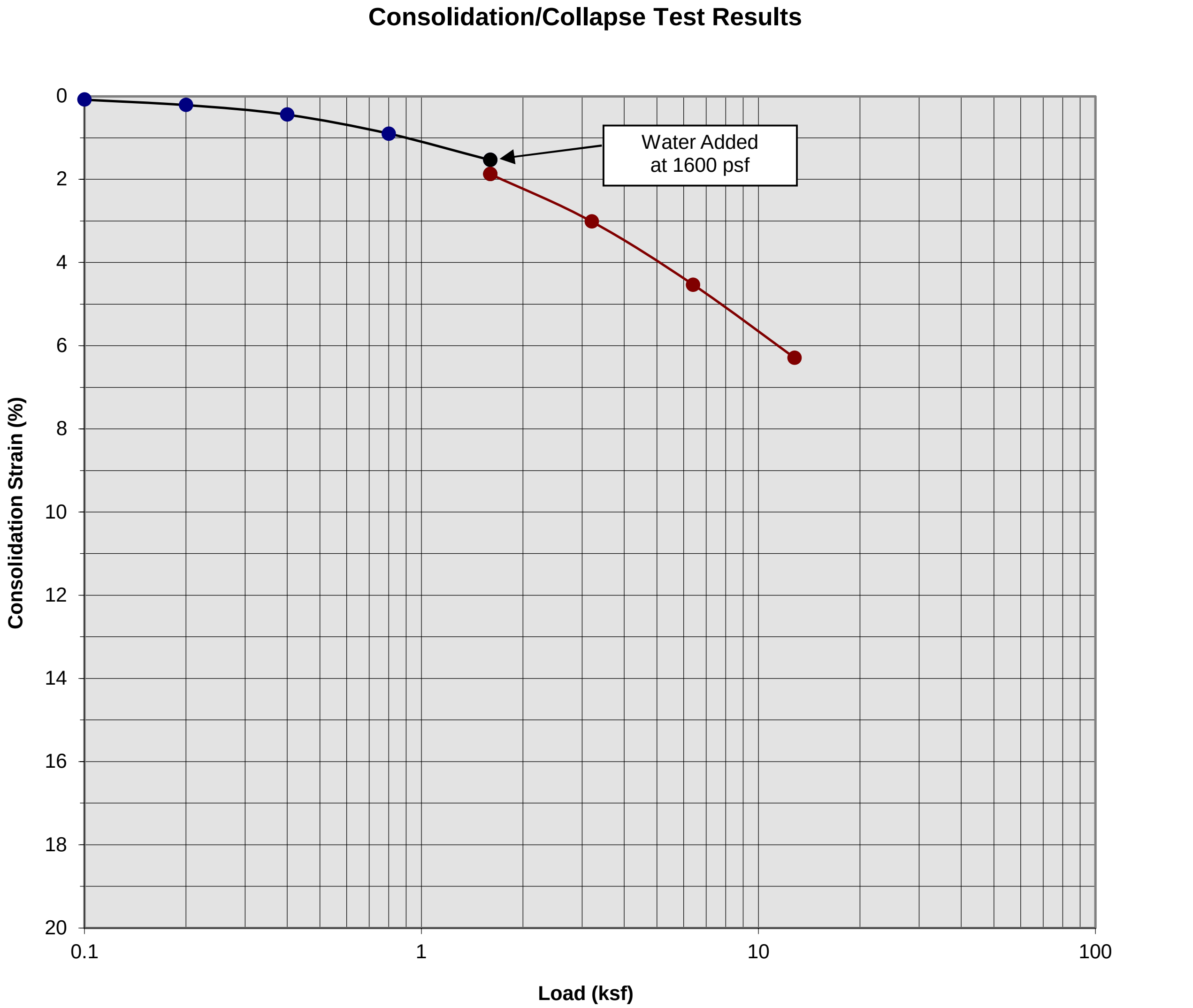

Classification: FILL: Brown Silty fine Sand, some Clay nodules

Boring Number:	B-5	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	100.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.34

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
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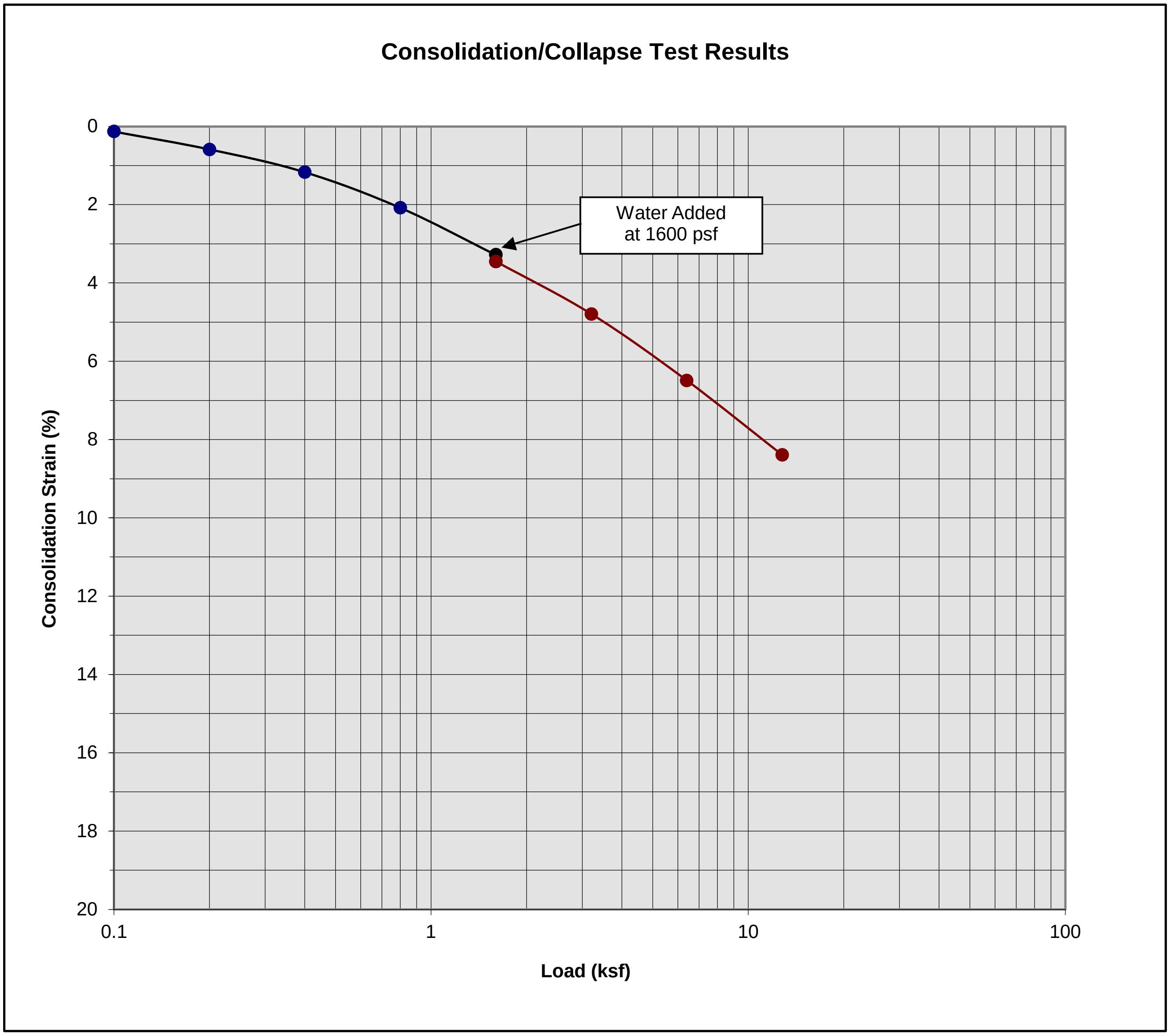
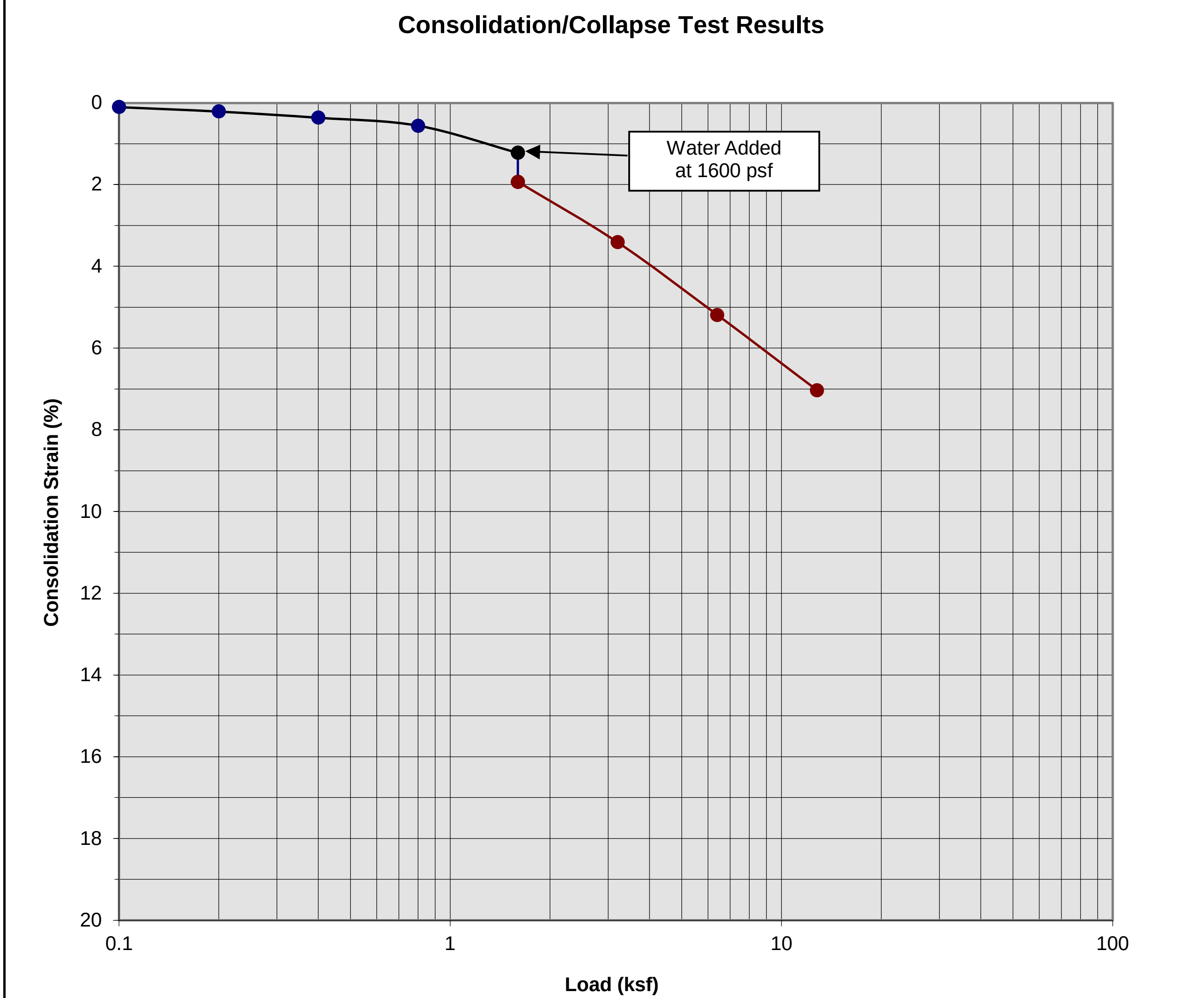


Classification: POSSIBLE FILL: Brown Silty fine Sand

Boring Number:	B-5	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	106.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.17

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: ALLUVIUM: Light Brown Clayey fine Sand

Boring Number:	B-5	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	9 to 10	Initial Dry Density (pcf)	104.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.71

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 8

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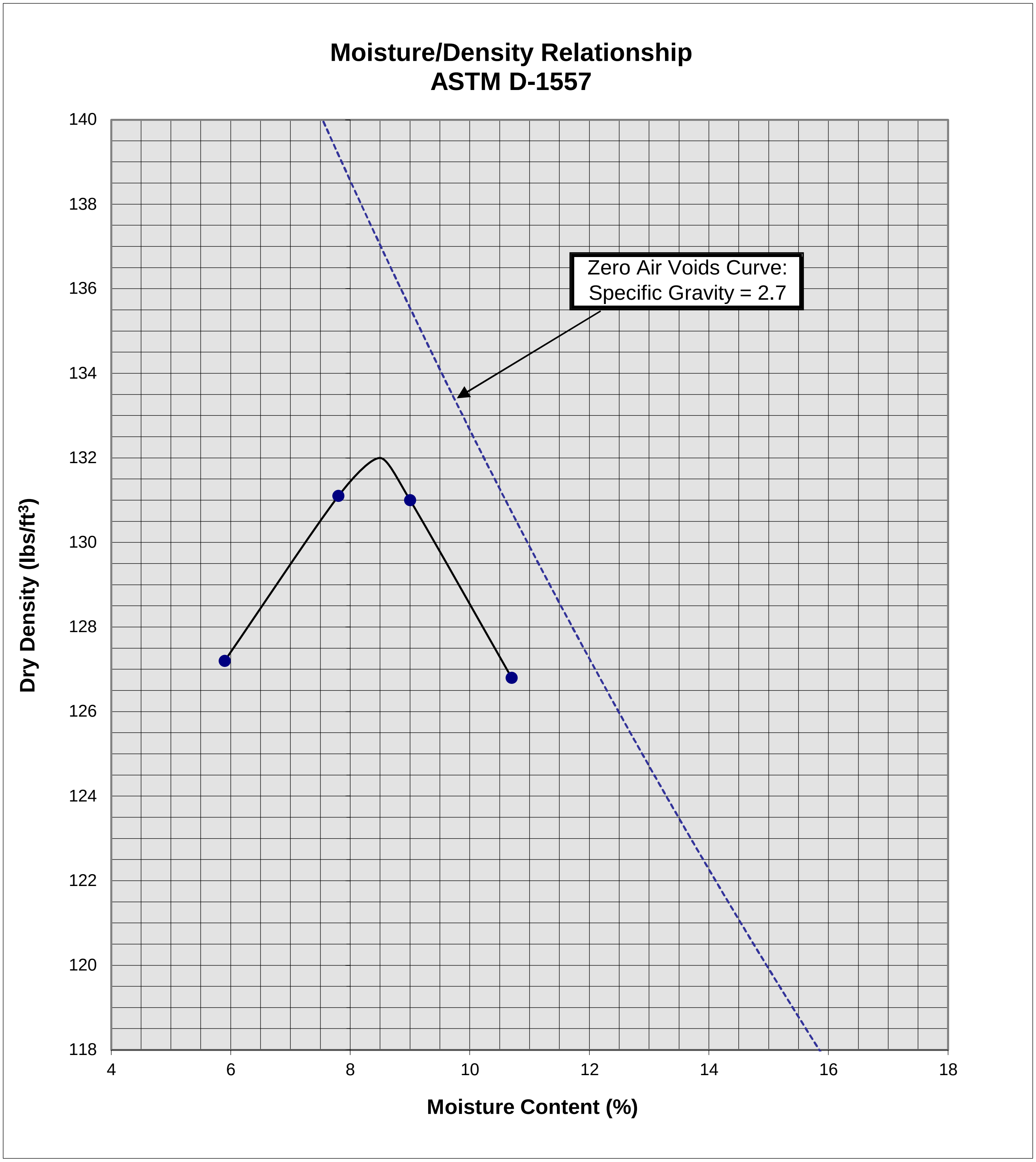

