Appendix F

Geotechnical Exploration

GEOTECHNICAL EXPLORATION PROPOSED INDUSTRIAL WAREHOUSE DEVELOPMENT, ASSESSOR'S PARCEL NUMBER (APN'S) 0463-213-05, 06, 07, 08, 09, 16, 33, 34, 35, AND 46, SOUTHEAST OF CORDOVA ROAD AND DACHSHUND AVENUE, APPLE VALLEY, SAN BERNARDINO COUNTY, CALIFORNIA

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Project No. 13673.003

February 1, 2023

February 1, 2023

Project No. 13673.003

VVLIG Holdings, LLC
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Attention: Ms. Jessica Haughton
President

Subject: Geotechnical Exploration
Proposed Industrial Warehouse Development, Assessor's Parcel Number (APN's) 0463-213-05, 06, 07, 08, 09, 16, 33, 34, 35, and 46, Southeast of Cordova Road and Dachshund Avenue, Apple Valley, San Bernardino County, California

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has conducted geotechnical exploration for the proposed industrial warehouse development within Assessor's Parcel Number (APN's) 0463-213-05, 06, 07, 08, 09, 16, 33, 34, 35, and 46, located southeast of Cordova Road and Dachshund Avenue, in the Town of Apple Valley in San Bernardino County, California. This approximately 81-acre project site is currently undeveloped. The purpose of this study has been to collect surface and subsurface geotechnical data at the site with regard to the proposed development, to evaluate the proposed development with respect to site geotechnical conditions, and to provide geotechnical recommendations for design and construction of the proposed development.

Based on this geotechnical exploration, construction of the proposed warehouse 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 near the surface. 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.

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1.0 INTRODUCTION

1.1 Site Location and Description

The property is approximately 81 acres in area and is located southeast of Cordova Road and Dachshund Avenue, in the Town of Apple Valley, San Bernardino County, California. The project is within Assessor's Parcel Number (APN's) 0463-213-05, 06, 07, 08, 09, 16, 33, 34, 35, and 46.

The site is undeveloped, with relatively sparse desert scrub vegetation. Other than a small residential/commercial development located directly southwest of the site, the surrounding area is also undeveloped with dirt roads present. Based on our review of available historic aerial imagery dating back to 1952, the area has been historically undeveloped, with the adjacent residential/commercial development being present since sometime between 1984 and 1994.

Based on the elevation model of Google Earth and a review of available topographic maps, site elevations (El.) range from approximately El. 3,070 to 3,100 feet above mean sea level (msl). The site is relatively flat, with a gentle drainage gradient towards the west-southwest.

1.2 Proposed Development

Our understanding of this project is based on email correspondence with you on June 17, 2022, and the *Overall Site Plan* prepared by LHA, Inc., dated June 6, 2022. Based on these, we understand that the proposed industrial warehouse development will consist of a warehouse building with a footprint of 1,388,220 square feet (SF) and 258 dock doors, 561 auto parking stalls, 911 trailer stalls, drive aisles, and infiltration facilities. Based on the preliminary grading plans, we understand that the proposed development will include up to 8 feet of fill at the western portion of the site and up to 8 feet of design cuts will be required at the northern and eastern portions. Finished pad grade elevation is planned to be approximately El. 3086 feet above msl at the eastern end of the building sloping down to approximately 3076 feet above msl at the western end. A detailed site plan and structural loading were not available at the time of this report. We anticipate that the warehouse will be composed of concrete tilt-up walls.

1.3 Previous Work

Previous geotechnical exploration reports and environmental studies for this project were not available to Leighton for review during the preparation of this report. Leighton is not aware of previous earthwork activities onsite.

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, infiltration tests, laboratory testing, surface geologic mapping, seismic refraction surveys, 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 and geologic maps and reports and aerial photographs available from our in-house library, available online, or those provided by you.
- Utility Coordination: We contacted Dig Alert (811) prior to excavating borings so that utility companies could mark utilities onsite. We coordinated our work with you and a site representative.
- Field Exploration: A total of ten (10) hollow-stem auger borings (LB-1 through LB-10) were logged and sampled onsite on September 15 and 16, 2022 to evaluate subsurface conditions onsite. These borings were drilled by a subcontracted rig to depths ranging from approximately 20 to 30 feet below the existing ground surface (bgs). One geotechnical boring (LB-8) was targeted a depth of 50 feet, however drilling refusal was reached at approximately 30 feet due to gravels/cobbles. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a Modified California split barrel sampler lined with rings. Standard Penetration Tests (SPT) were conducted at

selected depths and samples were obtained at those intervals. Representative bulk soil samples were also collected at shallow depths from the borings.

Excavations were backfilled with soil cuttings. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Figure 2, *Geotechnical Map*.

We conducted well permeameter tests at two locations (IT-1 and IT-2) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. The well permeameter tests were conducted based on the USBR 7300-89 method and in general accordance with San Bernardino County guidelines. Tests consisted of constant head and falling head infiltration using a water truck to transport water to each location. A 2-inch diameter, slotted PVC pipe was used within each boring, with sand backfilled around the pipe within the test zone. The tests were conducted at depths ranging from 10 to 15 feet bgs. Infiltration test logs are included in Appendix B.

Multichannel Analysis of Surface Waves (MASW) was performed along a one-dimensional array within the site to determine the Shear Wave Velocity (V_S) distribution within the subsurface strata.

- <u>Geotechnical Laboratory Testing:</u> 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:

  - Maximum dry density and optimum moisture content
  - In situ moisture content and density
  - Sieve analysis for grain-size distribution
  - Collapse/swell-settlement
  - R-Value
  - Water-soluble sulfate concentration in the soil
  - Resistivity, chloride content and pH

  Laboratory tests are provided in Appendix C, Laboratory Test Results.

- 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 in the western Mojave Desert, in San Bernardino County California, and is part of the Mojave Desert geomorphic province, a broad interior region of isolated mountain ranges separated by broad desert plains and deep alluvial valleys. The Mojave province is wedged between the Garlock Fault (southern boundary of the Sierra Nevada) and the San Andreas Fault, where it bends northerly from its northwest trend. The northern boundary of the Mojave province is separated from the prominent Basin and Range by the eastern extension of the Garlock Fault.

The geology of the region consists of the following rock groups: i) Surficial sediments (Qa); ii) Older alluvial sediments (Qoa); iii) Granitic and dioritic rocks (qm); iv) Metamorphic rocks (ml, mq, and ms); and v) Metamorphosed quarts latite (mql). The Pre-Tertiary and Tertiary rocks are hard, consolidated materials forming the surrounding mountains and rocky buttes that rise from the valley floors and underlie the alluvium at depths. The valley soil profile consists of up to several hundreds to thousands of feet of fine- to coarse-grained alluvial deposits underlain by consolidated rocks. The alluvial deposits consist of late Pleistocene to Holocene age (5 million years old to recent) fine- to coarse-grained soil layers formed as a result of uplift and erosion of the surrounding mountains. Figure 3, *Regional Geology Map,* presents the site location in relation to the predominate geologic materials (alluvium) of the area. Figure 4, *Regional Fault and Historical Seismicity Map,* presents the site location in relation to active faults and epicenters of relatively large (> M_w 4.0) historical earthquakes.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by surficial sediments consisting of Quaternary alluvium (Qa). Alluvium encountered in our borings and observed at the surface generally consisted of Sand with Silt (SP-SM) and Silty Sand (SM). These soils generally consisted of very dense material below 2 feet bgs based on field Standard Penetration Test (SPT) blowcounts. The encountered soils were observed to be relatively consistent in the various borings throughout the site.

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, native soils are considered very slightly compressible.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Laboratory tests performed on a representative soil sample indicated a collapse of 2.5%. However, field standard penetration tests generally indicate that onsite granular soils are dense. Based on our overexcavation and compaction recommendations provided in Section 3.1 of this report, soil collapse and consolidation are not a significant issue at this site.

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.

Based on the granular nature of the soils encountered in our borings and observed at the surface, onsite soils are anticipated to have "very 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, yielding a sulfate content of 0.02 percent by weight. Based on this laboratory test result, the sulfate exposure from onsite soils is anticipated to be negligible (Exposure Class S0). Recommendations for concrete in contact with onsite soils 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, a representative soil sample was tested during this investigation to screen for minimum resistivity, chloride content, and pH. These tests indicated a minimum resistivity of 2,820 ohm-cm, chloride content of 60 ppm, and pH of 7.8. Based on these results, the onsite soil is considered to be moderately corrosive to buried ferrous metals.

2.3 Groundwater

Groundwater was not encountered within our exploratory borings performed on September 15 and 16, 2022.