Soil ID Number		B-3 @ 0-5'
Optimum Moisture (%)		8.5
Maximum Dry Density (pcf)		132
Soil Classification	Brown Silty fine Sand trace fine Gravel, little Clay	

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 9

SOUTHERN CALIFORNIA GEOTECHNICAL
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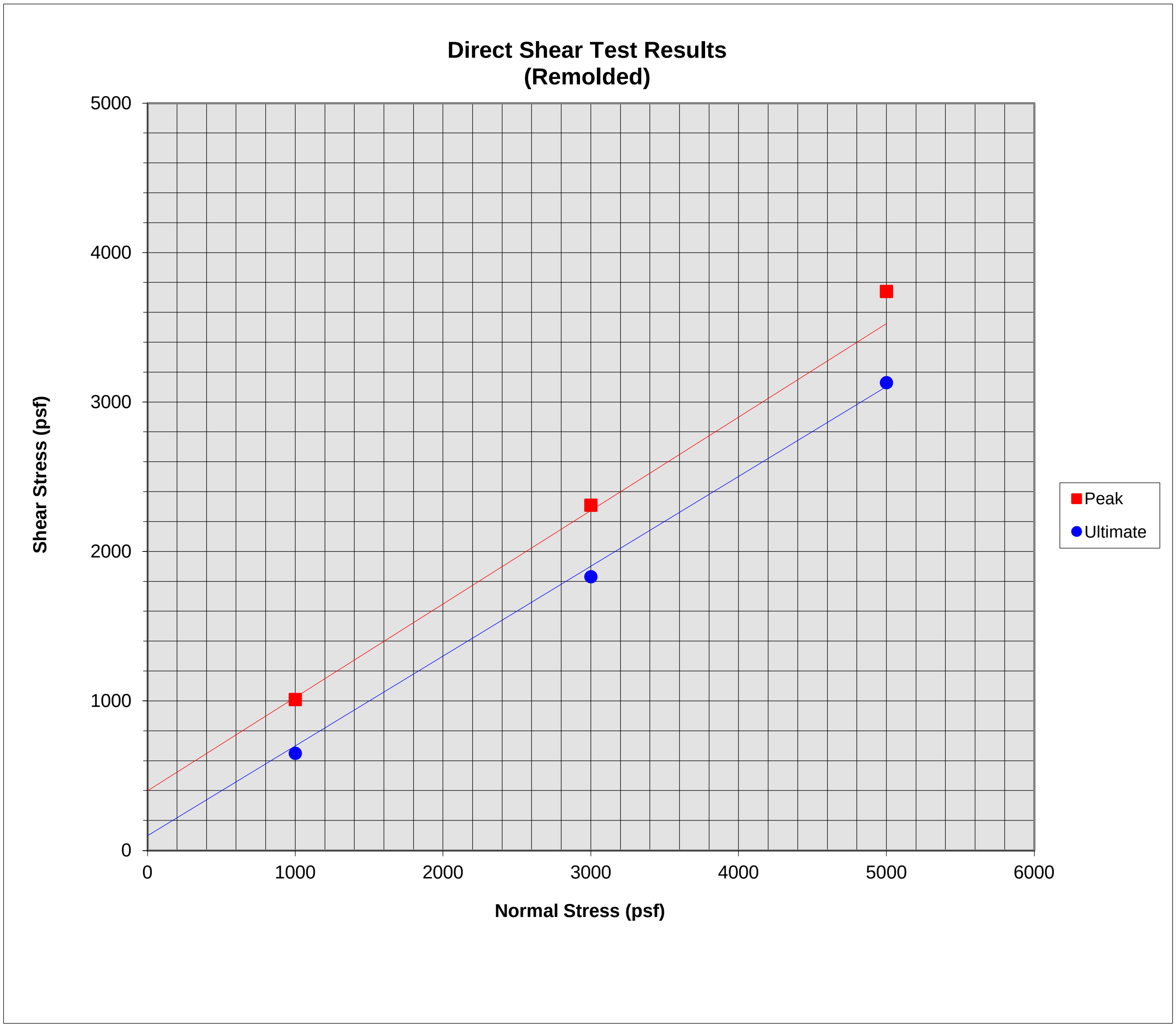

Sample Description: B-3 @ 0 to 5'

Classification: Brown Silty fine Sand, little Clay

Sample Data		Test Results		
Remolded Moisture Content	8.5			
Final Moisture Content	---		Peak	Ultimate
Remolded Dry Density	118.8	ϕ (°)	32.0	31.0
Percent Compaction	90.0	C (psf)	400	100
Final Dry Density	---			
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Proposed Industrial Building
Los Angeles, California
Project No. 20G102-1
PLATE C- 10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

OSHPD

Latitude, Longitude: 33.8154, -118.3018

Date	7/27/2020, 11:55:22 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.76	MCE_R ground motion. (for 0.2 second period)
S_1	0.639	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.76	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.173	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.777	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.854	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.76	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.969	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.125	Factored deterministic acceleration value. (0.2 second)
S1RT	0.639	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.717	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.767	Factored deterministic acceleration value. (1.0 second)
PGAd	0.874	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.894	Mapped value of the risk coefficient at short periods
C_{R1}	0.892	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.8154

Time Horizon

Return period in years

2475

Longitude

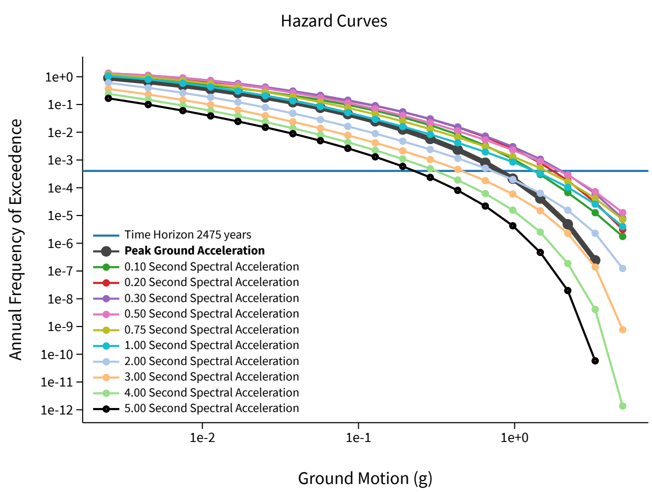
Decimal degrees, negative values for western longitudes

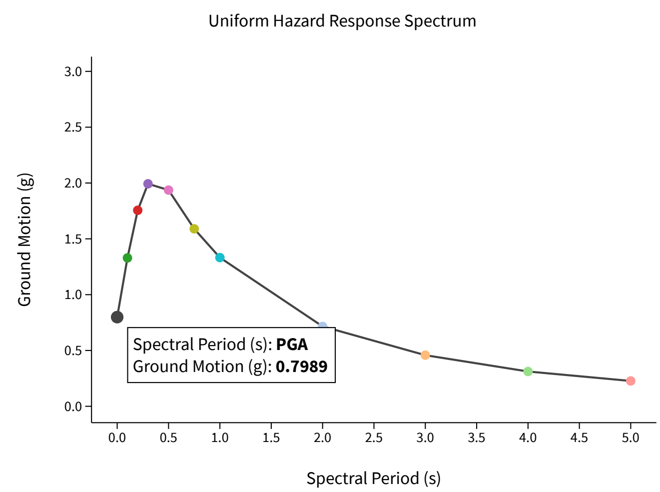
-118.3018

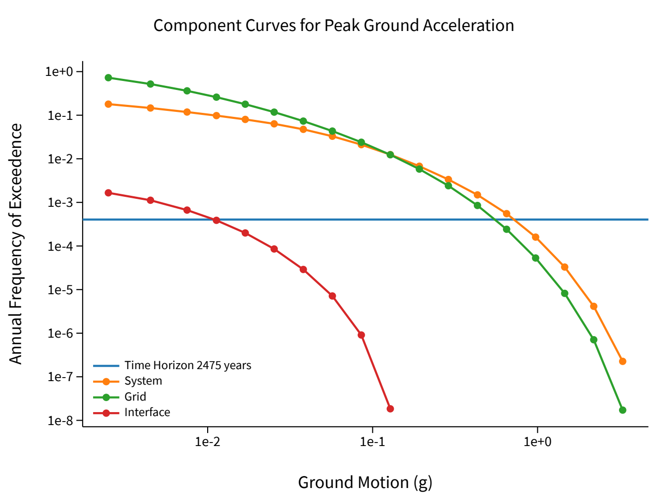
Site Class

259 m/s (Site class D)

Hazard Curve





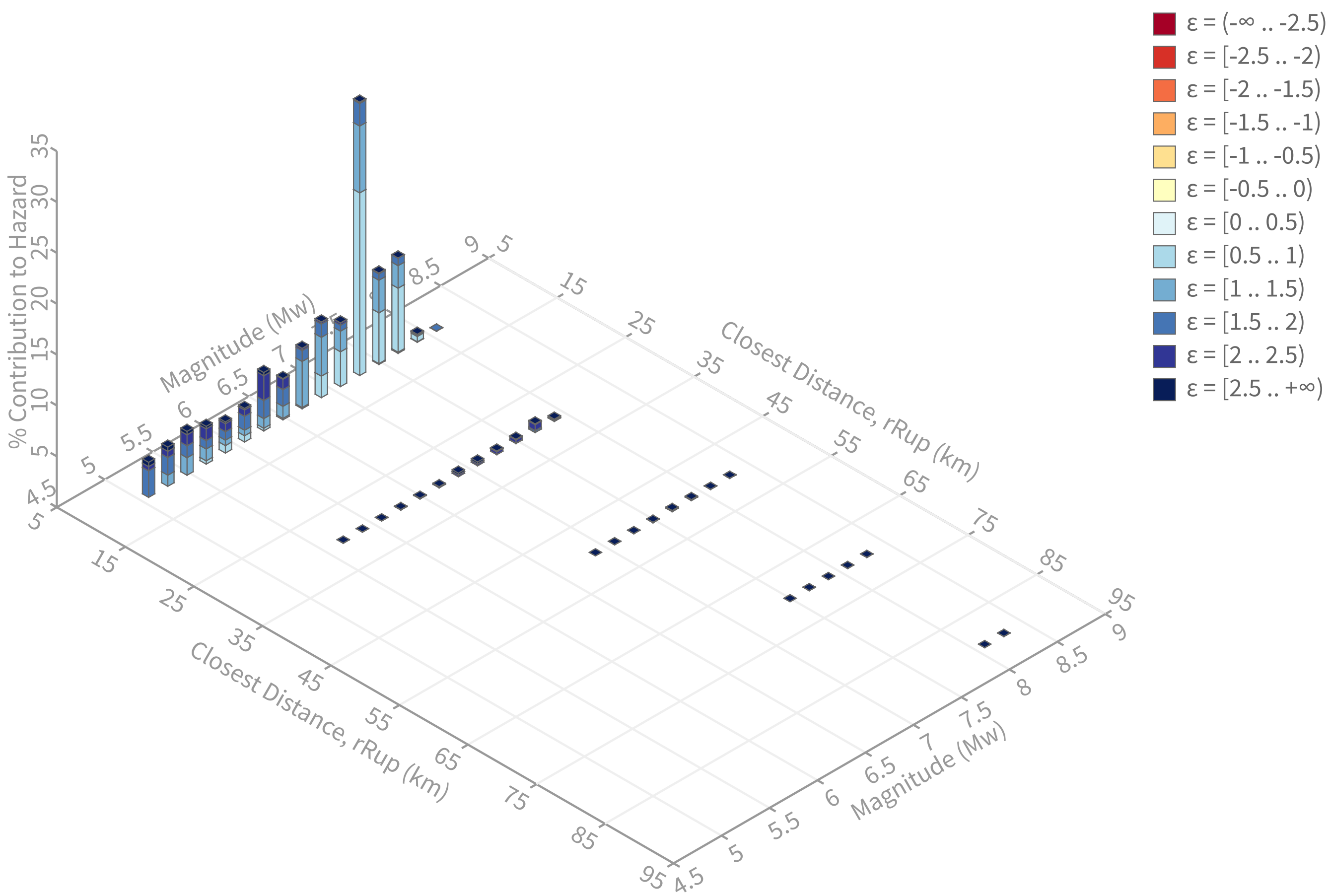


[View Raw Data]

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.79888368 g

Recovered targets

Return period: 2895.7534 yrs
Exceedance rate: 0.00034533328 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.07 %

Mean (over all sources)

m: 6.82
r: 7.07 km
ε_0: 1.32 σ

Mode (largest m-r bin)

m: 7.3
r: 5.01 km
ε_0: 1.01 σ
Contribution: 26.86 %

Mode (largest m-r-ε_0 bin)

m: 7.3
r: 4.83 km
ε_0: 0.85 σ
Contribution: 17.95 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							39.05
Palos Verdes [11]		4.26	7.20	1.06	118.322°W	33.783°N	207.96	18.21
Compton [2]		6.30	7.40	0.85	118.295°W	33.821°N	43.43	9.02
Newport-Inglewood alt 1 [5]		7.32	7.29	1.31	118.238°W	33.852°N	55.15	3.23
Compton [1]		6.42	6.99	0.98	118.286°W	33.817°N	82.69	1.42
Palos Verdes [12]		5.68	6.62	1.41	118.359°W	33.800°N	251.82	1.37
Redondo Canyon alt 1 [1]		8.71	6.25	1.92	118.395°W	33.812°N	267.43	1.16
UC33brAvg_FM32	System							34.67
Palos Verdes [11]		4.26	7.34	1.03	118.322°W	33.783°N	207.96	15.83
Compton [2]		6.30	7.40	0.85	118.295°W	33.821°N	43.43	9.05
Newport-Inglewood alt 2 [5]		8.14	7.28	1.40	118.244°W	33.869°N	41.82	2.14
Palos Verdes [12]		5.68	6.62	1.41	118.359°W	33.800°N	251.82	1.84
Compton [1]		6.42	6.97	0.98	118.286°W	33.817°N	82.69	1.19
UC33brAvg_FM31 (opt)	Grid							13.32
PointSourceFinite: -118.302, 33.829		5.30	5.58	1.42	118.302°W	33.829°N	0.00	3.78
PointSourceFinite: -118.302, 33.829		5.30	5.58	1.42	118.302°W	33.829°N	0.00	3.78
PointSourceFinite: -118.302, 33.901		9.92	5.85	2.02	118.302°W	33.901°N	0.00	1.96
PointSourceFinite: -118.302, 33.901		9.92	5.85	2.02	118.302°W	33.901°N	0.00	1.96
UC33brAvg_FM32 (opt)	Grid							12.96
PointSourceFinite: -118.302, 33.829		5.30	5.58	1.43	118.302°W	33.829°N	0.00	3.69
PointSourceFinite: -118.302, 33.829		5.30	5.58	1.43	118.302°W	33.829°N	0.00	3.69
PointSourceFinite: -118.302, 33.901		9.94	5.85	2.03	118.302°W	33.901°N	0.00	1.89
PointSourceFinite: -118.302, 33.901		9.94	5.85	2.03	118.302°W	33.901°N	0.00	1.89