Review of groundwater level data from the California Department of Water Resources indicated multiple groundwater wells approximately half to a mile from the site with measurements from 1953 through 1957. Although limited, these readings from these wells show that groundwater depths have been deeper than 70 feet bgs during the period of measurements. According to the *Data and Water Table Map of the Mojave River Ground-Water Basin* (Stamos and Predmore, 1995), groundwater levels in 1992 near the project site were at some depth between approximately 50 and 100 feet bgs.

Based on our review of available groundwater data, groundwater is not a significant constraint for this project.

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.

The project site has been mapped by the U.S. Geological Society (2022) to be outside of an area of land subsidence from intense removals of significant quantities of water, peat, or oil extraction in the area. Based on this and no known reports indicating land subsidence of the site's area, the potential for ground subsidence is considered very low and less than a significant impact.

2.4 Faulting and Seismicity

In general, 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 towards the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Helendale-South Lockhart fault zone	3.4 miles to the northeast
North Frontal thrust system	11.5 miles to the south
Lenwood Lockhart fault zone	17.6 miles to the northeast

Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered low.

2.4.2 Seismic Design Parameters

The site has and will 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 seismic parameters should be considered for design under the 2022 edition of the California Building Code (CBC). The following table lists seismic design parameters based on the 2022 CBC and ASCE 7-16 methodology:

Site Seismic Coefficients / Coordinates		Value (g)
Latitude: 34.6063	Longitude: -117.1943	
Site-Specific Analysis (ASCE 7-16)	Spectral Response – Class D (short), S_S	1.02
	Spectral Response – Class D (1 sec), S_1	0.39
	Site Modified Peak Ground Acceleration, PGA_M	0.53
	Max. Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.15
	Max. Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.58
	5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.77
	5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.39
	Maximum Considered Earthquake Geometric Mean MCE_G PGA	0.76

The project structural engineer should review the seismic parameters. Site-Specific analyses output is presented in Appendix D.

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 5.7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), with a corresponding peak ground acceleration of 0.51g.

2.4.5 Site Class

A geophysical survey line (Array 4) utilizing Multi-channel Analysis of Surface Wave (MASW) methodology was performed towards the central portion of the site (line location shown in Figure 2) and yielded a weighted average shear wave to a depth of 100 feet (V_{S100ft}) of 2,337 ft/s. In addition, we performed an analysis with field Standard Penetration Blowcounts (SPT) from the geotechnical borings that extended to a maximum depth of 30 feet, which yielded a weighted average N-Value of approximately 69 (with blowcount assumptions for soils below 30 feet). Therefore, based on the criteria in the 2022 CBC and ASCE 7-16, the site is classified as Site Class C, very dense soil and soft rock. A summary of Site Class evaluation is included in Appendix D. Geophysical survey data is included in Appendix E.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landslides, 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 due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,
(2) shallow groundwater, and
(3) strong, long-duration ground shaking

The site is mapped within a low liquefaction hazard zone of required investigation on the San Bernardino County General Plan (San Bernardino, 2009).

We have performed an analysis based on the modified Seed Simplified Procedure as detailed by Youd et al. (2001) and Martin and Lew (1999), which compares the seismic demand on a soil layer (Cyclic Stress Ratio, or CSR) to the capacity of the soil to resist liquefaction (Cyclic Resistance Ratio, or CRR), (Youd el al., 2001). A minimum required factor of safety of 1.3 was used in our analysis, with factor of safety defined as CRR/CSR. As required, our analysis assumes that the design earthquake would occur while the groundwater is at its estimated design level (historically highest level).

Due to the dense nature of the granular soils encountered and lack of shallow groundwater, liquefaction is not a significant hazard at this site.

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.

Based on the dense nature of the native soils in this area, we believe the onsite soils are susceptible to low seismic settlement (less than 1 inch, with differential settlement of 0.5 inch or less over a horizontal distance of 30 feet based on the MCE).

2.5.3 Lateral Spread

Lateral spread is liquefaction-induced lateral ground movement limited to on the order of several feet, and, thus, smaller than flow failures. A consideration in lateral spread analysis is to evaluate whether laterally continuous liquefiable layers exist. Due to the lack of liquefiable layers based on our analysis, lateral spread is considered to be less than significant.

2.5.4 Flow Failures

Based on $(N_1)_{60}$ values from the borings, lack of liquefiable soils, and the relatively flat nature of the site, the site is not considered susceptible to flow slides (large transitional or rotational failures), the most catastrophic form of liquefaction-induced ground failure.

2.5.5 Bearing Failures/Surface Manifestations

We performed an analysis of the potential for bearing failures/structural damage due to liquefaction (surface manifestations) based on the work of Ishihara (1995) and as described in Martin and Lew (1999). This method is based on empirical data and considers the thickness of non-liquefiable soil below the ground surface and foundations, compared to the thickness of underlying liquefiable soils. Due to the lack of liquefiable layers based on our analysis, latera spread is considered to be less than significant.

2.6 Infiltration Testing

Two well permeameter tests (LI-1 and LI-2) were conducted to estimate the infiltration rate at specific locations of the site. Test LI-1 was located north of the proposed building towards the center of the site, and test IT-2 was located towards the southern end of the site. The locations of the infiltration tests are based on the provided locations of the proposed detention basins in the site plan, with the northern location placed towards the center of the northern portion to target native alluvial soils in the northern region. The well permeameter tests were conducted inside the drilled borings at depths of approximately 10 and 15 feet below the 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 is placed in the boring bottom to support temporary perforated well casing pipe. In addition, sand 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. The volume of water percolated during timed intervals is converted into an incremental infiltration rate, which is defined as flow divided by infiltration surface area, in inches per hour. The test was conducted based on the USBR 7300-89 test method.

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)
LI-1	10 to 15	Silty Sand (17% fines)	0.4
LI-2	10 to 15	Sand with silt (9% fines)	2.5

[1] Factor of Safety should be applied to raw rates

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed warehouse 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 warehouses. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking. 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.

We are unaware of environmentally sensitive areas in the project site that would warrant remedial removals from an environmental standpoint. Undocumented fill, if encountered, should be completely removed and properly compacted during earthwork construction. Localized exposures of encountered fill material can be evaluated during grading on a case-by-case basis, and may be left in place if documentation is available and the material appears to be competent based on our field evaluation

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix F, unless specifically 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.

3.1.2 Removal of Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of onsite alluvial soil, if uncontrolled artificial fill is encountered during grading, it should be completely removed and may be used as compacted fill for the project, provided any oversized rock is suitably handled and any deleterious materials are removed. Undocumented fill was not encountered in our subsurface exploration.

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 undocumented artificial fill within the proposed building pad, if encountered during grading, should be removed.

Based on our seismic settlement analysis, we recommend that onsite soils in the proposed building pad area and site walls taller than 8 feet be overexcavated to a minimum depth of 3 feet bgs, or a depth of 2 feet below the bottoms of proposed footings, whichever is deeper.

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. Where this is not achievable, this should be reviewed on a case-by-case basis.

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 proposed structures and planned for new asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), site walls up to 8 feet tall and retaining walls retaining up to 3 feet of soil (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 2 feet 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 D1557 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 24 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 10 +/- 3 percent
Subsidence (overexcavation bottom processing)	Approximately 0.2 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 4,500 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 260 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 to be 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 estimated to be 0.5 inch over a horizontal distance of 30 feet for the design-level earthquake, or angular distortion of less than 0.0014L.

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.

- <u>Concrete Thickness and Reinforcement in Warehouse/Industrial Areas:</u> 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 14 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 1,100 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 Section 2.4.2 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 5 (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	35 pcf
At-Rest	55 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.

For retaining walls with a retained height of more than 6 feet, an incremental seismic load applied as a uniform additive pressure of 17 pcf should be considered for a cantilever (unrestrained) wall with level backfill, and 27 pcf for a basement wall (restrained) with level backfill. This pressure is in addition to the static active earth pressures presented above. Earthquake and at-rest earth pressures need not be combined for analyses.

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 Pavement: Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using a design R-value of 32 based on laboratory testing, 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 (auto access)	3.0	5.0
7 (light truck access)	4.0	9.0
8	5.0	10.0
9	5.5	12.0

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 14 feet on center each way for 7-inch-thick concrete, and 12 feet for 5-inch-thick concrete. 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 Infiltration Recommendations

In general, our geotechnical exploration encountered surficial sediment deposits generally uniform consisting of granular materials Sand with Silt (SP-SM) and Silty Sand (SM). Soils were relatively uniform throughout the project site. At our test locations, sieve analysis tests performed on soil samples from the infiltration test zone generally showed a percent fines (% silt and clay) ranging from 9 to 17 percent.