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*                    *
*    E Q F A U L T   *
*                    *
*    Version 3.00    *
*                    *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 9842-0000

DATE: 07-28-2020

JOB NAME: Bridge Torrance

CALCULATION NAME: Fault Search

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.8154
   SITE LONGITUDE: 118.3018

SEARCH RADIUS: 40 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
PALOS VERDES	2.7(4.4)	7.1	0.486	X
COMPTON THRUST	3.2(5.1)	6.8	0.609	X
NEWPORT-INGLEWOOD (L.A.Basin)	5.5(8.8)	6.9	0.396	X
ELYSIAN PARK THRUST	15.3(24.6)	6.7	0.181	VIII
SANTA MONICA	19.6(31.5)	6.6	0.123	VII
HOLLYWOOD	20.2(32.5)	6.4	0.102	VII
WHITTIER	20.2(32.5)	6.8	0.126	VIII
MALIBU COAST	20.3(32.7)	6.7	0.126	VIII
RAYMOND	22.2(35.7)	6.5	0.097	VII
VERDUGO	25.3(40.7)	6.7	0.095	VII
ANACAPA-DUME	26.9(43.3)	7.3	0.135	VIII
NEWPORT-INGLEWOOD (Offshore)	27.2(43.7)	6.9	0.096	VII
NORTHRIDGE (E. Oak Ridge)	28.1(45.3)	6.9	0.095	VII
SAN JOSE	28.8(46.4)	6.5	0.068	VI
SIERRA MADRE	30.1(48.4)	7.0	0.094	VII
CLAMSHELL-SAWPIT	31.3(50.3)	6.5	0.061	VI

CHINO-CENTRAL AVE. (Elsinore)	32.4(52.1)	6.7	0.068	VI
SIERRA MADRE (San Fernando)	33.7(54.2)	6.7	0.064	VI
SAN GABRIEL	34.8(56.0)	7.0	0.077	VII
SANTA SUSANA	37.7(60.7)	6.6	0.051	VI
ELSINORE-GLEN IVY	38.3(61.6)	6.8	0.057	VI
CUCAMONGA	39.6(63.7)	7.0	0.064	VI

****

-END OF SEARCH- 22 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE PALOS VERDES FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.7 MILES (4.4 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6093 g

Liquefaction Susceptibility Analysis: SPT Method

Based on Youd and Idriss (2001), Martin and Lew (1999).

Project: Bridge Torrance Warehouse

Project No.: 12809.001

Leighton

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)	
LB-1	80	50	0		-50
LB-2	80	50	0		-50
LB-3	80	50	0		-50
LB-4	80	50	0		-50
LB-5	80	50	0		-50
					0
					0
					0

General Parameters:		
a_{max} =	0.85g	MCE
M_W =	7.3	
MSF eq:	1	(Idriss, 2001)
MSF =	1.07	
Hammer Efficiency =	83	%
C_E =	1.38	
C_B =	1	
$C_{S(SPT)}$ =	1.2	
$C_{S(ring)}$ =	1	
Rod Stickup (feet) =	3	
Ring sample correction =	0.65	

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Bridge Torrance Warehouse

Project No.: 12809

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 4	3	4		30	102	31	2	1	20.2	306	35.5	45.7	>Range	306	0.55	0.51	**NonLiq**	45.7	0.02		0.01	0.1
LB-1	4 to 8	5	4		25	120	68	2	1	44.2	528	78.0	91.2	>Range	528	0.55	0.51	**NonLiq**	91.2	0.02		0.01	0.1
LB-1	8 to 13	10	5		20	105	52	2	1	33.8	1091	55.0	63.0	>Range	1090.5	0.54	0.51	**NonLiq**	63.0	0.04		0.03	0.1
LB-1	13 to 18	15	5		30	100	39	2	1	25.4	1603	34.0	44.0	>Range	1603	0.54	0.50	**NonLiq**	44.0	0.04		0.02	0.1
LB-1	18 to 22	20	5		35	110	42	2	1	27.3	2128	35.5	47.6	>Range	2128	0.53	0.49	**NonLiq**	47.6	0.06		0.03	0.0
LB-2	0 to 4	3	4		30	125	30	2	1	19.5	375	34.4	44.4	>Range	375	0.55	0.51	**NonLiq**	44.4	0.03		0.01	1.2
LB-2	4 to 8	5	4		40	105	40	2	1	26.0	605	45.9	60.0	>Range	605	0.55	0.51	**NonLiq**	60.0	0.06		0.03	1.2
LB-2	8 to 13	10	5		**35**	126	22	2	1	14.3	1183	22.3	31.8	>Range	1182.5	0.54	0.51	**NonLiq**	31.8	0.31		0.19	1.2
LB-2	13 to 18	15	5		30	120	39	2	1	25.4	1798	32.1	41.8	>Range	1797.5	0.54	0.50	**NonLiq**	41.8	0.05		0.03	1.0
LB-2	18 to 23	20	5		40	105	49	2	1	31.9	2360	39.4	52.2	>Range	2360	0.53	0.49	**NonLiq**	52.2	0.06		0.04	1.0
LB-2	23 to 28	25	5		**86**	120	14	1	1.2	16.8	2923	18.7	27.4	0.350	2922.5	0.52	0.49	**NonLiq**	27.4	0.91		0.54	0.9
LB-2	28 to 33	30	5		**85**	120	26	2	1	16.9	3523	18.0	26.6	0.328	3522.5	0.52	0.48	**NonLiq**	26.6	0.53		0.32	0.4
LB-2	33 to 38	35	5		20	115	82	1	1.2	98.4	4110	97.0	108.3	>Range	4110	0.49	0.46	**NonLiq**	108.3	0.03		0.02	0.1
LB-2	38 to 43	40	5		10	115	80	2	1	52.0	4685	48.0	49.9	>Range	4685	0.47	0.44	**NonLiq**	49.9	0.06		0.03	0.1
LB-2	43 to 48	45	5		10	105	75	1	1.2	90.0	5235	78.6	81.2	>Range	5235	0.45	0.42	**NonLiq**	81.2	0.04		0.02	0.0
LB-2	48 to 52	50	5		5	110	100	2	1	65.0	5773	54.1	54.1	>Range	5772.5	0.43	0.40	**NonLiq**	54.1			0.00	0.0
LB-3	0 to 4	3	4		20	125	46	2	1	29.9	375	52.7	60.5	>Range	375	0.55	0.51	**NonLiq**	60.5	0.02		0.01	1.2
LB-3	4 to 8	5	4		40	125	49	2	1	31.9	625	56.2	72.4	>Range	625	0.55	0.51	**NonLiq**	72.4	0.01		0.01	1.2
LB-3	8 to 13	10	5		**40**	125	10	2	1	6.5	1250	9.9	16.9	0.179	1250	0.54	0.51	**NonLiq**	16.9	1.43		0.86	1.2
LB-3	13 to 18	15	5		30	130	33	2	1	21.5	1888	26.5	35.3	>Range	1887.5	0.54	0.50	**NonLiq**	35.3	0.21		0.12	0.3
LB-3	18 to 22	20	5		**76**	120	33	2	1	21.5	2513	25.7	35.8	>Range	2512.5	0.53	0.49	**NonLiq**	35.8	0.36		0.20	0.2
LB-4	0 to 4	3	4		20	107	22	2	1	14.3	321	25.2	30.8	>Range	321	0.55	0.51	**NonLiq**	30.8	0.10		0.05	0.4
LB-4	4 to 8	5	4		20	110	28	2	1	18.2	538	32.1	38.3	>Range	538	0.55	0.51	**NonLiq**	38.3	0.27		0.11	0.3
LB-4	8 to 13	10	5		20	122	80	2	1	52.0	1118	83.6	93.8	>Range	1118	0.54	0.51	**NonLiq**	93.8	0.03		0.02	0.2
LB-4	13 to 18	15	5		20	125	60	2	1	39.0	1736	50.3	57.9	>Range	1735.5	0.54	0.50	**NonLiq**	57.9	0.03		0.02	0.2
LB-4	18 to 22	20	5		40	116	27	2	1	17.6	2338	21.8	31.2	>Range	2338	0.53	0.49	**NonLiq**	31.2	0.37		0.20	0.2

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-5	0 to 4	3	4		20	100	40	2	1	26.0	300	45.9	53.1	>Range	300	0.55	0.51	**NonLiq**	53.1	0.01		0.01	1.1
LB-5	4 to 8	5	4		20	120	41	2	1	26.7	520	47.0	54.4	>Range	520	0.55	0.51	**NonLiq**	54.4	0.05		0.02	1.1
LB-5	8 to 13	10	5		30	127	28	2	1	18.2	1138	29.0	38.2	>Range	1137.5	0.54	0.51	**NonLiq**	38.2	0.26		0.15	1.1
LB-5	13 to 18	15	5		**53**	127	22	2	1	14.3	1773	18.2	26.9	0.336	1772.5	0.54	0.50	**NonLiq**	26.9	0.47		0.28	1.0
LB-5	18 to 23	20	5		60	117	24	2	1	15.6	2383	19.2	28.0	0.371	2382.5	0.53	0.49	**NonLiq**	28.0	0.72		0.43	0.7
LB-5	23 to 28	25	5		20	120	43	1	1.2	51.6	2975	56.8	64.9	>Range	2975	0.52	0.49	**NonLiq**	64.9	0.07		0.04	0.2
LB-5	28 to 33	30	5		30	105	34	2	1	22.1	3538	23.5	31.8	>Range	3537.5	0.52	0.48	**NonLiq**	31.8	0.24		0.14	0.2
LB-5	33 to 38	35	5		10	105	59	1	1.2	70.8	4063	70.2	72.6	>Range	4062.5	0.49	0.46	**NonLiq**	72.6	0.04		0.02	0.1
LB-5	38 to 43	40	5		10	101	100	2	1	65.0	4578	60.7	62.9	>Range	4577.5	0.47	0.44	**NonLiq**	62.9	0.04		0.03	0.0
LB-5	43 to 48	45	5		5	110	100	1	1.2	120.0	5105	106.2	106.2	>Range	5105	0.45	0.42	**NonLiq**	106.2	0.03		0.02	0.0
LB-5	48 to 52	50	5		5	110	100	2	1	65.0	5655	54.6	54.6	>Range	5655	0.43	0.40	**NonLiq**	54.6			0.00	0.0

Liquefaction Susceptibility Analysis: SPT Method

Based on Youd and Idriss (2001), Martin and Lew (1999).

Project: Bridge Torrance Warehouse

Project No.: 12809.001

Leighton

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)	
B-1	80	50	0		-50
B-2	80	50	0		-50
B-3	80	50	0		-50
B-4	80	50	0		-50
B-5	80	50	0		-50
					0
					0
					0

General Parameters:		
a_{max} =	0.85g	MCE
M_W =	7.3	
MSF eq:	1	(Idriss, 2001)
MSF =	1.07	
Hammer Efficiency =	83	%
C_E =	1.38	
C_B =	1	
$C_{S(SPT)}$ =	1.2	
$C_{S(ring)}$ =	1	
Rod Stickup (feet) =	3	
Ring sample correction =	0.65	

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Bridge Torrance Warehouse

Project No.: 12809

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
B-1	0 to 4	3	4		20	120	21	2	1	13.7	360	24.1	29.6	0.442	360	0.55	0.51	**NonLiq**	29.6	0.29		0.14	1.4
B-1	4 to 6	5	2		20	120	7	2	1	4.6	600	8.0	12.3	0.134	600	0.55	0.51	**NonLiq**	12.3	2.19		0.52	1.3
B-1	6 to 8	7	2		20	120	15	2	1	9.8	840	17.0	22.0	0.242	840	0.55	0.51	**NonLiq**	22.0	0.49		0.12	0.8
B-1	8 to 11	9	3		20	120	18	2	1	11.7	1080	18.0	23.0	0.258	1080	0.54	0.51	**NonLiq**	23.0	0.66		0.24	0.7
B-1	11 to 16	13	5		20	120	21	1	1.2	25.2	1560	34.3	40.6	>Range	1560	0.54	0.50	**NonLiq**	40.6	0.04		0.02	0.4
B-1	16 to 21	18	5		5	120	22	1	1.2	26.4	2160	34.1	34.1	>Range	2160	0.53	0.50	**NonLiq**	34.1	0.28		0.17	0.4
B-1	21 to 25	23	5		5	120	24	1	1.2	28.8	2760	32.9	32.9	>Range	2760	0.53	0.49	**NonLiq**	32.9	0.43		0.23	0.2
B-2	0 to 3	1	3		20	120	14	1	1.2	16.8	120	29.6	35.6	>Range	120	0.55	0.52	**NonLiq**	35.6	0.02		0.01	0.5
B-2	3 to 5	4	3		20	120	7	1	1.2	8.4	480	14.8	19.6	0.211	480	0.55	0.51	**NonLiq**	19.6	1.13		0.34	0.5
B-2	5 to 7	6	2		40	120	22	1	1.2	26.4	720	46.6	60.9	>Range	720	0.55	0.51	**NonLiq**	60.9	0.02		0.00	0.2
B-2	7 to 11	8	4		60	120	15	1	1.2	18.0	960	29.4	40.3	>Range	960	0.54	0.51	**NonLiq**	40.3	0.06		0.03	0.2
B-2	11 to 16	13	5		30	120	28	1	1.2	33.6	1560	45.7	57.5	>Range	1560	0.54	0.50	**NonLiq**	57.5	0.03		0.02	0.2
B-2	16 to 20	18	5		5	120	24	1	1.2	28.8	2160	37.2	37.2	>Range	2160	0.53	0.50	**NonLiq**	37.2	0.25		0.14	0.1
B-3	0 to 2	1	2		20	120	32	2	1	20.8	120	36.7	43.2	>Range	120	0.55	0.52	**NonLiq**	43.2	0.01		0.00	0.4
B-3	2 to 4	3	2		20	120	28	2	1	18.2	360	32.1	38.3	>Range	360	0.55	0.51	**NonLiq**	38.3	0.10		0.02	0.4
B-3	4 to 6	5	2		20	120	26	2	1	16.9	600	29.8	35.8	>Range	600	0.55	0.51	**NonLiq**	35.8	0.33		0.08	0.3
B-3	6 to 8	7	2		60	120	14	2	1	9.1	840	15.9	24.1	0.274	840	0.55	0.51	**NonLiq**	24.1	0.46		0.11	0.3
B-3	8 to 11	9	3		20	120	27	2	1	17.6	1080	27.0	32.8	>Range	1080	0.54	0.51	**NonLiq**	32.8	0.27		0.10	0.2
B-3	11 to 15	13	4		20	120	20	1	1.2	24.0	1560	32.6	38.9	>Range	1560	0.54	0.50	**NonLiq**	38.9	0.12		0.06	0.1
B-4	0 to 3	1	3		20	120	13	1	1.2	15.6	120	27.5	33.3	>Range	120	0.55	0.52	**NonLiq**	33.3	0.02		0.01	0.4
B-4	3 to 5	4	3		20	120	25	1	1.2	30.0	480	52.9	60.7	>Range	480	0.55	0.51	**NonLiq**	60.7	0.03		0.01	0.4
B-4	5 to 7	6	2		20	120	20	1	1.2	24.0	720	42.3	49.3	>Range	720	0.55	0.51	**NonLiq**	49.3	0.03		0.01	0.4
B-4	7 to 11	8	4		60	120	14	1	1.2	16.8	960	27.4	37.9	>Range	960	0.54	0.51	**NonLiq**	37.9	0.21		0.09	0.3
B-4	11 to 16	13	5		60	120	16	1	1.2	19.2	1560	26.1	36.3	>Range	1560	0.54	0.50	**NonLiq**	36.3	0.13		0.08	0.3
B-4	16 to 20	18	5		60	120	14	1	1.2	16.8	2160	21.7	31.0	>Range	2160	0.53	0.50	**NonLiq**	31.0	0.33		0.18	0.2

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87)	Sat Sand Strain (%) (Tok/ Seed 87)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
B-5	0 to 2	1	2		20	120	14	2	1	9.1	120	16.1	20.9	0.227	120	0.55	0.52	**NonLiq**	20.9	0.05		0.01	1.2
B-5	2 to 4	3	2		20	120	19	2	1	12.4	360	21.8	27.1	0.342	360	0.55	0.51	**NonLiq**	27.1	0.35		0.08	1.2
B-5	4 to 6	5	2		20	120	15	2	1	9.8	600	17.2	22.2	0.245	600	0.55	0.51	**NonLiq**	22.2	0.92		0.22	1.1
B-5	6 to 8	7	2		20	120	19	2	1	12.4	840	21.5	26.9	0.335	840	0.55	0.51	**NonLiq**	26.9	0.42		0.10	0.9
B-5	8 to 11	9	3		20	120	25	2	1	16.3	1080	25.0	30.6	>Range	1080	0.54	0.51	**NonLiq**	30.6	0.29		0.10	0.8
B-5	11 to 16	13	5		60	120	20	1	1.2	24.0	1560	32.6	44.2	>Range	1560	0.54	0.50	**NonLiq**	44.2	0.04		0.02	0.7
B-5	16 to 21	18	5		60	120	16	1	1.2	19.2	2160	24.8	34.8	>Range	2160	0.53	0.50	**NonLiq**	34.8	0.28		0.17	0.7
B-5	21 to 25	23	5		60	120	11	1	1.2	13.2	2760	15.1	23.1	0.259	2760	0.53	0.49	**NonLiq**	23.1	0.93		0.50	0.5

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

LEIGHTON CONSULTING, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

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 Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

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

2.0 Preparation of Areas to be Filled

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 the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. 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.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. 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.

2.3 Overexcavation: In addition to removals and overexcavations 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 overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

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 the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant 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 the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. 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 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

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 D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). 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.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant'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).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant 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, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

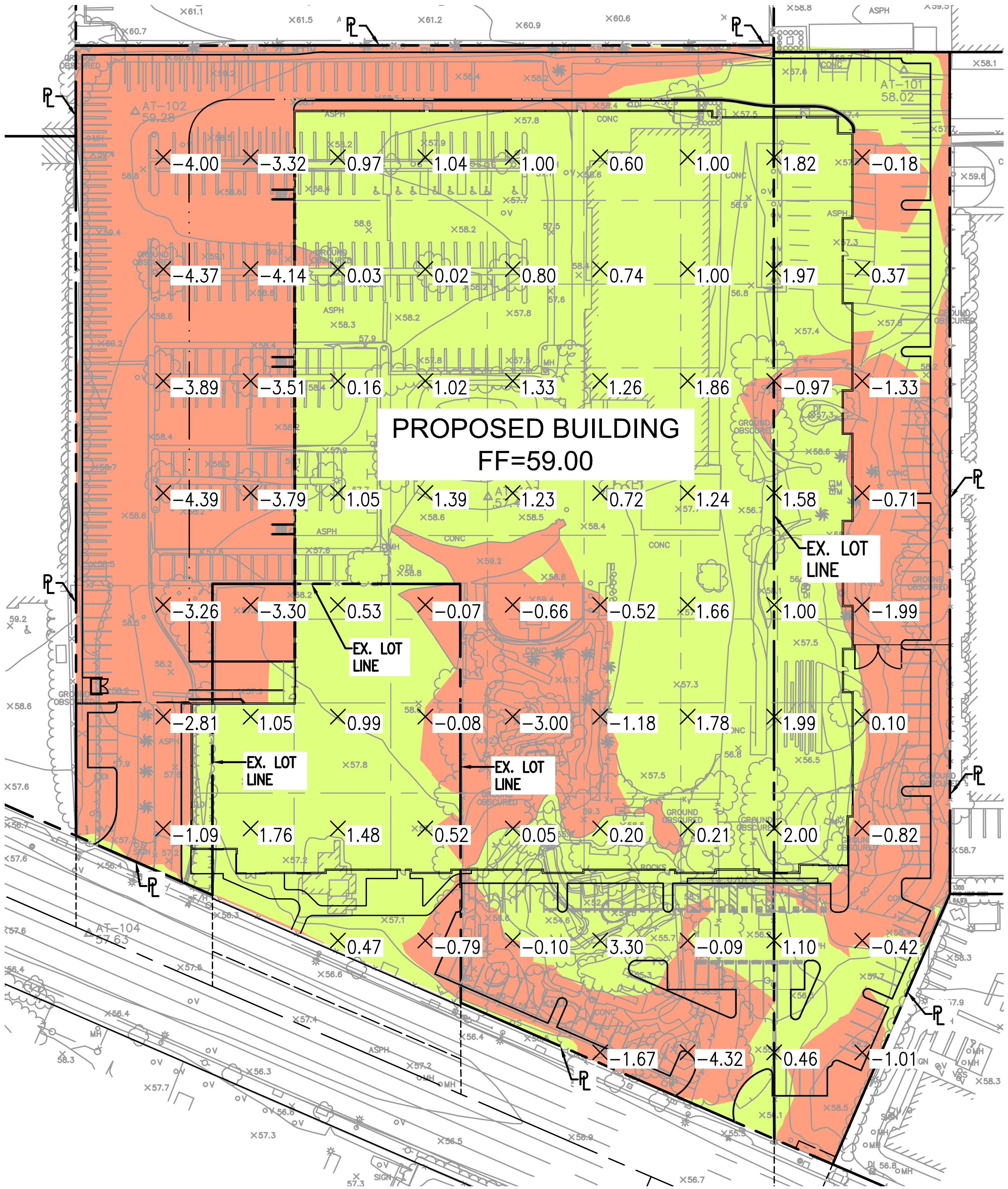
7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

APPENDIX F

EARTHWORK EXHIBIT

Leighton



ELEVATIONS TABLE			
	MIN ELEVATION	MAX ELEVATION	COLOR
CUT	-6.00	0	
FILL	0	6.00	

80' 0' 40' 80' 160'

SCALE: 1" = 80'

EARTHWORK EXHIBIT

BRIDGE POINT SOUTH BAY VII
1355 W. SEPULVEDA BLVD
SHEET 1 OF 1
DATE: 09/15/2020

WestLAND Group, Inc. Land Surveyors • Civil Engineers • GIS

09/15/2020

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

November 20, 2020

Project No. 12809.002

To: Bridge 1355 Sepulveda, LLC
11100 Santa Monica Boulevard, Suite 700
Los Angeles, California 90025

Attention: Mr. Tom Fitzpatrick

Subject: Results of Infiltration Testing
Proposed Warehouse/Industrial Development
1355 West Sepulveda Boulevard
Torrance Area, City of Los Angeles, California

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has conducted infiltration testing in support of the proposed warehouse/industrial development at 1355 West Sepulveda Boulevard in the Torrance Area, City of Los Angeles, California. Leighton previously provided a design-level geotechnical investigation of the site (Leighton, 2020).

We understand that a buried chamber infiltration system will be designed as part of the proposed warehouse/industrial development. We conducted infiltration testing to estimate general infiltration characteristics of onsite soils and provide design recommendations. This report addresses the infiltration characteristics of the onsite soils with respect to the proposed buried infiltration chambers.

Based on our infiltration test results, the tested soils yielded low infiltration rates within silty sands in the upper 25 feet; however, soils encountered became more granular below 25 feet and yielded high infiltration rates. Infiltration system design within deeper sands at the site is feasible. The following report summarizes our field exploration and testing, and presents our conclusions and recommendations.

10532 Acacia Street, Suite B-6 ▪ Rancho Cucamonga, CA 91730-5455
909.484.2205 ▪ 909.484.2170 Fax

Scope of Work

The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical geologic maps, reports, and aerial photographs available in our in-house library.

- Utility Coordination: We contacted Underground Service Alert (USA) prior to excavating borings so that utility companies could mark their utilities onsite.

- Field Exploration: We excavated, logged, and sampled three (3) hollow-stem auger borings (LB-6 to LB-8) to a maximum depth of 34 feet below the existing ground surface in the general areas of the proposed infiltration facilities. The borings were drilled by a subcontracted drill rig operator and logged by a member of our technical staff during drilling. A bulk bag of surface material was collected and Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained.

  All excavations were backfilled with the onsite soil cuttings. Logs of the geotechnical borings and the well permeameter test results are attached. Approximate boring and well permeameter test locations are shown on Figure 2 - *Boring Location Map*.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Sieve analysis for grain-size distribution

  In-situ moisture content and dry density test results are provided on the boring logs. The remaining test results are provided in Appendix C.

- Infiltration Testing: We conducted well permeameter tests within three borings (LB-6 to LB-8) to evaluate general infiltration rates of the subsurface soils at the depth and location tested. The well permeameter tests were conducted based on the USBR 7300-89 method and in general accordance with Los Angeles County guidelines.

The tests were conducted at a depth of approximately 11, 20, and 30 feet (bgs) to estimate infiltration rates. We brought water to the site for the testing.

- Engineering Analysis: Data obtained from our testing was evaluated and analyzed to provide the recommendations presented in this report.

- Report Preparation: Results of our infiltration study have been summarized in this report.

Site Location and Description

The property is approximately 7.4 acres in area and is located at 1355 West Sepulveda Boulevard in the Torrance area in the City of Los Angeles, California. The majority of the property is developed as the Mulligan Miniature Golf amusement center, which includes parking areas and drive aisles in the west, northwest and northeast, amusement buildings in the north, former batting cages, miniature golf course and go-cart track to the south and southwest. The amusement park appears to have been constructed between 1982 and 1994. A building was present on the site before 1975, which appears to have been remodeled and expanded for use in the amusement park. The park appears to have ceased operations sometime in late 2019 to early 2020.

A mostly vacant parcel, not a part of the amusement park, is present in the south central portion of the property and fronts Sepulveda Boulevard to the south. A large metal structure is present in the central portion of the parcel. This or a similar structure has been present since the early 1960s. This structure is reportedly a ready mix tower associated with an Associated Ready Mix and Concrete Plant that was formerly onsite (1361 Sepulveda Blvd). Cement trucks can be seen in aerial photographs taken in this area in the mid to late 2000s until around 2011. End-dumped piles of soil can be observed in this area in photos taken in 2013.

Topographic maps reviewed appear to indicate oil wells were present in the southern portion of the site along future Sepulveda Boulevard and were drilled after 1930. Aerial photographs also appear to show an oil well onsite in the early 1950's. Evidence of the wells is not obvious in subsequent aerial photographs.

The site is generally flat with an average elevation of approximately 60 feet above sea level with slight gradient to the east.

Proposed Development

The Conceptual Grading Plan for the site prepared by WestLAND Group, Inc., dated July 2020 includes construction of a single, approximate 174,211-square-foot warehouse building with associated utility, drainage, parking hardscape and landscape improvements. The western portion of the proposed building includes dock-high truck loading docks.

A marked-up site plan exhibit showing the requested infiltration test location towards the western and southern portion of the site was provided to us by WestLAND Group, Inc. Layouts of the proposed infiltration facilities have not been determined, but are anticipated to be in the tested area in the southern portion of the site (see Figure 2, *Boring Location Map*). The proposed infiltration location on the southern proposed parking lot was inaccessible to drilling equipment due to the existing mini-golf course; borings and infiltration testing were performed within the former concrete plant directly west of the mini-golf course.

Subsurface Soil Conditions

The alluvial soil encountered within our infiltration test borings generally consisted of silty sand, with layers of silt interspersed. In general, the alluvial soil in the upper 25 feet consisted of medium dense to dense silty sand. These soils tended to be moist to very moist. A silt layer was encountered in boring LB-7 at 15 feet.

Soils were observed to become more granular below a depth of 30 feet within boring LB-8. A similar granular layer was encountered in previous site borings at depths of 35 to 50 feet.

More detailed descriptions of the subsurface conditions are presented on the boring logs (Appendix A).

Groundwater

Groundwater was not encountered in our hollow-stem auger borings extending to a depth of 34 feet during our current exploration, or during our previous geotechnical exploration to a maximum depth of 50 feet below the existing ground surface (Leighton, 2020).

Regional data for water wells located within a 1.7-mile radius of the site dating back to 1934 (CDRW, 2018, LA County DPW, 2020 and Water Replenishment District, 2020) indicates water levels in the area at depths in excess of 60 feet with most recent water levels in the range of 80 feet below the ground surface. Shallow groundwater is not anticipated to impact the site.

Infiltration Testing

Three well permeameter tests (LB-6 to LB-8) were conducted to estimate the infiltration rate at specific locations of the site. The well permeameter test was conducted inside the drilled borings at depths of 11 to 30 feet below ground surface.

A well permeameter test is useful for field measurements of soil infiltration rates, and is suited for testing when the design depth of the basin or chamber is deeper than current existing grades. The test consists of excavating a boring to the depth of the test. A layer of clean sand/gravel is placed in the boring bottom to support temporary perforated well casing pipe. In addition, sand/gravel is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. A float valve apparatus, placed inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a constant water head in the boring. The volume percolated during timed intervals is converted into an incremental infiltration rate, in inches per hour. The test was conducted based on the USBR 7300-89 test method.

Our test P-1 located towards the south end of the site indicated small-scale infiltration rates as summarized in the table below. Results of the infiltration testing are provided in Appendix B.

Boring	Test Depth (ft)	Soil Classification	Raw Infiltration Rates (in./hr)	Corrected [1] Infiltration Rates (in./hr)
LB-6	20	Silty Sand	0.7	0.3
LB-7	11	Silty Sand	0.3	0.1
LB-8	30	Sand with Silt	38	19

[1] Factor of Safety of 2 applied for buried chambers.

Infiltration Recommendations

Based on the provided site layout, we understand that the infiltration facilities will be located in planned pavement areas to the west and/or south of the proposed warehouse building.

The silty sand soils encountered within the upper 25 feet yielded low infiltration rates (corrected infiltration rates of 0.1 to 0.3 in./hr), therefore reliance on infiltration within these soils is not recommended. However, soils were observed to become more granular below 30 feet, and an infiltration test within this sand layer at 30 feet yielded high infiltration rates. We recommend that the onsite infiltration be achieved by infiltrating into the encountered sand layer by installing drywells extending to a minimum depth of 40 feet below grade, and using an infiltration rate of 19 in./hr for soils below 28 feet.