Based on our infiltration testing, field observations and laboratory testing, the project site is considered to be feasible for groundwater infiltration. A raw infiltration rate of 2.5 inches per hour may be utilized for infiltration system design for the southern portion of the site. However, the infiltration test on the northern portion of the site yielded poor rates and should not be relied upon for infiltration. As site layout and infiltration system design progresses, supplemental infiltration testing could be performed to further refine these infiltration system recommendations.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with San Bernardino County guidelines, since monitoring of actual facility performance has shown that actual infiltration rates are lower than measured in small-scale tests. Infiltration basins are subject to siltation, which can result in reduced infiltration rates. *This small-scale infiltration rate should be divided by a design factor of at least 3 for buried chambers and at least 4 for open basins; although the design/safety factor may be higher based on project-specific aspects.* It should be noted that during periods of prolonged precipitation, underlying soils tend to become saturated to greater depths/extent. Therefore, infiltration rates tend to decrease with prolonged rainfall.

Some design considerations are presented in the following paragraphs:

- **Adjacent Structure Impact**: As infiltrating water can seep within soil strata partially horizontally, it is important to consider impact that infiltration facilities can play on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these facilities. Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process, but a building setback of at least 15 feet horizontally is initially suggested.

- **Infiltration Basins Type and Geometry**: Further testing may be required depending on final design of infiltration facilities. Infiltration rates are anticipated to vary based on location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. We should review all infiltration plans, including locations and depths of proposed facilities. Further testing may be required depending on infiltration facilities design details, particularly considering type, depth and location.

- **Siltation and Soil Changes**: These infiltration rates are for a clean, un-silted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of soil particles, gradation (uniform versus well graded), particle shape, fines content and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill. For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

- **De-silting Weir/Facilities**: Periodic flow of water carrying sediments into the basin or chamber, plus deposition of fine wind-blown sediments and sediments from erosion of basin side walls, will eventually cause the basin bottom or chamber to accumulate a layer of silt, which has the potential to significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape onsite. 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 be constructed with spillways or other appropriate means that would prevent overfilling that could damage the facility or adjacent improvements.

- **Drainage/Infiltration Time Cycle**: In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with 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. In open basins with compacted or silty bottoms, this could be problematic, in that even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating prolonged open-water safety concern (such as potential for mosquitos and waterborne diseases, algae odor, etc.). In a buried/cover infiltration chamber, these conditions would be of less concern.

- **Maintenance**: Infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented if and as needed. Things to check for include removal of trash or dumping, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained as recommended by the manufacturer or designer. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed.

3.8 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.

3.9 Trench Backfill

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, organic 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 and will allow water to freely permeate. Gravel or rock should not be used for trench backfill without written approval by Leighton. If gravel or open-graded rock is approved and used as bedding or shading, it should be wrapped in Mirafi 140N filter fabric, or equivalent, to prevent surrounding soil from washing into the pore spaces in the gap-graded rock. Shading should extend at least 12 inches above the top of the pipe. The bedding/shading materials should be densified in-place by mechanical means, or in accordance with Greenbook specifications.

Subsequent to pipe bedding and shading, backfill soils should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction (ASTMS D1557). The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook). The upper 6 inches in pavement areas should be compacted to 95 percent compaction.

3.10 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.

3.11 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 moderately 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.

3.12 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 VVLIG Holdings, LLC, for application to the design of the proposed warehouse buildings 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

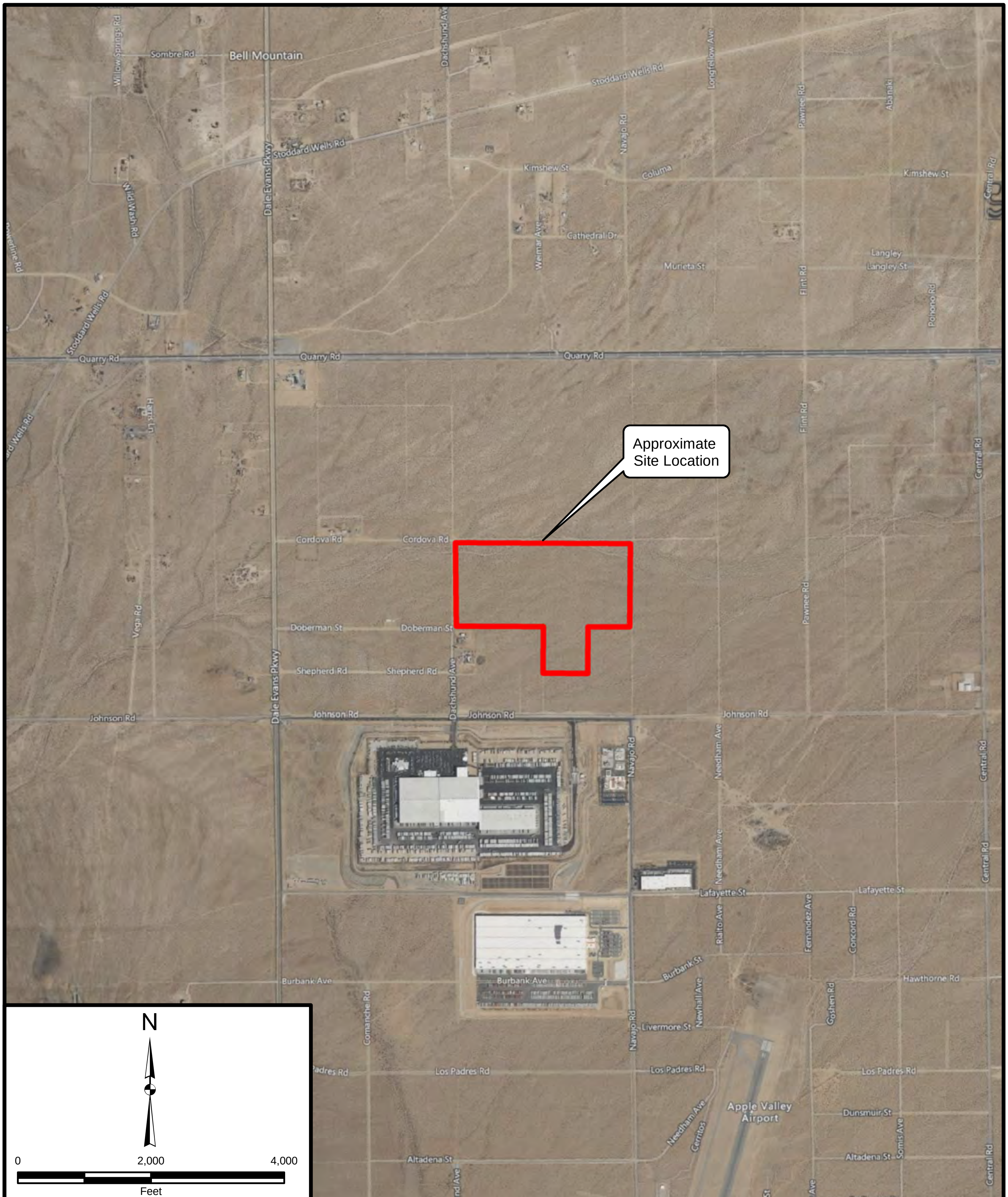
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate 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.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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



Project: 13673.003	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: February 2023