Drywells may encounter soils with higher infiltration rates. As such, if drywells are planned, we suggest that drywells be planned with clusters of drywells per general location based on the presumed-conservative infiltration rate. After the first drywell is constructed in each general location, it should be tested for infiltration. If the tested infiltration rates are sufficient to reduce the number of drywells at that location, some or all of the remaining planned drywells may be omitted, as appropriate, based on review of the test data.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

Additional Review and Evaluation:
Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review all infiltration plans, including locations and depths of proposed facilities. Further testing may be required depending on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations:
The periodic flow of water carrying sediments in the drywell, can eventually cause the bottom of the chamber to accumulate a layer of silt, which has the potential of

significantly reducing the overall infiltration rate of the chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within storm water, especially during construction of the project and prior to achieving a mature landscape on site. As it is typically very difficult to remove accumulated silt from buried infiltration facilities, we recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces the prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration.

Estimated infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates.

Construction Considerations:
We recommend that Leighton evaluate drywell excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty of clayey soils are exposed.

Maintenance Considerations:
The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

Closing

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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LP/JDH/SGOrsm

Attachments: References
Figure 1 - Site Location Map
Figure 2 - Boring Location Map
Appendix A - Borings Logs
Appendix B - Infiltration Logs
Appendix C - Laboratory Results

Distribution: (1) Addressee

Leighton

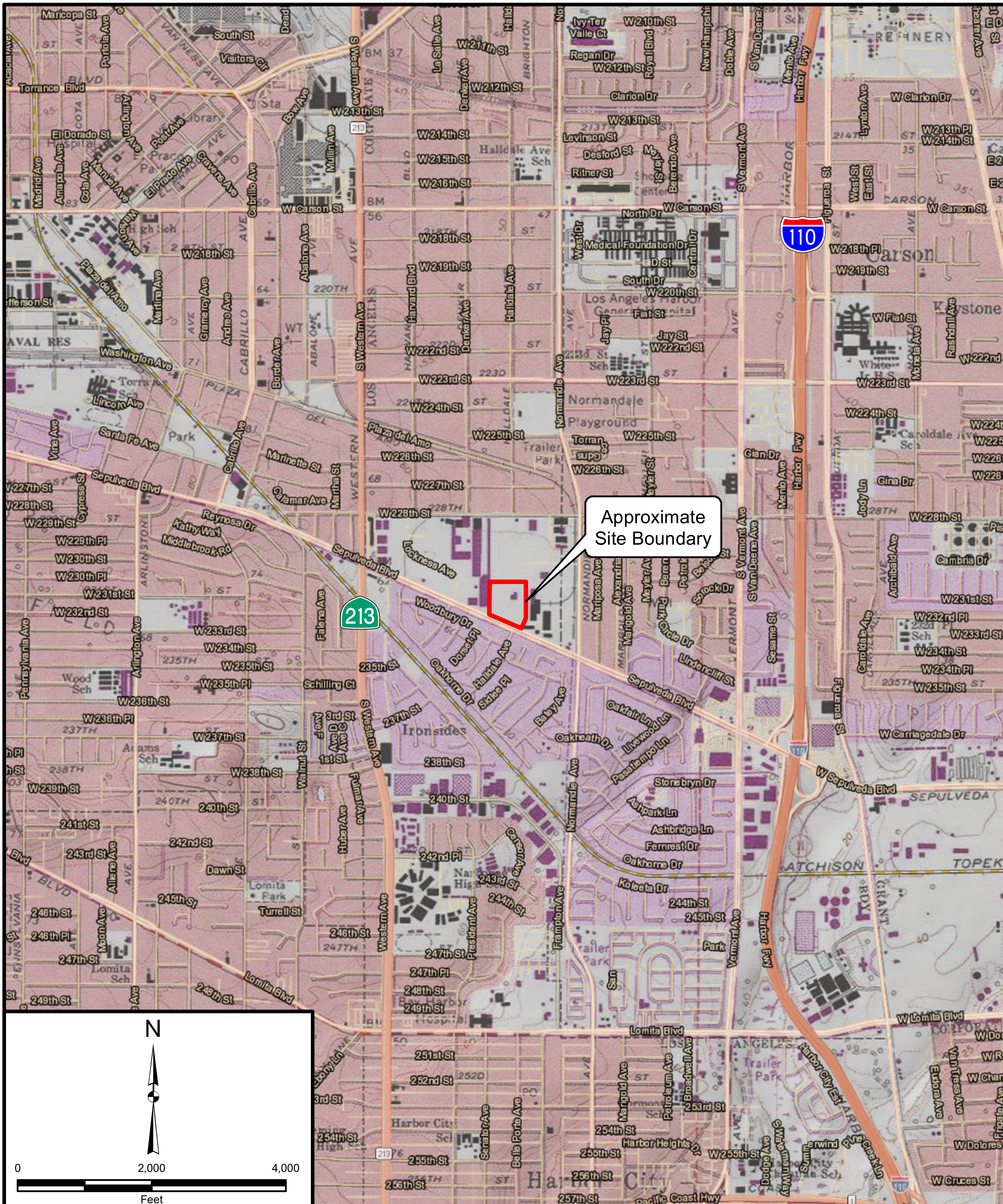
REFERENCES

California Department of Water Resources (CDWR), 2018, California Statewide Groundwater Elevation Monitoring (CASGEM).

Dibblee, T.W., Jr, 1999, Geologic Map of the Palos Verdes Peninsula and Vicinity, Redondo Beach, Torrance and San Pedro Quadrangles, Los Angeles County, California, Dibble Geology Center Map# DF-70.

Leighton Consulting, Inc., 2020, Geotechnical Investigation, Proposed Warehouse/ Industrial Development, 1355 West Sepulveda Boulevard, Torrance Area, City of Los Angeles, California, Project No. 12809.001, dated September 23, 2020.

Los Angeles County Department of Public Works, 2020, Historic Well Measurement Data https://dpw.lacounty.gov/general/wells/



Project: 12809.002	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: November 2020

Base Map: ESRI ArcGIS Online 2020

Author: (mmurphy)

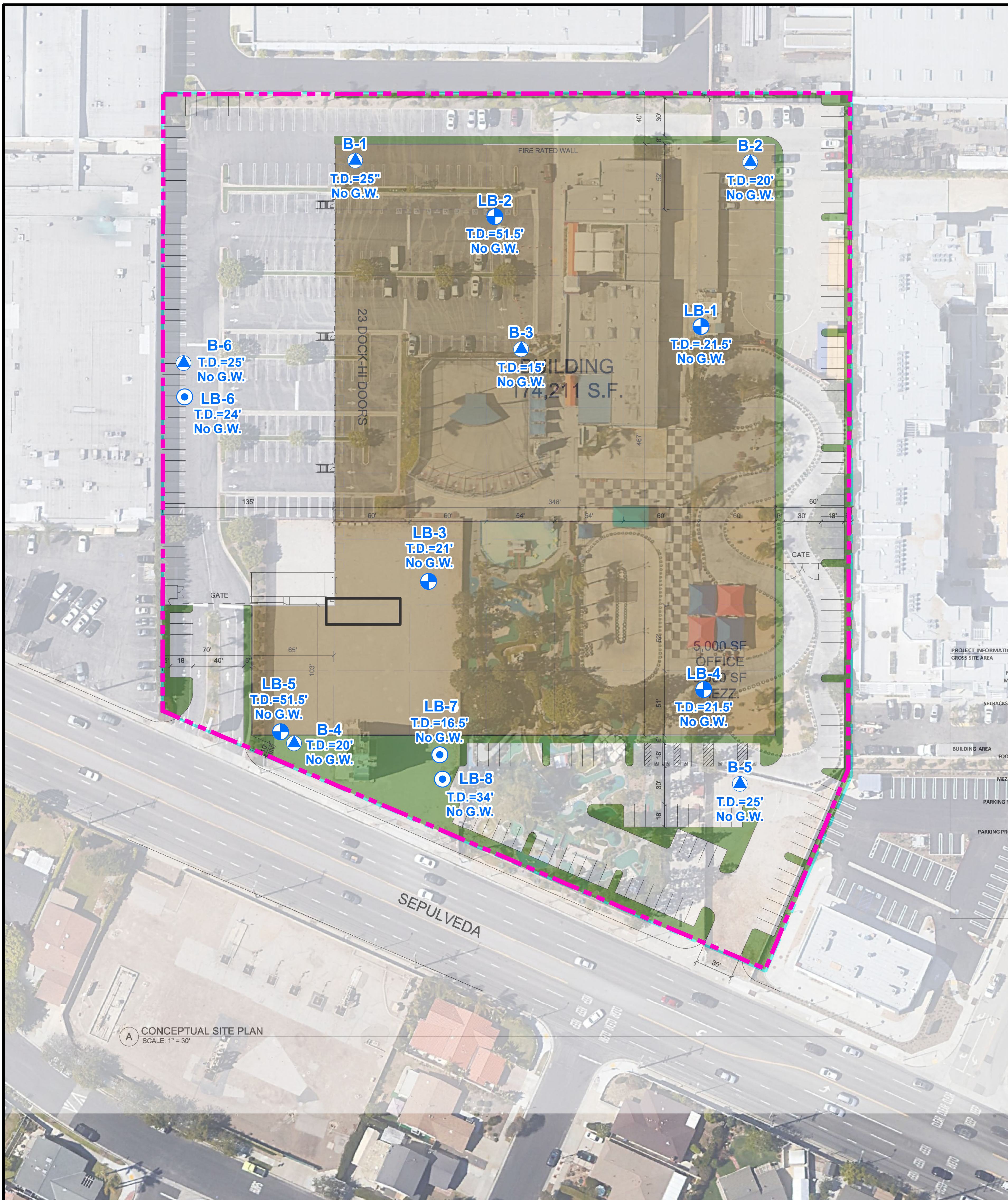
SITE LOCATION MAP

Proposed Warehouse/Industrial Development
1355 W. Sepulveda Blvd
City of Los Angeles, California

Figure 1

Leighton

Map Saved as V:\Drafting\12809\001\Maps\12809-001_F01_SLM_2020-07-23.mxd on 7/23/2020 4:02:16 PM



Legend

LB-8 T.D.=34' No G.W. — Approximate Location of Infiltration Test Boring showing Total Depth of Boring and Groundwater (Leighton, 2020)

LB-5 T.D.=51.5' No G.W. — Approximate Location of Boring showing Total Depth of Boring and Groundwater (Leighton, 2020)

B-6 T.D.=25' No G.W. — Approximate Location of Boring showing Total Depth of Boring and Groundwater (Southern California Geotechnical, 2020)

Approximate Former Location of UST (Roux, 2012)

Approximate Site Boundary

N

0 80 160
Feet

Project: 12809.002	Eng/Geol: JDH/SGO	**BORING LOCATION MAP**	Figure 2
Scale: 1 " = 80 '	Date: November 2020	Proposed Warehouse	
Base Map: Google Earth, 2020 and Conceptual Site Plan by Herdman Architecture + Design, 5/12/2020.		1355 W. Sepulveda Blvd	
Author: (mmurphy)		City of Los Angeles, California	Leighton

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APPENDIX A

BORING LOGS

Leighton

GEOTECHNICAL BORING LOG LB-6

Project No.	12809.002	**Date Drilled**	10-27-20
Project	Proposed Warehouse Development	**Logged By**	ECB
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	52'
Location	see Figure 2	**Sampled By**	ECB

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								@Surface: asphalt pavement	
50										
	5			R1	4 8 12	98	11	SM	@5' SILTY SAND, medium dense, orange brown, moist, nonplastic, 21% fines (lab), fine sand	
45										
	10			R2	5 9 14	106	20	SM	@10' SILTY SAND, medium dense, orange brown, very moist, nonplastic, 29% fines (lab), fine sand	
40										
	15			R3	10 15 26	101	13	SM	@15' SILTY SAND, medium dense, orange brown, very moist, nonplastic, 40% fines (field estimate), fine sand	
35										
	20			R4	11 19 30	109	11	SM	@20' SILTY SAND, medium dense, light brown, moist, nonplastic, 30% fines (field estimate), fine sand	
30										
	25								**T.D.: 24'** **No groundwater encountered** **Boring caved to 23' 3"** **Infiltration Test at 20'** **Backfilled with soil cuttings 10/27/2020**	
25										
	30									
20										
	35									
15										
	40									

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

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

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

GEOTECHNICAL BORING LOG LB-7

Project No.	12809.002	**Date Drilled**	10-27-20
Project	Proposed Warehouse Development	**Logged By**	ECB
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	50'
Location	see Figure 2	**Sampled By**	ECB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
50	0								@Surface: sand, gravel, trash, twigs, grass	
				R1	23 27 31	119	8	SM	@2.5' SILTY SAND, dense, dark brown, slightly moist, nonplastic, 30% fines (field estimate), fine to medium sand, <5% 0.25" in dimension subrounded gravel	
45	5			R2	8 16 23	108	18	SM	@5' SILTY SAND, medium dense, orange brown, moist, nonplastic, 40% fines (field estimate), fine to medium sand	
40	10			R3	10 15 20	105	20	SC-SM	@10' SILTY CLAYEY SAND, medium dense, orange brown, moist, nonplastic, 30% fines (lab), fine sand	
35	15			R4	9 14 22	101	24	ML	@15' SILT with sand, very stiff, yellowish brown, slightly moist, nonplastic, 69% fines (lab), fine sand	
									T.D.: 16.5' **No groundwater encountered** **Boring caved to 13.5'** **Infiltration Test at 11'** **Backfilled with soil cuttings 10/27/2020**	
30	20									
25	25									
20	30									
15	35									
10	40									

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

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

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

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

GEOTECHNICAL BORING LOG LB-8

Project No.	12809.002	**Date Drilled**	10-27-20
Project	Proposed Warehouse Development	**Logged By**	ECB
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	50'
Location	see Figure 2	**Sampled By**	ECB

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
50	0								@Surface: sand, gravel, trash, sticks	
45	5									
40	10								SPOILS: SILTY SAND, brown, moist	
35	15									
30	20									
25	25									
20	30			R1	21 43 50/4"	96	6	SP	@30' SAND with Silt, very dense, tannish brown, slightly moist, nonplastic, 7% fines (lab), fine to medium sand	
15	35								**T.D.: 34'** **No groundwater encountered** **Boring caved to 30' 8"** **Backfilled to 30' 4"** **Infiltration Test at 30'** **Backfilled with soil cuttings 10/27/2020**	
10	40									

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

TYPE OF TESTS:

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

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

APPENDIX B

INFILTRATION LOGS

Leighton

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project:	12809	Initial estimated Depth to Water Surface (in.):	195
Exploration #/Location:	*LB-6*	Average depth of water in well, "h" (in.):	44
Depth Boring drilled to (ft):	24	approx. h/r:	11.1
Tested by:	EB	Tu (Fig. 8) (ft):	43.8
USCS Soil Type in test zone:	SC	Tu>3h?:	yes, OK
Weather (start to finish):	Sunny		
Liquid Used/pH:	H2O		
Measured boring diameter:	8 in.	4 in. Well Radius	Cross-sectional area for vol calcs (in.^2): 20.1
Approx Depth to GW below GS:	60 ft		

Well Prep: Tamped bottom to 20.5', set 3" sand, placed pipe, backfill with sand around pipe in test zone.