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

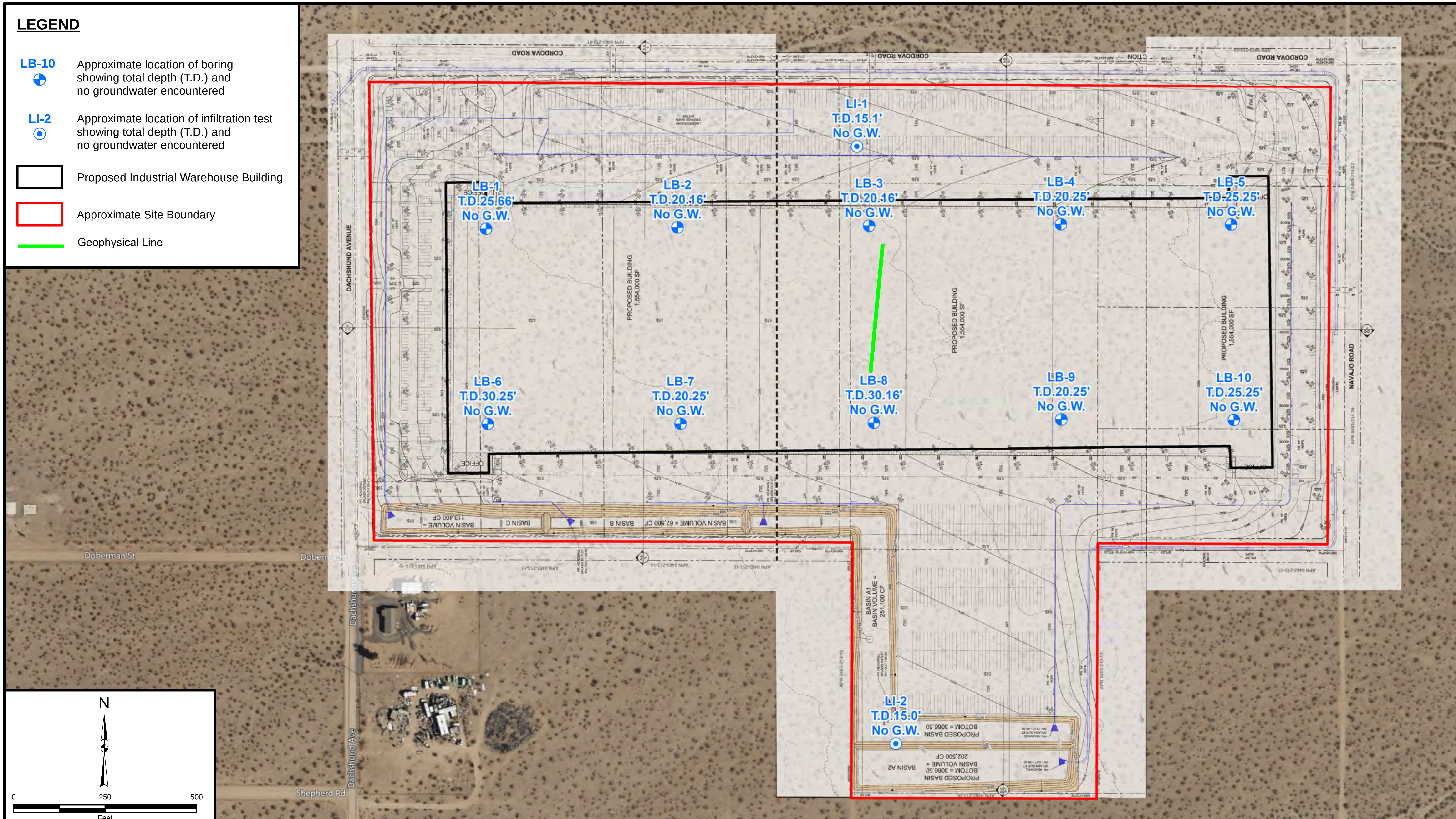
SITE LOCATION MAP

Proposed Industrial Warehouse Building Development
Southeast of Cordova Road and Dachshund Avenue
Apple Valley, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\13673\003\Maps\13673-003_F01_SLM_2022-10-04.mxd on 10/4/2022 10:22:45 AM Author: KVM (btran)



Project: 13673.003	Eng/Geol: JDH/SGO
Scale:1 " = 250 '	Date: February 2023

Reference: Site Plan Review, Conceptual Grading and Drainage Sheets 2-4 of 10 by David Evans and Associates, Inc.

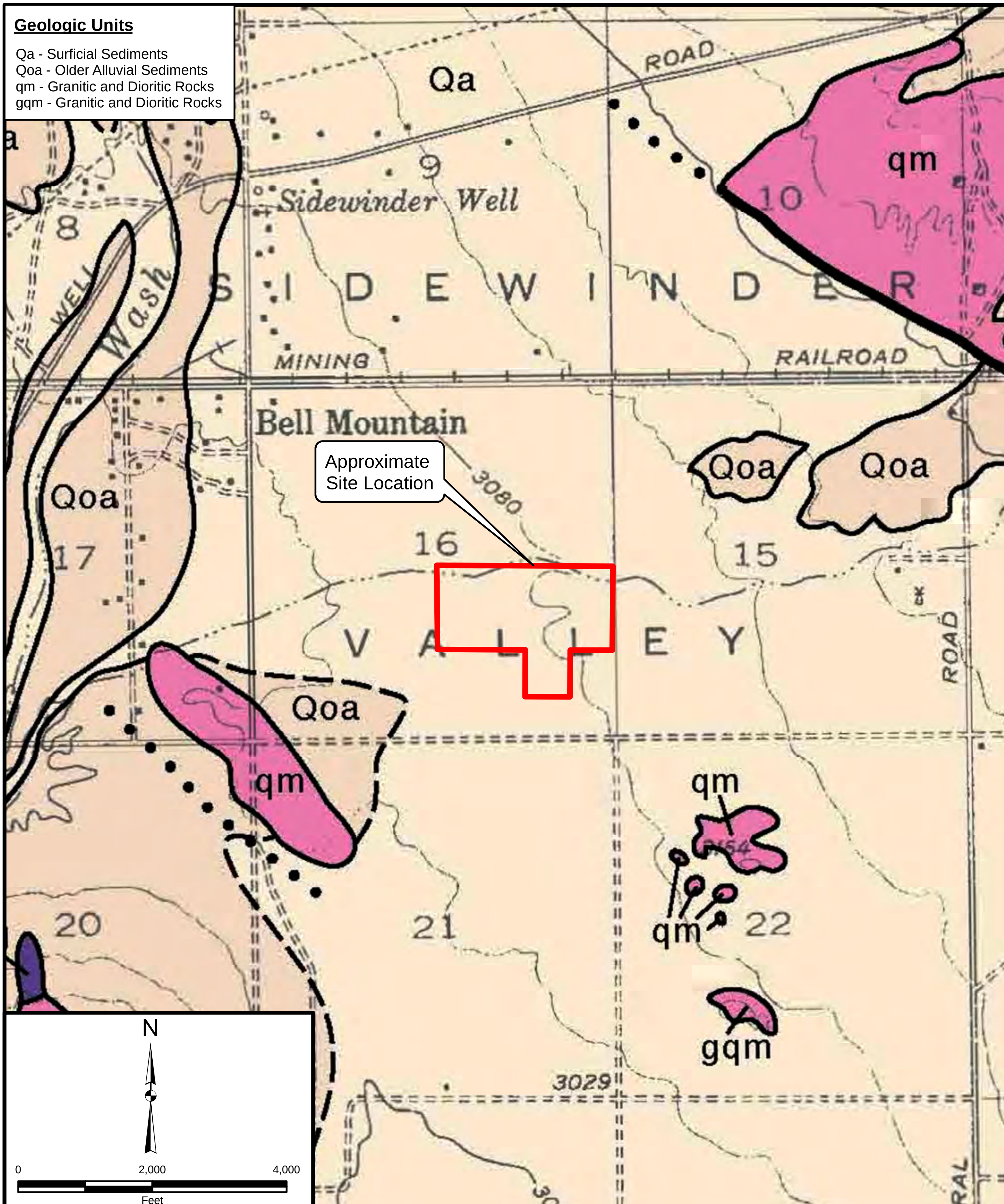
GEOTECHNICAL MAP

Proposed Industrial Warehouse Building Development
Southeast of Cordova Road and Dachshund Avenue
Apple Valley, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\13673\003\Maps\13673-003_F02_ELM_2022-11-21.mxd on 11/21/2022 10:08:12 AM Author: KVM (btran)



Project: 13673.003	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: February 2023