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	19. ft	11. in.	239	
Pilot Tube stickup (+ is above ground)		2. in.	2	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	15. ft	0. in.	180	178 Depth below GS (in.)
Float Assembly ID		F		
Float assembly Extension length (in.)		30		

Diameter of barrels (in.):	22.5
No. of Supply barrels:	1
Total Area of barrels (in.^2):	397.4

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of pilot tube) ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	Vol Change from Δh	Vol Change Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 10/27/2020	Start time: 13:55																			
10/27/20	13:55	29	16.7					0	198.4	40.6										
10/27/20	14:00	26	16.45				5	5	195.4	43.6	3	42	1192	-60	1132	226	13584	1108	0.9	**11.30**
10/27/20	14:05	24.755	16.35				5	10	194.2	44.8	1.2	44	495	-24	471	94	5648	1161	0.9	**4.48**
10/27/20	14:10	24.25	16.35				5	15	194.2	44.8	0	45	201	0	201	40	2408	1176	0.9	**1.89**
10/27/20	14:20	23.75	16.35				10	25	194.2	44.8	0	45	199	0	199	20	1192	1176	0.9	**0.93**
10/27/20	14:35	22.75	16.35				15	40	194.2	44.8	0	45	397	0	397	26	1590	1176	0.9	**1.25**
10/27/20	15:10	21	16.4				35	75	194.8	44.2	-0.6	45	695	12	708	20	1213	1168	0.9	**0.96**
10/27/20	15:35	20	16.4				25	100	194.8	44.2	0	44	397	0	397	16	954	1161	0.9	**0.76**
10/27/20	15:55	19.25	16.4				20	120	194.8	44.2	0	44	298	0	298	15	894	1161	0.9	**0.71**
10/27/20	16:20	18	16.4				25	145	194.8	44.2	0	44	497	0	497	20	1192	1161	0.9	**0.95**
10/27/20	16:40	17.25	16.4				20	165	194.8	44.2	0	44	298	0	298	15	894	1161	0.9	**0.71**
10/27/20																				
10/27/20																				

template updated: 6/30/2020

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project:	12809	Initial estimated Depth to Water Surface (in.):	92
Exploration #/Location:	LB-7	Average depth of water in well, "h" (in.):	39
Depth Boring drilled to (ft):	15	approx. h/r:	9.7
Tested by:	EB	Tu (Fig. 8) (ft):	52.3
USCS Soil Type in test zone:	SC	Tu>3h?:	yes, OK
Weather (start to finish):	Sunny		
Liquid Used/pH:	H2O		
Measured boring diameter:	8 in.	4 in. Well Radius	Cross-sectional area for vol calcs (in.^2): 20.1
Approx Depth to GW below GS:	60 ft		

Well Prep: Tamped bottom to 11', set 3" sand, placed pipe, backfill with sand around pipe in test zone.

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	10. ft	11. in.	131	
Pilot Tube stickup (+ is above ground)		0. in.	0	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	6. ft	0. in.	72	72 Depth below GS (in.)
Float Assembly ID		E		
Float assembly Extension length (in.)		30		

Diameter of barrels (in.): 22.5

No. of Supply barrels: 1

Total Area of barrels (in.^2): 397.4

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of pilot tube) ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	Vol Change from Δh	Vol Change Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 10/27/2020	Start time: 13:05																			
10/27/20	13:05	31	7.7					0	92.4	38.6										
10/27/20	13:10	27	7.7				5	5	92.4	38.6	0	39	1590	0	1590	318	19076	1020	0.9	**17.23**
10/27/20	13:15	27.25	7.7				5	10	92.4	38.6	0	39	-99	0	-99	-20	-1192	1020	0.9	**-1.08**
10/27/20	13:20	27.25	7.7				5	15	92.4	38.6	0	39	0	0	0	0	0	1020	0.9	**0.00**
10/27/20	13:30	27.12	7.7				10	25	92.4	38.6	0	39	52	0	52	5	310	1020	0.9	**0.28**
10/27/20	13:45	27	7.7				15	40	92.4	38.6	0	39	48	0	48	3	191	1020	0.9	**0.17**
10/27/20	14:15	26.75	7.7				30	70	92.4	38.6	0	39	99	0	99	3	199	1020	0.9	**0.18**
10/27/20	14:38	26.25	7.7				23	93	92.4	38.6	0	39	199	0	199	9	518	1020	0.9	**0.47**

template updated: 6/30/2020

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: 12809

Exploration #/Location:	LB-8		Initial estimated Depth to Water Surface (in.):	349
Depth Boring drilled to (ft):	34		Average depth of water in well, "h" (in.):	11
Tested by:	EB		approx. h/r:	2.8
USCS Soil Type in test zone:	SP		Tu (Fig. 8) (ft):	30.9
Weather (start to finish):	Sunny		Tu>3h?:	yes, OK
Liquid Used/pH:	H2O			
Measured boring diameter:	8 in.	4 in. Well Radius	Cross-sectional area for vol calcs (in.^2):	20.1
Approx Depth to GW below GS:	60 ft			

Well Prep: Drilled to 34, caved to 32, Tamped bottom to 30', set 3" sand, placed pipe, backfill with sand around pipe in test zone.

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	30. ft	0. in.	360	
Pilot Tube stickup (+ is above ground)		4. in.	4	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	25. ft	0. in.	300	296 Depth below GS (in.)
Float Assembly ID		E		
Float assembly Extension length (in.)		30		

Diameter of barrels (in.): 22.5

No. of Supply barrels: 1

Total Area of barrels (in.^2): 397.4

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of pilot tube) ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 10/27/2020	Start time: 14:47																			
10/27/20	14:47	26	29.6					0	351.2	8.8										
10/27/20	14:55	20.75	29.6				8	8	351.2	8.8	0	9	2086	0	2086	261	15648	271	0.9	**53.14**
10/27/20	15:00	18	29.5				5	13	350.0	10.0	1.2	9	1093	-24	1069	214	12825	286	0.9	**41.27**
10/27/20	15:05	14.25	29.45				5	18	349.4	10.6	0.6	10	1490	-12	1478	296	17739	309	0.9	**52.90**
10/27/20	15:15	7	29.4				10	28	348.8	11.2	0.6	11	2881	-12	2869	287	17215	324	0.9	**48.95**
10/27/20	15:23	2	29.4				8	36	348.8	11.2	0	11	1987	0	1987	248	14903	332	0.9	**41.41**
10/27/20						2nd barrel														
10/27/20	15:24	29	29.4					37	348.8	11.2										
10/27/20	15:30	25.75	29.4				6	43	348.8	11.2	0	11	1292	0	1292	215	12916	332	0.9	**35.89**
10/27/20	15:40	19.75	29.4				10	53	348.8	11.2	0	11	2384	0	2384	238	14307	332	0.9	**39.76**
10/27/20	15:50	13.25	29.4				10	63	348.8	11.2	0	11	2583	0	2583	258	15499	332	0.9	**43.07**
10/27/20	16:00	7.25	29.4				10	73	348.8	11.2	0	11	2384	0	2384	238	14307	332	0.9	**39.76**
10/27/20	16:07	3	29.4			Empty	7	80	348.8	11.2	0	11	1689	0	1689	241	14477	332	0.9	**40.23**

APPENDIX C

LABORATORY RESULTS

Boring No.	LB-6	LB-6	LB-7	LB-7	LB-8			
Sample No.	R1	R2	R3	R4	R1			
Depth (ft.)	5.0	10.0	10.0	15.0	30.0			
Sample Type	Ring	Ring	Ring	Ring	Ring			
Soil Identification	Brown silty sand (SM)	Brown silty sand (SM)	Brown silty sand (SM)	Brown sandy silt s(ML)	Brown poorly-graded sand with silt (SP-SM)			
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00			
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00			
Sample Dry Weight Determination								
Weight of Sample + Container (g)	722.40	751.70	830.10	710.00	696.80			
Weight of Container (g)	109.10	110.40	108.20	106.30	110.20			
Weight of Dry Sample (g)	613.30	641.30	721.90	603.70	586.60			
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A			
Dry Weight of Sample + Cont. (g)	591.70	565.60	611.30	294.20	653.10			
Weight of Container (g)	109.10	110.40	108.20	106.30	110.20			
Dry Weight of Sample (g)	482.60	455.20	503.10	187.90	542.90			
% Passing No. 200 Sieve	**21.3**	**29.0**	**30.3**	**68.9**	**7.4**			
% Retained No. 200 Sieve	78.7	71.0	69.7	31.1	92.6			

Leighton

PERCENT PASSING No. 200 SIEVE ASTM D 1140

Project Name: Bridge 1355 Sepulveda
Project No.: 12809.002
Tested By: S. Felter Date: 11/12/20

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

January 29, 2021

Project No. 12809.001

Bridge 1355 Sepulveda, LLC
11100 Santa Monica Boulevard ,Suite 700
Los Angeles, California 90025

Attention: Mr. Tom Fitzpatrick

Subject: Response to Los Angeles Grading Division of Building and Safety Review Sheet dated January 12, 2021, Proposed 1355 W. Sepulveda Boulevard Warehouse, Torrance Area, City of Los Angeles, California

In accordance with your request, Leighton Consulting, Inc. (Leighton) is providing this response to the City of Los Angeles Grading Division of Building and Safety (LADBS) review of our geotechnical investigation report (Leighton, 2020a) and infiltration testing report (Leighton, 2020b) for the proposed 1355 W. Sepulveda Boulevard warehouse development. LADBS's review sheet dated January 12, 2021, is attached to this letter. The proposed project is located at 1355 W. Sepulveda Boulevard in the Torrance area of the City of Los Angeles, California.

Our responses to LADBS's comments have been numbered to match the sequence of comments from their review sheet.

10532 Acacia Street, Suite B-6 ▪ Rancho Cucamonga, CA 91730-5455
909.484.2205 ▪ 909.484.2170 Fax

<u>Comment 1</u>:
Provide a site plan drawn at the appropriate engineering scale showing clearly all existing and proposed structures, grades, borings, and property lines. The plan shall be suitable for reproduction and archival purposes. <u>Note</u>: Aerial photos on their own are not acceptable. (P/BC 2014-113)

Response 1:
An updated site plan showing proposed structures, proposed infiltration dry well locations, existing and proposed grades, boring locations, and property lines is attached as Figure 2.

<u>Comment 2</u>:
Provide direct shear test results for the earth material to substantiate the bearing capacity, anticipated settlement determinations, and design calculations.

Response 2:
A remolded direct shear test previously performed by Southern California Geotechnical was included in Appendix C of our geotechnical investigation report (Leighton, 2020a). The direct shear test was remolded to 90 percent relative compaction from a silty sand bulk sample obtained within the upper 5 feet from existing grade. The direct shear test resulted in peak values of 32 internal friction angle and 400 psf cohesion, and ultimate values of 31 internal friction angle and 100 psf cohesion.

In prepration of this response, we have performed two additional direct shear tests on relatively undisturbed modified California ring samples from Boring LB-2 at a depth of 5 feet and Boring LB-5 at a depth of 2.5 feet. The direct shear laboratory test results are as follows:

Boring	Depth (ft)	Material	Strength Parameters (Ultimate)	
			Cohesion (psf)	Friction Angle (deg)
LB-2	5	Sandy Silt	48	33
LB-5	2.5	Silt with Sand	538	32

The direct shear lab test results are attached to this response letter.

The geotechnical design recommendations provided in our geotechnical report (Leighton, 2020a) were based on soil with a friction angle of 31 degrees and remain applicable based on the additional direct shear laboratory tests performed. Design calculations for bearing capacity based on the direct shear laboratory tests are provided herein.

Bearing Capacity Calculation

Bearing capacity was determined using Terzaghi's bearing capacity equation:

$$q_{ult} = c\ N_c\ s_c + q\ D_f\ N_q + ½\ \gamma\ N_\gamma\ B\ s_\gamma$$

where,

q_{ult} = ultimate soil bearing pressure
c = apparent cohesion in psf
γ = unit weight in pcf
D_f = footing depth below grade in feet
B = footing width in feet
N_c, N_q and N_γ are bearing capacity factors
S_c and S_y are shape factors (strip footing S_c = 1.0, S_y = 1.0; spread footing S_c = 1.3, S_y = 0.8)

For the angle of internal friction value of 31 degrees, the corresponding bearing capacity factors are: N_c = 33, N_q = 10, N_γ = 12

For Strip Footings:

q_{ult} = (100 psf)(33)(1.0) + (120 pcf)(1.5 ft)(10) + (1/2)(120 pcf)(12)(2 ft)(1.0)
q_{ult} = 6,540 psf
q_{all} = 2,180 psf [Factor of Safety = 3]

For Spread Footings:

q_{ult} = (100 psf)(33)(1.3) + (120 pcf)(1.5 ft)(10) + (1/2)(120 pcf)(12)(2 ft)(0.8)
q_{ult} = 7,242 psf
q_{all} = 2,414 psf [Factor of Safety = 3]

The resultant allowable bearing capacities are greater than the recommended allowable bearing capacity of 2,000 psf. Performing the bearing capacity calculations with parameters from the additional direct shear tests would result in higher allowable bearing pressures.

Comment 3:

Provide shoring design calculation to justify the recommendations. All surcharge loads shall be included into design.

Response 3:

Section 3.7 of our geotechnical investigation report (Leighton, 2020a) included recommendations for temporary excavations. An active equivalent fluid pressure of 35 pcf was provided for cantilever shoring design. For excavation braced at the top and at specific design intervals, the active pressure may be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

Earth pressures for temporary braced walls shoring were based on Terzaghi and Peck. For sands, earth pressure was calculated with the following equation:

$\sigma = 0.65\ K_A\ \gamma\ H$

where:

$K_A = TAN^2\ (45 - \phi/2)$

For an angle of internal friction value of 31 degrees and soil unit weight of 120 pcf, this is (σ = 25H), as recommended in our report. Additional direct shear laboratory tests performed resulted in friction angles of 32 and 33. Using a friction angle of 33, the earth pressure is reduced to 23H. The lateral earth pressure of 25H recommended in our geotechnical report is more conservative and remains applicable.

Surcharge loads are not included in the design and should be evaluated on a case-by-case basis if required during construction.

Comment 4:

Provide recommendations for unsurcharged temporary excavations.

Response 4:

Our recommendations included in Section 3.7 of the geotechnical investigation report (Leighton, 2020a) do not include surcharge loading and are applicable for unsurcharged temporary excavations. Design calculations are provided in the response to comment 3 of this letter.

If surcharged loads are within a horizontal distance equal to the height of the cut or 5 feet, whichever is greater, then additional recommendations would be provided on a case by case basis.

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent footings.

Comment 5:

In the event retaining walls are proposed, provide design calculations to justify the recommendations. A plan showing the location of the retaining walls shall be submitted to the Grading Division for review.

Response 5:
Retaining wall recommendations are provided in Section 3.5 of our geotechnical investigation report (Leighton, 2020a). Our recommendations include lateral earth pressure parameters to be used for retaining wall design. Retaining wall calculations (sliding, overturning, etc.) would be provided by the project structural engineer. We understand that retaining walls over 6 feet are not planned for this development.

The current site plan exhibit shows locations of retaining walls with retained soil heights less than 6 feet located on the western and northern portions of the site near the property lines. The proposed retaining walls have an offset of 3 feet from the property line for the western wall, and 3 to 4½ feet for the northern wall.

Lateral earth pressure calculations are provided herein, and our recommendations remain applicable:

Based on direct shear test data (conservatively assuming $\phi = 31^{o}$) and soil unit weight of 120 pcf:

Static Equivalent Fluid Weights (pcf) = (unit weight of soil)x(lateral earth coefficient)

Active: $[K_A]\ \gamma = [TAN^2\ (45 - \phi/2)]\ \gamma = (0.32)(120) = 38$ pcf

At-Rest: $[K_o]\ \gamma = [1\text{-}SIN(\phi)]\ \gamma = (0.49)(120) = 58$ pcf

Passive: $[K_P]\ \gamma = [TAN^2\ (45 + \phi/2)]\ \gamma = (3.12)(120) = 374$ pcf / 1.5 FS = 250 pcf

These values should be used in the design of retaining walls.

<u>Comment 6</u>:

Provide a revised plan(s) drawn to scale and suitable for reproduction and achieving purposes that clearly shows all property lines, proposed and existing grades and structures, and the location of the proposed infiltration system. The plan shall be provided on the soils consultant's stationery or shall be signed and stamped by the soils engineer.

Note: On-site infiltration systems are required to be a minimum of 10 feet (in any direction) from any foundation, and a minimum of 10 feet horizontally from private property lines.

Response 6:

Similar to Comment 1, an updated Figure 2 site plan showing proposed structures, proposed infiltration dry well locations, existing and proposed grades, boring locations, and property lines is attached as Figure 2.

If you have any questions regarding this letter, please contact our office. We appreciate the opportunity to be of continued service.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Jason D. Hertzberg, GE 2711
Principal Engineer
Ext. 8772, jhertzberg@leightongroup.com

Steven G. Okubo, CEG 2706
Project Geologist
Ext. 8773, sokubo@leightongroup.com

Luis Perez-Milicua, PE 89389
Project Engineer
Ext. 8777, LPerez-Milicua@leightongroup.com

LP/SGO/JDH

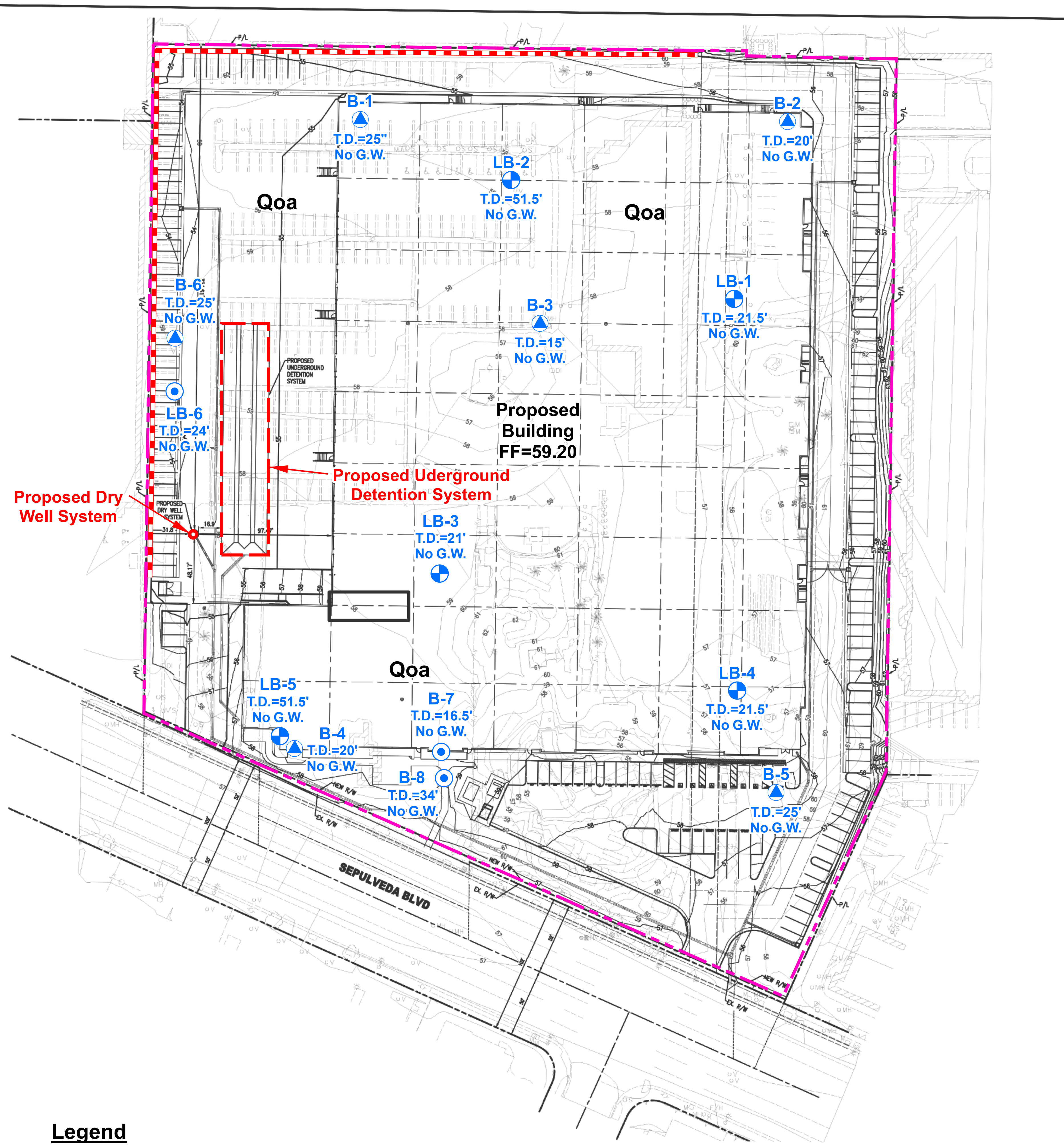
Attachment: References
Figure 2 – Boring Location Map
Direct Shear Lab Tests
LADBS Review Letter dated January 12, 2021

Distribution: (1) Addressee

REFERENCES

Leighton Consulting, Inc., 2020a, Geotechnical Investigation, Proposed Warehouse /Industrial Development, 1355 West Sepulveda Boulevard, Torrance Area, City of Los Angeles, California, Project No. 12809.001, dated September 23, 2020.

Leighton Consulting, Inc., 2020b, Results of Infiltration Testing, Proposed Warehouse /Industrial Development, 1355 West Sepulveda Boulevard, Torrance Area, City of Los Angeles, California, Project No. 12809.002 dated November 20, 2020.



Legend

LB-8 T.D.=34' No G.W.	Approximate Location of Infiltration Test Boring showing Total Depth of Boring and Groundwater (Leighton, 2020)
LB-5 T.D.=51.5' No G.W.	Approximate Location of Boring showing Total Depth of Boring and Groundwater (Leighton, 2020)
B-6 T.D.=25' No G.W.	Approximate Location of Boring showing Total Depth of Boring and Groundwater (Southern California Geotechnical, 2020)
	Proposed Retaining Wall (Less Than 6-Feet Retained Soil Height)
	Approximate Former Location of UST (Roux, 2012)
	Approximate Site Boundary
Qoa	Older Alluvium
58	Existing Site Grade (ft)
56	Proposed Grades (ft)

N

0 80 160
Feet

Project: 12809.001	Eng/Geol: JDH	BORING LOCATION MAP Proposed Warehouse 1355 W. Sepluveda Blvd City of Los Angeles, California	Figure 2
Scale: 1 " = 80 '	Date: January 2021		Leighton
Base Map: Site Plan Exhibit, Bridge Development City of Los Angeles, Sheet 1 of 1, Dated: 01/14/2021			
Author: (btran)			

Map Saved as V:\Drafting\12809\001\Maps\12809-001_F02_BLM_Aerial_2021-01-28.mxd on 1/20/2021 4:36:21 PM

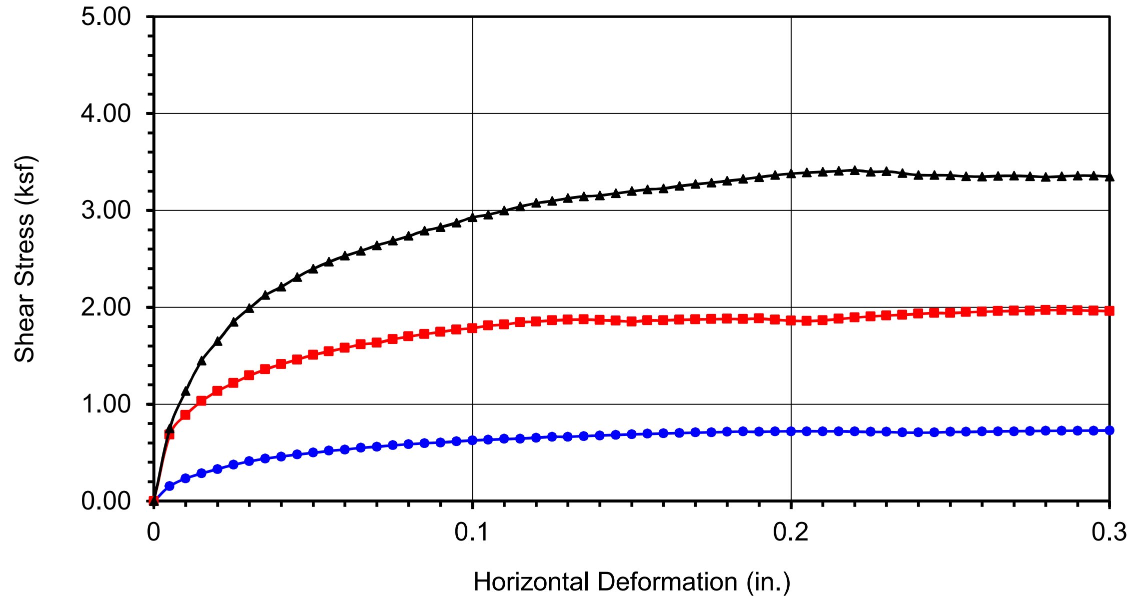
Leighton

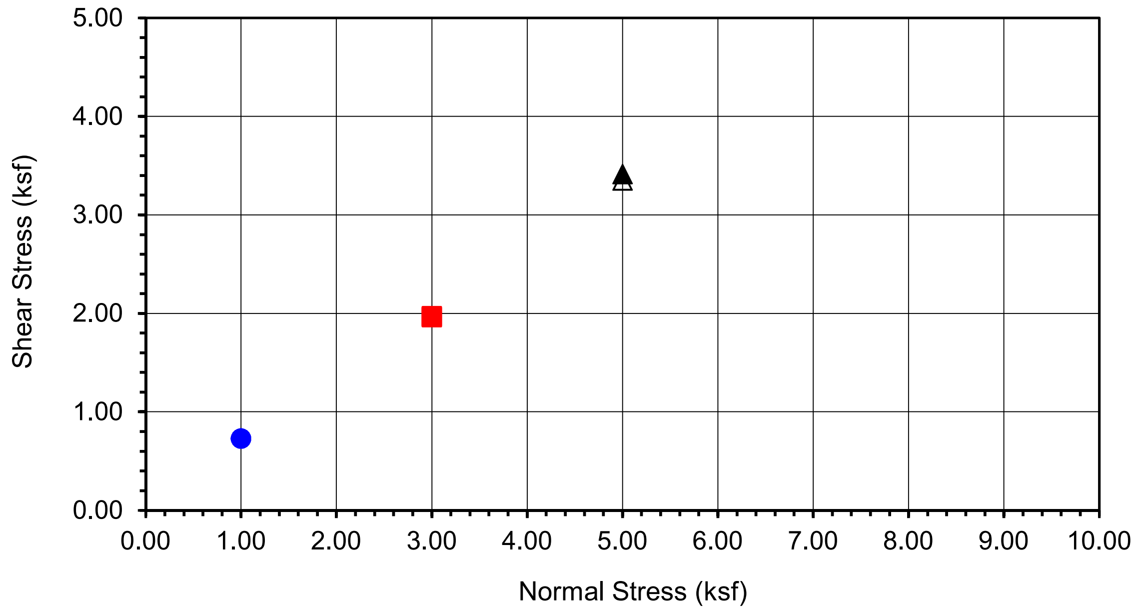
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Bridge/Torrance	Tested By:	G. Bathala	Date:	01/22/21
Project No.:	12809.001	Checked By:	A. Santos	Date:	01/25/21
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R2	Depth (ft.):	5.0		
Soil Identification:	Brown sandy silt s(ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	173.48	173.02	174.97
Weight of Ring(gm):	45.48	44.52	42.54
Before Shearing			
Weight of Wet Sample+Cont.(gm):	118.99	118.99	118.99
Weight of Dry Sample+Cont.(gm):	115.64	115.64	115.64
Weight of Container(gm):	62.09	62.09	62.09
Vertical Rdg.(in): Initial	0.0000	0.0000	0.0000
Vertical Rdg.(in): Final	-0.0105	-0.0315	-0.0256
After Shearing			
Weight of Wet Sample+Cont.(gm):	172.62	190.65	175.01
Weight of Dry Sample+Cont.(gm):	153.34	171.40	155.20
Weight of Container(gm):	39.01	58.18	35.69
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**R2**
Depth (ft)	**5**

Sample Type:

Ring

Soil Identification:
Brown sandy silt s(ML)

Normal Stress (kip/ft²)	1.000	3.000	5.000
Peak Shear Stress (kip/ft²)	● 0.729	■ 1.971	▲ 3.414
Shear Stress @ End of Test (ksf)	○ 0.729	□ 1.959	△ 3.348
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.26	6.26	6.26
Dry Density (pcf)	100.2	100.6	103.7
Saturation (%)	24.7	25.0	27.0
Soil Height Before Shearing (in.)	0.9895	0.9685	0.9744
Final Moisture Content (%)	16.9	17.0	16.6