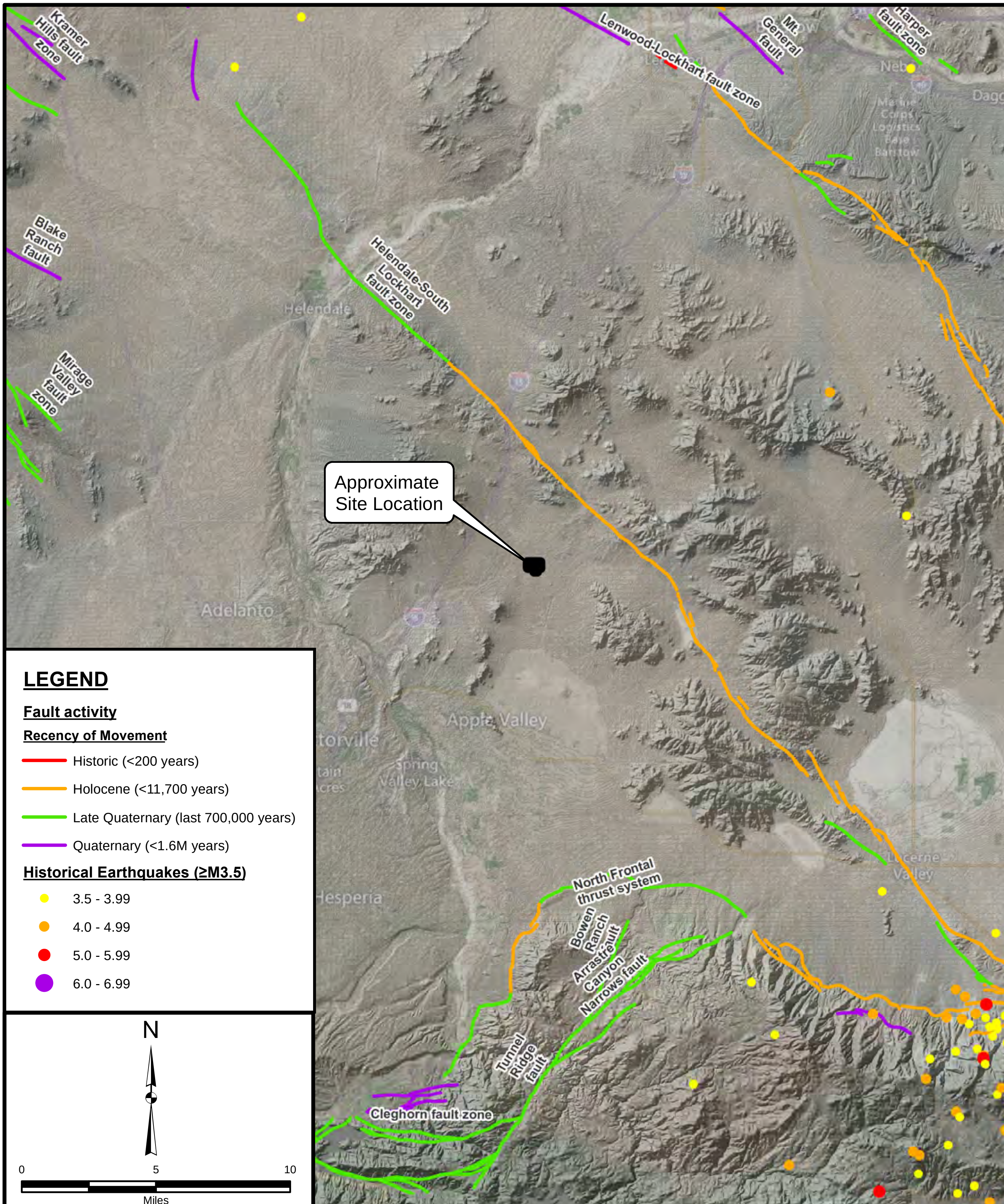
Reference:
Geologic Map of the Apple Valley & Old Mountains
by Thomas W. Dibblee, JR., 2008

REGIONAL GEOLOGY MAP

Proposed Industrial Warehouse Building Development
Southeast of Cordova Road and Dachshund Avenue
Apple Valley, California

FIGURE 3

Leighton



Project: 13673.003	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: February 2023

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

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Industrial Warehouse Building Development
Southeast of Cordova Road and Dachshund Avenue
Apple Valley, California

FIGURE 4

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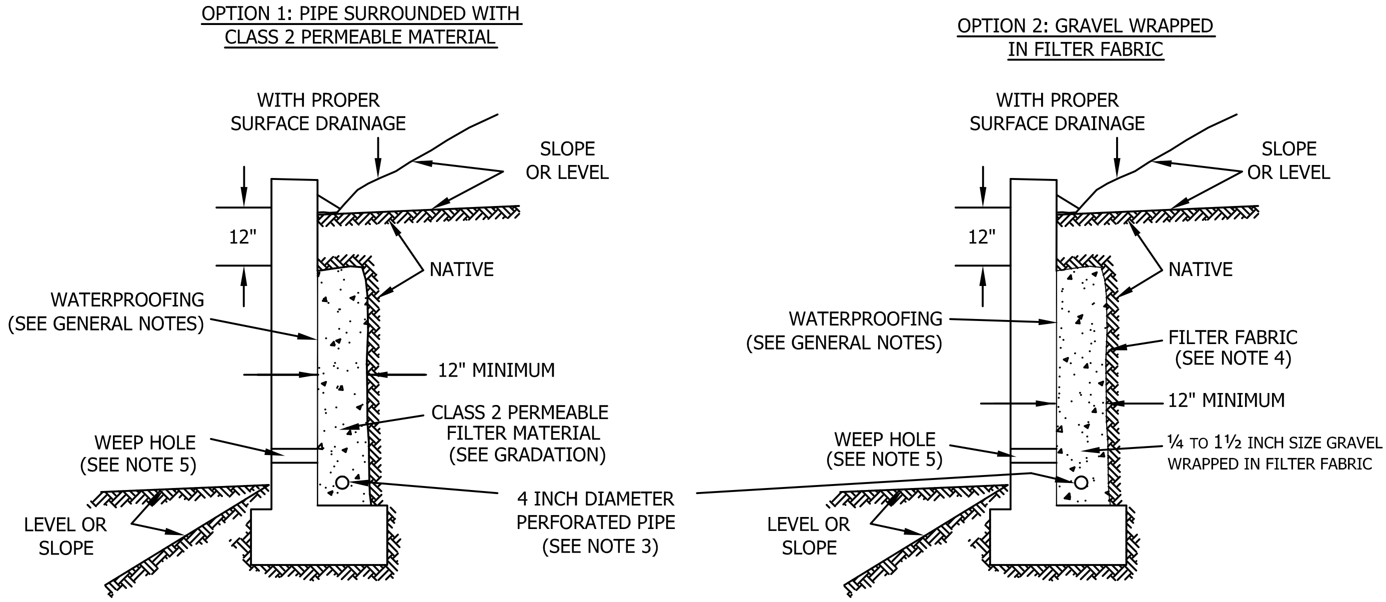
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

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FIGURE 5

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

APPENDIX B

FIELD EXPLORATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration. Approximate exploration locations are shown on Figure 2, *Geotechnical Map*.

Borings: On September 15 and 16, 2022, 12 hollow-stem-auger borings (LB-1 through LB-10, LI-1 and LI-2) were drilled, logged and sampled to depths ranging from 20 feet to 30 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 Modified 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.	13673.003	**Date Drilled**	9-16-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3079'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
				R-1	24 32 40	116	1	SP-SM	@2.5': WELL GRADED SAND with silt (SW-SM): dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (lab)	-200
3075	5			R-2	12 50/6"			SW-SM	@5': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
				R-3	26 47 43	114	6	SM	@7.5': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3070	10			R-4	19 38 50/4"			SP-SM	@10': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3065	15			R-5	50/3"			SP-SM	@15': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 5% fines (field estimate)	
3060	20			S-1	29 50/4"			SP-SM	@20': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 5% fines (field estimate)	
3055	25			R-6	44 50/2"			SP-SM	@25': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 5% fines (field estimate)	
									TOTAL EXPLORED DEPTH = 25.66 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3050	30									

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

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

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GEOTECHNICAL BORING LOG LB-10

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3103'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SP-SM	@Surface: SAND with silt (SP-SM)	
3100				R-1	36 50/6"	110	4	SM	@2.5': SILTY SAND (SM): very dense, brown, dry to moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
	5			R-2	25 50/5"			SM	@5': SILTY SAND (SM): very dense, brown, dry to moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3095				R-3	15 32 50/3"	116	3	SM	@7.5': SILTY SAND (SM): very dense, brown, dry to moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
	10			R-4	50/6"			SP-SM	@10': SAND with silt (SP-SM): very dense, brown, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3090										
	15				50/4"				@15': SAND with silt (SP-SM): very dense, gray, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3085										
	20				50/2"				@20': SAND with silt (SP-SM): very dense, gray, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3080										
	25				50/3"				@25': SAND with silt (SP-SM): very dense, gray, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3075										
	30									

TOTAL EXPLORED DEPTH = 25.25 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH SOIL CUTTINGS TO SURFACE

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

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

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GEOTECHNICAL BORING LOG LB-2

Project No. 13673.003
Project Synergy Warehouses - Cordova Drive
Drilling Co. 2R Drilling, Inc.
Drilling Method Hollow Stem Auger - 140lb - Autohammer
Location See Figure 2 - Geotechnical Map
Date Drilled 9-16-22
Logged By JP
Hole Diameter 8"
Ground Elevation 3085'
Sampled By JP

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
3085	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
				R-1	30 50/6"	108	3	SM	@2.5': SILTY SAND (SM): very dense, reddish brown, dry to moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3080	5			R-2	9 50/4"	98	4	SP-SM	@5': SAND with silt (SP-SM): very dense, grayish brown, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
				R-3	50/4"			SP-SM	@7.5': SAND with silt (SP-SM): very dense, grayish brown, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate) Poor Recovery	
3075	10			R-4	50/4"			SP-SM	@10': SAND with silt (SP-SM): very dense, grayish brown, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3070	15			S-1	50/4"			SP-SM	@15': SAND with silt (SP-SM): very dense, grayish brown, dry to moist, fine to coarse sand, fine gravel, 8% fines (lab)	-200
3065	20			R-5	50/2"			SP-SM	@20': Spoils: SAND with silt (SP-SM): very dense, reddish brown, dry to moist, fine to coarse sand, fine gravel, 10% fines (field estimate) **TOTAL EXPLORED DEPTH = 20.16 FEET NO GROUNDWATER ENCOUNTERED BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3060	25									
3055	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