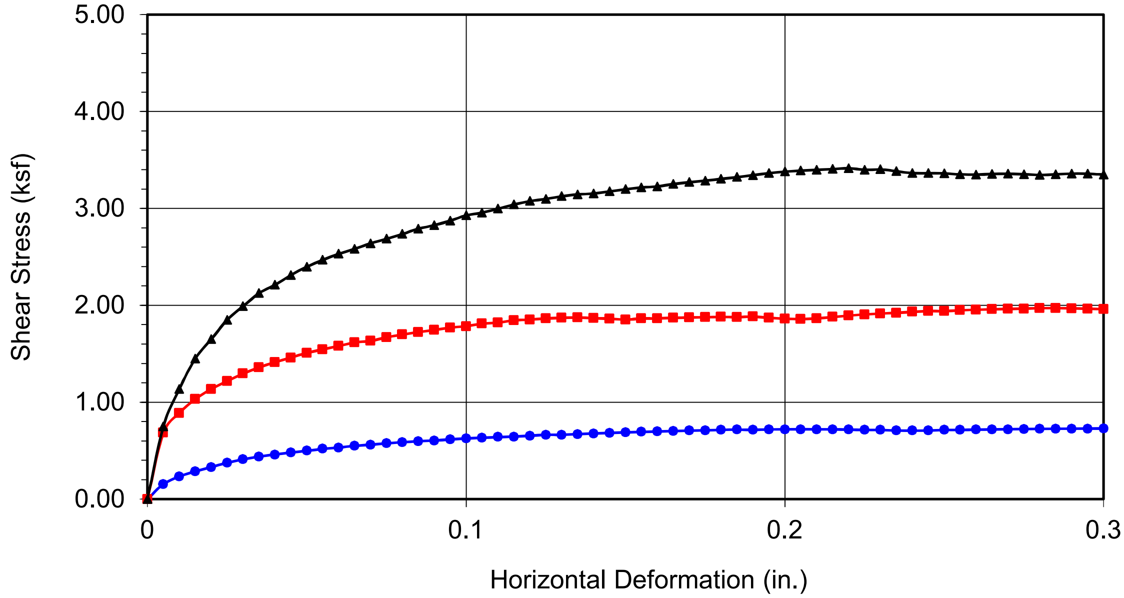
Leighton

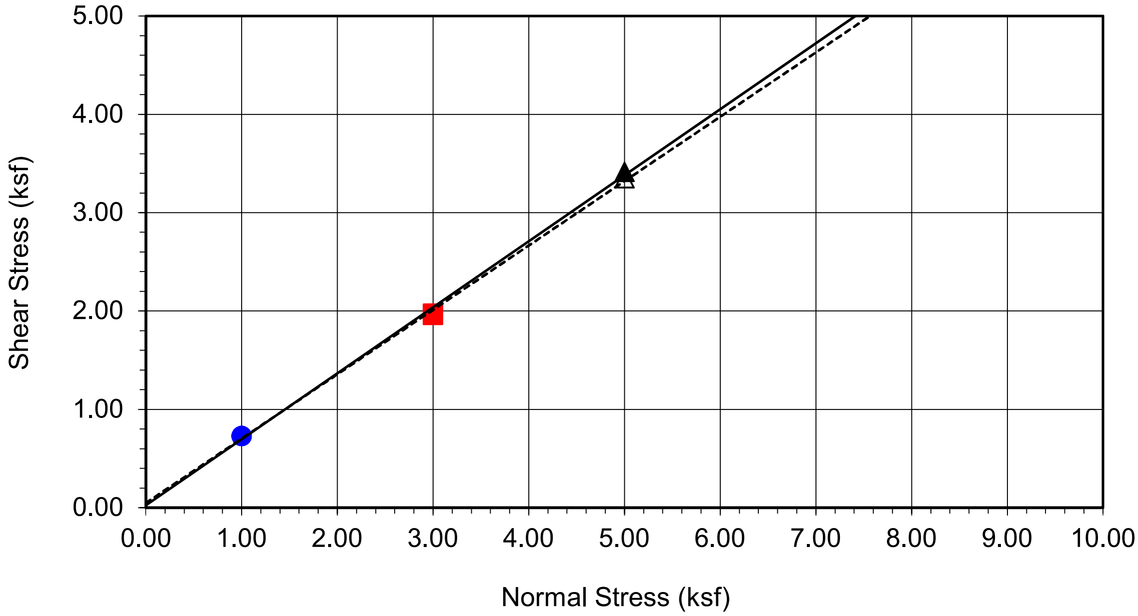
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12809.001

Bridge/Torrance

01-21





Boring No.	**LB-2**
Sample No.	**R2**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:
Brown sandy silt s(ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	24	34
Ultimate	48	33

Normal Stress (kip/ft²)	1.000	3.000	5.000
Peak Shear Stress (kip/ft²)	● 0.729	■ 1.971	▲ 3.414
Shear Stress @ End of Test (ksf)	○ 0.729	□ 1.959	△ 3.348
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.26	6.26	6.26
Dry Density (pcf)	100.2	100.6	103.7
Saturation (%)	24.7	25.0	27.0
Soil Height Before Shearing (in.)	0.9895	0.9685	0.9744
Final Moisture Content (%)	16.9	17.0	16.6

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12809.001

Bridge/Torrance

01-21

Leighton

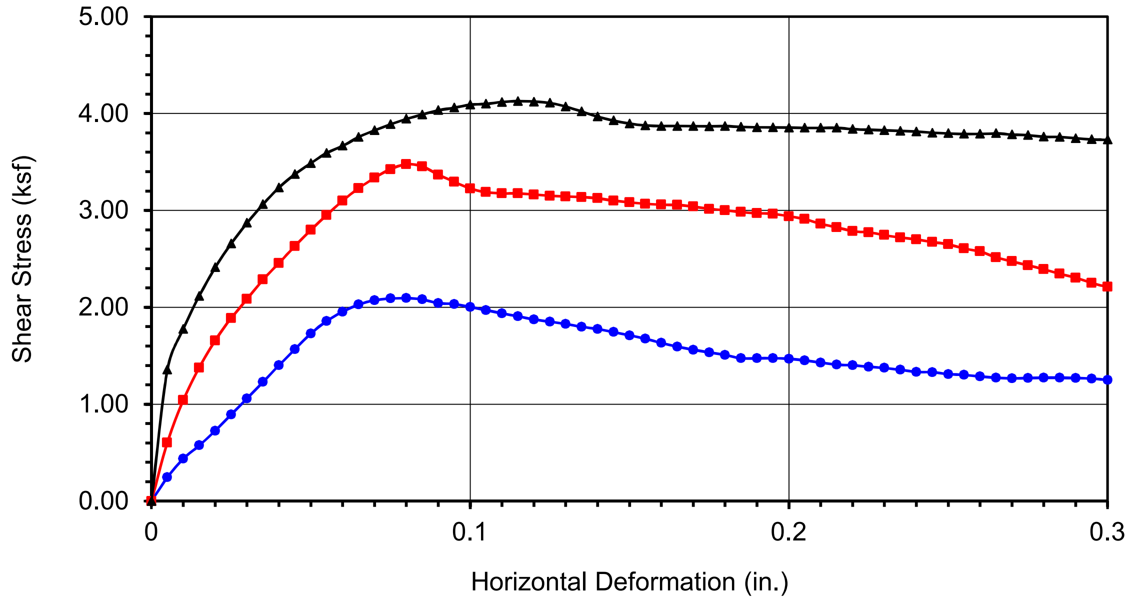
DIRECT SHEAR TEST

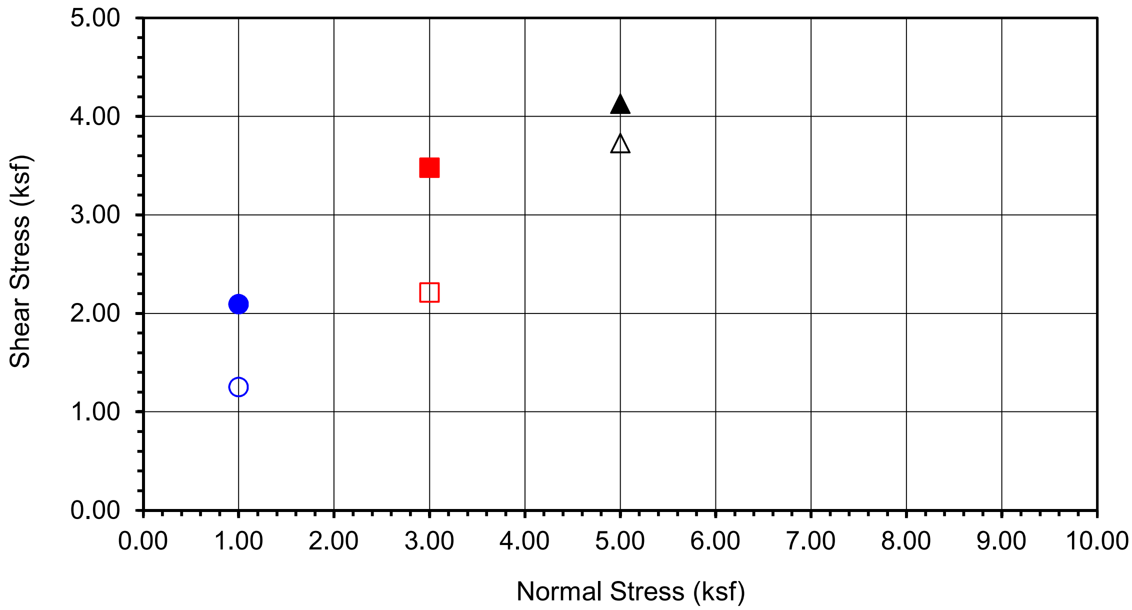
Consolidated Drained - ASTM D 3080

Project Name: Bridge/Torrance
Project No.: 12809.001
Boring No.: LB-5
Sample No.: R1
Soil Identification: Dark olive brown silt (ML)

Tested By: G. Bathala — Date: 01/23/21
Checked By: A. Santos — Date: 01/25/21
Sample Type: Ring
Depth (ft.): 2.5

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	162.90	166.42	174.09
Weight of Ring(gm):	38.40	37.77	38.17
Before Shearing			
Weight of Wet Sample+Cont.(gm):	152.00	152.00	152.00
Weight of Dry Sample+Cont.(gm):	145.03	145.03	145.03
Weight of Container(gm):	65.40	65.40	65.40
Vertical Rdg.(in): Initial	0.0000	0.0000	0.0000
Vertical Rdg.(in): Final	-0.0130	-0.0221	-0.0321
After Shearing			
Weight of Wet Sample+Cont.(gm):	195.30	191.14	208.30
Weight of Dry Sample+Cont.(gm):	171.89	170.31	187.12
Weight of Container(gm):	61.71	57.20	65.89
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-5**
Sample No.	**R1**
Depth (ft)	**2.5**

<u>Sample Type:</u>

Ring

<u>Soil Identification:</u>

Dark olive brown silt (ML)

Normal Stress (kip/ft²)	1.000	3.000	5.000
Peak Shear Stress (kip/ft²)	● 2.094	■ 3.477	▲ 4.128
Shear Stress @ End of Test (ksf)	○ 1.251	□ 2.210	△ 3.729
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.75	8.75	8.75
Dry Density (pcf)	95.2	98.4	103.9
Saturation (%)	30.7	33.1	38.0
Soil Height Before Shearing (in.)	0.9870	0.9779	0.9679
Final Moisture Content (%)	21.2	18.4	17.5

Leighton

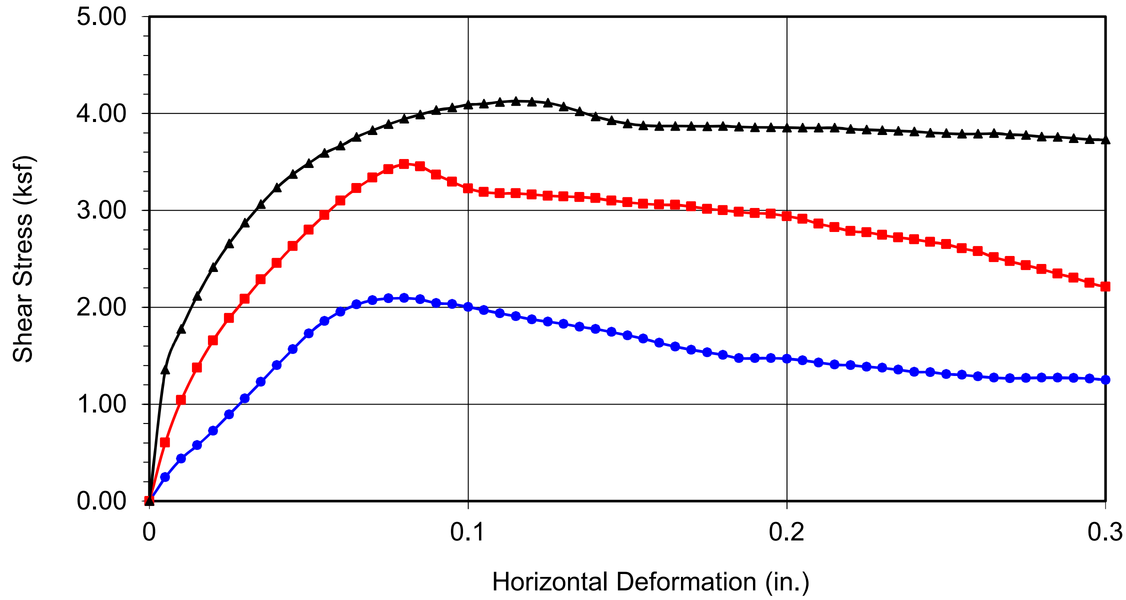
DIRECT SHEAR TEST RESULTS

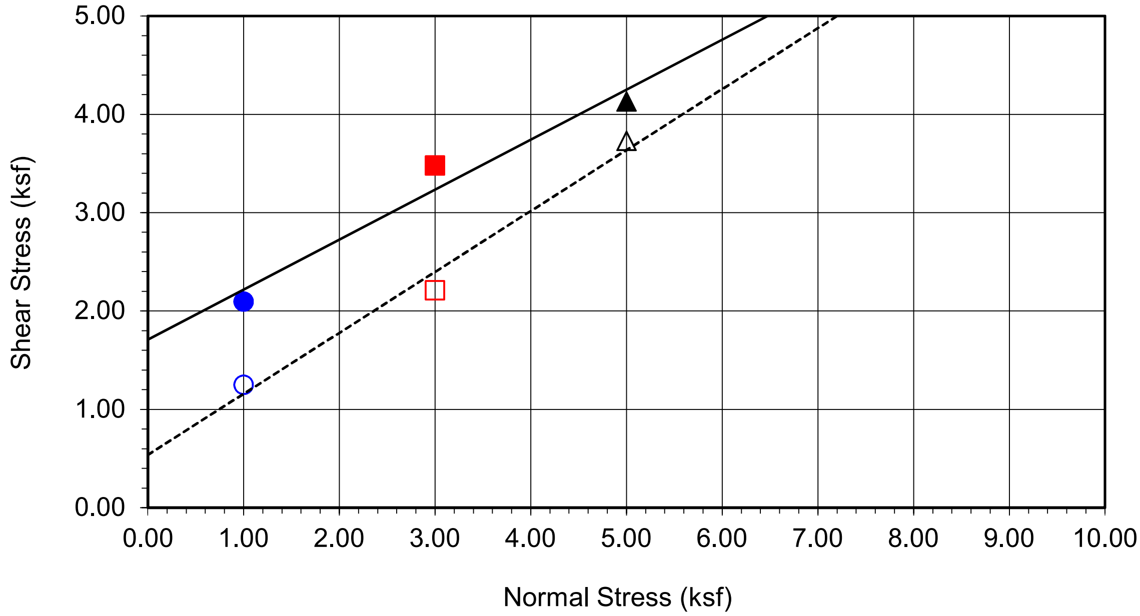
Consolidated Drained - ASTM D 3080

Project No.: 12809.001

Bridge/Torrance

01-21





Boring No.	**LB-5**
Sample No.	**R1**
Depth (ft)	**2.5**
Sample Type:	Ring

Soil Identification:
Dark olive brown silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	1708	27
Ultimate	538	32

Normal Stress (kip/ft²)	1.000	3.000	5.000
Peak Shear Stress (kip/ft²)	● 2.094	■ 3.477	▲ 4.128
Shear Stress @ End of Test (ksf)	○ 1.251	□ 2.210	△ 3.729
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.75	8.75	8.75
Dry Density (pcf)	95.2	98.4	103.9
Saturation (%)	30.7	33.1	38.0
Soil Height Before Shearing (in.)	0.9870	0.9779	0.9679
Final Moisture Content (%)	21.2	18.4	17.5

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12809.001

Bridge/Torrance

01-21