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GEOTECHNICAL BORING LOG LB-3

Project No.	13673.003	**Date Drilled**	9-16-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3089'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SP-SM	**<u>Quaternary Alluvium(Qa)</u>** @Surface: SAND with silt (SP-SM)	
				R-1	4 48 11	110	2	SM	@2.5': SILTY SAND (SM): dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3085	5			R-2	10 32 50/3"	125	2	SM	@5': SILTY SAND (SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 17% fines (lab)	-200
					50/4"				@7.5': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3080	10			50/4"	50/4"			SP-SM	@10': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3075	15				13 50/2"				@15': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3070	20				50/2"				@20': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3065	25									
3060	30									

TOTAL EXPLORED DEPTH = 20.16 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH SOIL CUTTINGS TO SURFACE

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

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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13673.003	**Date Drilled**	9-16-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3096'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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			B-1				SP-SM	**Quaternary Alluvium(Qa)**	
3095									@Surface: SAND with silt (SP-SM)	
				R-1	7 16 44	119	4	SM	@2.5': SILTY SAND (SM): dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
	5			R-2	5 25 25	142	2	SM	@5': SILTY SAND (SM): medium dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3090										
					50/2"				@7.5': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
	10			R-3	50/3"			SP-SM	@10': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3085										
	15			S-1	50/3"			SP-SM	@15': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3080										
	20				50/3"				@20': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3075									**TOTAL EXPLORED DEPTH = 20.25 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	25									
3070										
	30									

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

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

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GEOTECHNICAL BORING LOG LB-5

Project No.	13673.003	**Date Drilled**	9-16-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3103'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
3100				R-1	5 36 50/3"	120	4	SP-SM	@2.5': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 5-7% fines (field estimate)	
	5			R-2	20 26 50/2"			SP-SM	@5': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 8% fines (lab)	-200
3095				R-3	23 25 50/6"	119	2	SP-SM	@7.5': SAND with silt (SP-SM): dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 5-7% fines (field estimate), fragments of broken cobble	CO
	10			R-4	40 50/3"	103	2	SP-SM	@10': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 5-7% fines (field estimate)	
3090										
	15				50/2"				@15': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 5-7% fines (field estimate)	
3085										
	20			S-1	50/3"			SP-SM	@20': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 5-7% fines (field estimate) **TOTAL EXPLORED DEPTH = 20.25 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3080										
	25									
3075										
	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

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

GEOTECHNICAL BORING LOG LB-6

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3075'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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
3075	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
				R-1	3 4 20			SP-SM	@2.5': SAND with silt (SP-SM): medium dense, brown, slightly moist, fine to medium sand, fine gravel, 12% fines (field estimate)	
3070	5			R-2	30 50/3"	106	5	SP-SM	@5': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
				R-3	24 50/4"			SP-SM	@7.5': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3065	10				40 50/2"				@10': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3060	15			S-1	50/3"			SP-SM	@15': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3055	20			R-4	50/3"	94	4	SM	@20': SILTY SAND (SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 13% fines (lab)	-200
3050	25				50/5"				@25': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
3045	30									

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

TYPE OF TESTS:

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

Leighton

GEOTECHNICAL BORING LOG LB-6

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3075'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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.	
3045	30				50/3"				@30': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 10% fines (field estimate)	
									TOTAL EXPLORED DEPTH = 30.25 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3040	35									
3035	40									
3030	45									
3025	50									
3020	55									
3015	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

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GEOTECHNICAL BORING LOG LB-7

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3083'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
3080				R-1	7 26 25	111	3	SM	@2.5': SILTY SAND (SM): dense, brown, slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
	5			R-2	14 50/6"	105	10	SM	@5': SILTY SAND (SM): very dense, reddish brown, slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate), carbonate lenses	
3075				R-3	45 50/3"			SP-SM	@7.5': SAND with silt (SP-SM): very dense, grayish brown, slightly moist, fine to coarse sand, fine gravel, 5% fines (lab)	-200
	10			R-4	50/3"			SP-SM	@10': SAND with silt (SP-SM): very dense, grayish brown, slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3070										
	15			R-5	50/4"	114	3	SM	@15': SILTY SAND (SM): very dense, grayish brown, slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3065										
	20				50/3"				@20': SAND with silt (SP-SM): very dense, light gray, dry to slightly moist, fine to coarse sand, fine gravel, 5% fines (field estimate)	
									TOTAL EXPLORED DEPTH = 20.25 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3060										
	25									
3055										
	30									

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

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

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GEOTECHNICAL BORING LOG LB-8

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3089'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	MD, CR, SA, RV
3085				R-1	10 21 33	104	4	SM	@2.5': SILTY SAND (SM): dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 31% fines (lab)	
	5			R-2	8 28 26	117	3	SM	@5': SILTY SAND (SM): dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 31% fines (lab)	
3080				R-3	50/5"			SP-SM	@7.5': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
	10			R-4	50/3			SP-SM	@10': SAND with silt (SP-SM): very dense, grayish brown, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3075	15				50/3"				@15': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3070	20			R-5	50/4"	165	1	SP-SM	@20': SILTY SAND (SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3065	25			S-1	50/1"			SM	@25': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3060	30									

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

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

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GEOTECHNICAL BORING LOG LB-8

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3089'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30				50/2"				@30': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
									TOTAL EXPLORED DEPTH = 30.16 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3055	35									
3050	40									
3045	45									
3040	50									
3035	55									
3030	60									

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

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

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

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GEOTECHNICAL BORING LOG LB-9

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3098'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
3095				R-1	5 6 8	111	2	SM	@2.5': SILTY SAND (SM): loose, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
	5			R-2	3 4 12			SM	@5': SILTY SAND (SM): loose, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate)	
3090				R-3	7 16 43	106	10	SM	@7.5': SILTY SAND (SM): dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 15-20% fines (field estimate), trace carbonate lenses	
	10			R-4	33 50/5"			SP-SM	@10': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3085										
	15			S-1	50/5"			SP-SM	@15': SAND with silt (SP-SM): very dense, reddish brown, dry to slightly moist, fine to coarse sand, fine gravel, 9% fines (lab)	-200
3080										
	20				50/3"				@20': SAND with silt (SP-SM): very dense, gray, dry to slightly moist, fine to coarse sand, fine gravel, 7% fines (field estimate)	
3075										
	25									
3070										
	30									

TOTAL EXPLORED DEPTH = 20.25 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH SOIL CUTTINGS TO SURFACE

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

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

Leighton

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GEOTECHNICAL BORING LOG LI-1

Project No.	13673.003	**Date Drilled**	9-16-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3087'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
3085										
	5									
3080										
	10			S-1	29 50/6"			SP-SM	@10': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 7-10% fines (field estimate)	
3075										
				S-2	36 50/5"			SM	@13.5': SILTY SAND (SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 17% fines (lab)	SA
	15								**TOTAL EXPLORED DEPTH = 15.1 FEET** **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @ 15.1 FT**	
3070										
	20									
3065										
	25									
3060										
	30									

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.

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

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

Leighton

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GEOTECHNICAL BORING LOG LI-2

Project No.	13673.003	**Date Drilled**	9-15-22
Project	Synergy Warehouses - Cordova Drive	**Logged By**	JP
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	3083'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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
3083	0			B-1				SP-SM	**Quaternary Alluvium(Qa)** @Surface: SAND with silt (SP-SM)	
3080										
	5									
3075										
	10			S-1	29 50/6"			SP-SM	@10': SAND with silt (SP-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 7-10% fines (field estimate)	
3070										
				S-2	36 50/5"			SW-SM	@13.5': SAND with silt (SW-SM): very dense, brown, dry to slightly moist, fine to coarse sand, fine gravel, 9% fines (lab)	SA
	15								**TOTAL EXPLORED DEPTH = 15 FEET** **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @ 15 FT**	
3065										
	20									
3060										
	25									
3055										
	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
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Results of Well Permeameter, from USBR 7300-89 Method

Project: 13673.003
Exploration #/Location: *LI-1*
Depth Boring drilled, bgs (ft): 15
Tested by: AA
USCS Soil Type in test zone: SM / SP-SM
Weather (start to finish): Sunny
Water Source/pH: H2O
Measured boring diameter: 8 in. — 4 in. Well Radius
Depth to GW or aquitard, bgs: 100 ft
Well Prep: Drill to 15', bottom 10' screen pipe, sand backfilll in test zone

Initial estimated Depth to Water Surface (in.): 133
Average depth of water in well, "h" (in.): 51
approx. h/r: 12.7
Tu (Fig. 8) (ft): 88.9
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity: 0.4
Casing outer diameter, in.: 2.3
Casing inner diameter, in.: 2.1
Cross-sectional area, in.^2: 21.9

Use of Barrels: No
Use of Flow Meter: Yes
Test Type: Constant Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surfa	15. ft	3.5 in.	184
Casing stickup measured above top of auger (or ground surface) (+ i	0. ft	0. in.	0
Depth to top of sand from top of casing			

Depth of well bottom below top of casing (in): 184

Flow Meter ID: 2497 Meter Units: Gallons 0.05 gallons/pulse
Data logger ID:

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or 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)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/28/2022	Start time: 11:35	Gallons																				
9/28/22	11:35	1677.77		11.9					0	142.8	40.7											
9/28/22	11:37	1677.95		11.85				2	2	142.2	41.3	0.6	41	42	-13	28	14	853	1081	0.9	0.16	0.73
9/28/22	11:40	1678.23		11.78				3	5	141.4	42.1	0.84	42	65	-18	46	15	926	1099	0.9	0.16	0.78
9/28/22							Adjust Flow															
9/28/22	11:42	1678.3		11.82					7	141.8	41.7											
9/28/22	11:52	1678.92		11.67				10	17	140.0	43.5	1.8	43	143	-39	104	10	623	1120	0.9	0.10	0.51
9/28/22	12:02	1679.57		11.5				10	27	138.0	45.5	2.04	44	150	-45	105	11	633	1168	0.9	0.10	0.50
9/28/22	12:12	1680.21		11.26				10	37	135.1	48.4	2.88	47	148	-63	85	8	508	1230	0.9	0.07	0.38
9/28/22	12:21	1680.8		11.08				9	46	133.0	50.5	2.16	49	136	-47	89	10	593	1293	0.9	0.08	0.42
9/28/22	12:31	1681.44		10.87				10	56	130.4	53.1	2.52	52	148	-55	93	9	556	1352	0.9	0.07	0.38
9/28/22	12:42	1682.16		10.6				11	67	127.2	56.3	3.24	55	166	-71	95	9	520	1425	0.9	0.06	0.34
9/28/22	12:51	1682.76		10.45				9	76	125.4	58.1	1.8	57	139	-39	99	11	661	1488	0.9	0.07	0.41
9/28/22							Adjust Flow															
9/28/22	12:57	1682.82		10.62			(slow for readings)		82	127.4	56.1											
9/28/22	13:00	1682.86		10.54			(slow for readings)	3	85	126.5	57.0	0.96	57	9	-21	-12	-4	-236	1471	0.9	-0.03	-0.15
9/28/22	13:09	1682.87		10.54			(slow for readings)	9	94	126.5	57.0	0	57	2	0	2	0	15	1483	0.9	0.00	0.01
9/28/22	13:20	1683.03		10.52			(slow for readings)	11	105	126.2	57.3	0.24	57	37	-5	32	3	173	1486	0.9	0.02	0.11
9/28/22							Switch to Falling Head															
9/28/22	13:36			9.5					121	114.0	69.5											
9/28/22	13:38			9.54				2	123	114.5	69.0	-0.48	69	0	11	11	5	315	1791	0.9	0.03	0.16
9/28/22	13:40			9.58				2	125	115.0	68.5	-0.48	69	0	11	11	5	315	1779	0.9	0.03	0.16
9/28/22	13:44			9.75				4	129	117.0	66.5	-2.04	68	0	45	45	11	670	1747	0.9	0.06	0.35
9/28/22	13:54			10.73				10	139	128.8	54.7	-11.76	61	0	258	258	26	1546	1574	0.9	0.19	0.91
9/28/22	13:59			11.07				5	144	132.8	50.7	-4.08	53	0	89	89	18	1073	1375	0.9	0.14	0.72
9/28/22	14:05			11.58				6	150	139.0	44.5	-6.12	48	0	134	134	22	1341	1247	0.9	0.22	0.99
9/28/22	14:10			11.98				5	155	143.8	39.7	-4.8	42	0	105	105	21	1262	1109	0.9	0.25	1.05
9/28/22	14:15			12.3				5	160	147.6	35.9	-3.84	38	0	84	84	17	1010	1001	0.9	0.23	0.93
9/28/22	14:21			12.56				6	166	150.7	32.8	-3.12	34	0	68	68	11	684	913	0.9	0.18	0.69
9/28/22	14:30			12.65				9	175	151.8	31.7	-1.08	32	0	24	24	3	158	861	0.9	0.04	0.17
9/28/22																						
9/28/22																						
																			Minimum Rate:			0.3
														Raw Rate for design, prior to application of adjustment factors:								**0.4**

Results of Well Permeameter, from USBR 7300-89 Method

Leighton

Project: 13673.003
Exploration #/Location: *LI-2*
Depth Boring drilled, bgs (ft): 15
Tested by: AA
USCS Soil Type in test zone: SM / SP-SM
Weather (start to finish): Sunny
Water Source/pH: H2O
Measured boring diameter: 8 in. 4 in. Well Radius
Depth to GW or aquitard, bgs: 100 ft
Well Prep: Drill to 15', bottom 10' screen pipe, sand backfilll in test zone

Initial estimated Depth to Water Surface (in.): 155
Average depth of water in well, "h" (in.): 25
approx. h/r: 6.3
Tu (Fig. 8) (ft): 87.1
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity 0.4
Casing outer diameter, in. 2.3
Casing inner diameter, in. 2.1
Cross-sectional area, in.^2 21.9

Use of Barrels: No
Use of Flow Meter: Yes
Test Type: Constant Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surfa	15. ft	1.5 in.	182
Casing stickup measured above top of auger (or ground surface) (+ i	0. ft	1.5 in.	1.5
Depth to top of sand from top of casing			

Depth of well bottom below top of casing (in): 183

Flow Meter ID: 2497 Meter Units: Gallons 0.05 gallons/pulse
Data logger ID:

Field Data **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or 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)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/29/2022	Start time: 9:12	Gallons																				
9/29/22	9:12	1694.11		13.5					0	160.5	21.0											
9/29/22	9:14	1694.46		13.47				2	2	160.1	21.4	0.36	21	81	-8	73	36	2189	583	0.9	1.08	3.46
9/29/22	9:19	1695.34		13.32				5	7	158.3	23.2	1.8	22	203	-39	164	33	1966	610	0.9	0.85	2.97
9/29/22	9:24	1696.28		13.27			Adjust Flow	5	12	157.7	23.8	0.6	23	217	-13	204	41	2448	640	0.9	1.03	3.53
9/29/22	9:34	1697.98		13.18				10	22	156.7	24.8	1.08	24	393	-24	369	37	2214	661	0.9	0.87	3.09
9/29/22	9:45	1699.91		13.1				11	33	155.7	25.8	0.96	25	446	-21	425	39	2317	687	0.9	0.86	3.11
9/29/22	9:55	1701.65		13.04				10	43	155.0	26.5	0.72	26	402	-16	386	39	2317	708	0.9	0.83	3.02
9/29/22	10:05	1703.4		13				10	53	154.5	27.0	0.48	27	404	-11	394	39	2362	723	0.9	0.83	3.01
9/29/22																						
9/29/22	10:10			11.3					58	134.1	47.4											
9/29/22	10:11			11.69				1	59	138.8	42.7	-4.68	45	0	103	103	103	6152	1183	0.9	1.09	4.79
9/29/22	10:12			12.15				1	60	144.3	37.2	-5.52	40	0	121	121	121	7256	1055	0.9	1.61	6.34
9/29/22	10:13			12.39				1	61	147.2	34.3	-2.88	36	0	63	63	63	3786	949	0.9	0.94	3.68
9/29/22	10:14			12.64				1	62	150.2	31.3	-3	33	0	66	66	66	3943	875	0.9	1.13	4.15
9/29/22	10:16			12.93				2	64	153.7	27.8	-3.48	30	0	76	76	38	2287	794	0.9	0.79	2.66
9/29/22	10:18			13.23				2	66	157.3	24.2	-3.6	26	0	79	79	39	2366	705	0.9	1.01	3.10
9/29/22	10:20			13.5				2	68	160.5	21.0	-3.24	23	0	71	71	35	2129	619	0.9	1.13	3.17
9/29/22	10:22			13.8				2	70	164.1	17.4	-3.6	19	0	79	79	39	2366	533	0.9	1.67	4.09
9/29/22	10:27			14.14				5	75	168.2	13.3	-4.08	15	0	89	89	18	1073	436	0.9	1.14	2.27
9/29/22	10:32			14.43				5	80	171.7	9.8	-3.48	12	0	76	76	15	915	341	0.9	1.49	2.47
9/29/22	10:37			14.71				5	85	175.0	6.5	-3.36	8	0	74	74	15	883	255	0.9	2.59	3.19
9/29/22																						
9/29/22	10:50			11.1					98	131.7	49.8											
9/29/22	10:52			11.23				2	100	133.3	48.2	-1.56	49	0	34	34	17	1025	1282	0.9	0.15	0.74
9/29/22	10:57			12.98				5	105	154.3	27.2	-21	38	0	460	460	92	5521	999	0.9	2.23	5.10
9/29/22	11:00			13.33				3	108	158.5	23.0	-4.2	25	0	92	92	31	1840	682	0.9	0.86	2.49
9/29/22	11:05			14.05				5	113	167.1	14.4	-8.64	19	0	189	189	38	2271	521	0.9	2.31	4.02
9/29/22	11:10			14.37				5	118	170.9	10.6	-3.84	12	0	84	84	17	1010	364	0.9	1.50	2.56
9/29/22	11:12			14.5				2	120	172.5	9.0	-1.56	10	0	34	34	17	1025	296	0.9	1.77	3.19
9/29/22																						
9/29/22																						

Minimum Rate: 2.3
Raw Rate for design, prior to application of adjustment factors: **2.5**

APPENDIX C

LABORATORY TEST RESULTS

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.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of grain size analyses, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our boring logs.

R-value Test: One R-value test was performed on collected bulk soil sample to evaluate pavement support characteristics of the near-surface soils. R-value test was performed in accordance with Caltrans Standard Test Method 301. The test result is presented 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.

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	VVLIG Apple Valley Cordova Road	Tested By:	M. Vinet	Date:	09/28/22
Project No.:	13673.003	Input By:	M. Vinet	Date:	09/29/22
Boring No.:	LB-8	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Brown.				

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

Mold Volume (ft^3) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5522	5643	5696	5630		
Weight of Mold (g)	3530	3530	3530	3530		
Net Weight of Soil (g)	1992	2113	2166	2100		
Wet Weight of Soil + Cont. (g)	1107.4	1243.1	1058.8	1152.2		
Dry Weight of Soil + Cont. (g)	1074.0	1189.6	1005.0	1078.5		
Weight of Container (g)	420.0	418.7	420.8	419.8		
Moisture Content (%)	5.1	6.9	9.2	11.2		
Wet Density (pcf)	131.5	139.5	143.0	138.6		
Dry Density (pcf)	125.1	130.4	130.9	124.7		

Maximum Dry Density (pcf) **131.6** **Optimum Moisture Content (%)** **8.1**

PROCEDURE USED

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

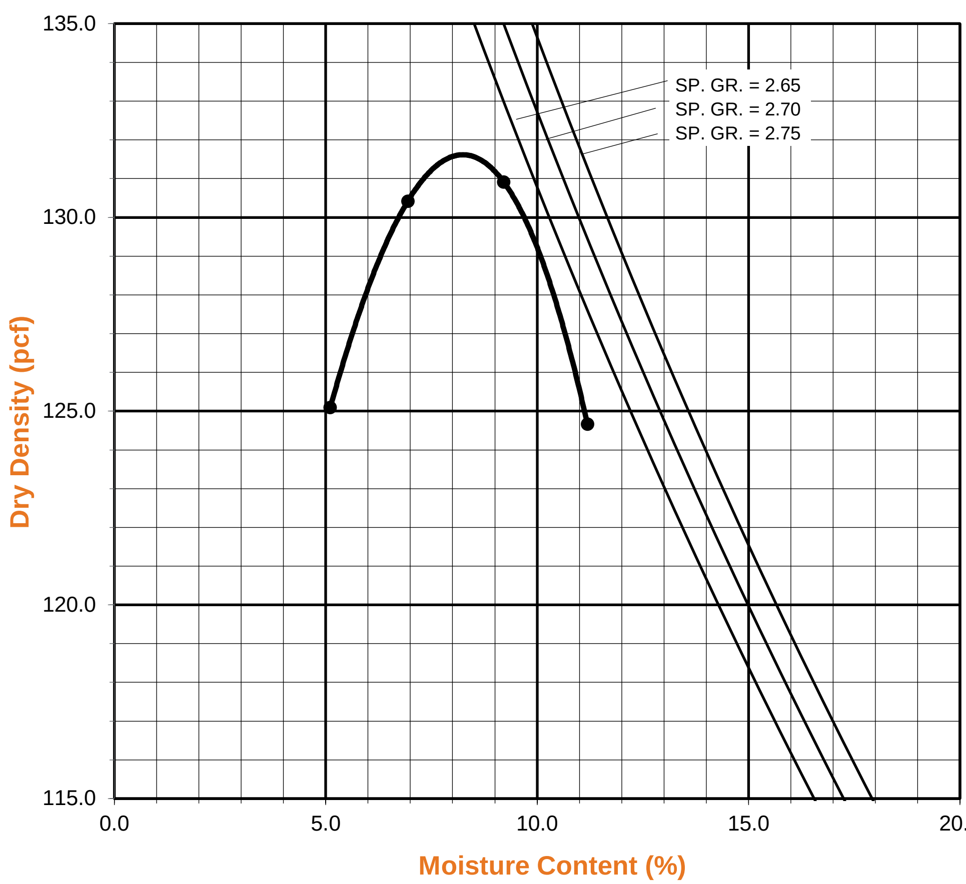
[X] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **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:
12:57:31
GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-1	LB-2	LB-3	LB-5	LB-6	LB-7	LB-9	
Sample No.	R-1	S-1	R-2	R-2	R-4	R-3	S-1	
Depth (ft.)	2.5	15.0	5.0	5.0	20.0	7.5	15.0	
Sample Type	RING	SPT	RING	RING	RING	RING	SPT	
Soil Classification	SW-SM	SP-SM	SM	SP-SM	SM	SP-SM	SP-SM	
Soak Time (min)	10	10	10	10	10	10	10	
Moisture Correction								
Wet Weight of Soil + Container (gm.)	718.0	684.2	619.2	670.5	611.5	627.7	486.5	
Dry Weight of Soil + Container (gm.)	713.6	682.4	611.1	651.0	599.0	610.7	483.7	
Weight of Container (gm)	277.1	281.3	279.9	279.3	277.9	277.7	276.3	
Moisture Content (%)	1.0	0.4	2.4	5.2	3.9	5.1	1.4	
Container No.:	AB	LA	MA	B1	K2	CC	R2	
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	713.6	682.4	611.1	651.0	599.0	610.7	483.7	
Weight of Container (gm.)	277.1	281.3	279.9	279.3	277.9	277.7	276.3	
Weight of Dry Sample (gm.)	436.5	401.1	331.2	371.7	321.1	333.0	207.4	
Container No.:	AB	LA	MA	B1	K2	CC	R2	
After Wash								
Dry Weight of Sample + Container (gm)	672.1	648.8	554.0	622.5	558.0	595.6	465.6	
Weight of Container (gm)	277.1	281.3	279.9	279.3	277.9	277.7	276.3	
Dry Weight of Sample (gm)	395.0	367.5	274.1	343.2	280.1	317.9	189.3	
% Passing No. 200 Sieve	**10**	**8**	**17**	**8**	**13**	**5**	**9**	
% Retained No. 200 Sieve	90	92	83	92	87	95	91	

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: VVLIG Apple Valley Cordova Road
Project No.: 13673.003
Client Name: VVLIG, Holdings, LLC
Tested By: M. Vinet Date: 09/26/22

Leighton

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

ASTM D 6913

Project Name:	VVLIG Apple Valley Cordova Road	Tested By:	MRV	Date:	09/26/22
Project No.:	13673.003	Checked By:	MRV	Date:	09/29/22
Boring No.:	LB-8	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	BL	BL	Wt. of Air-Dry Soil + Cont.(g)	1373.8	591.3
Wt. Air-Dried Soil + Cont.(g)	1373.8	591.3	Wt. of Dry Soil + Cont. (g)	1342.2	591.3
Wt. of Container (g)	278.3	278.3	Wt. of Container No._____(g)	278.3	278.3
Dry Wt. of Soil (g)	1063.6	313.0	Moisture Content (%)	3.0	0.0

Passing #4 Material After Wet Sieve	Container No.	BL
	Wt. of Dry Soil + Container (g)	493.8
	Wt. of Container (g)	278.3
	Dry Wt. of Soil Retained on # 200 Sieve (g)	215.5

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500			100.0
1"	25.000			100.0
3/4"	19.000	0.0		100.0
1/2"	12.500	37.6		96.5
3/8"	9.500	59.6		94.4
#4	4.750	130.0		87.8
#8	2.360		16.6	83.1
#16	1.180		43.4	75.6
#30	0.600		72.1	67.6
#50	0.300		108.6	57.3
#100	0.150		149.2	45.9
#200	0.075		203.7	30.7
PAN				

GRAVEL: **12 %**

SAND: **57 %**

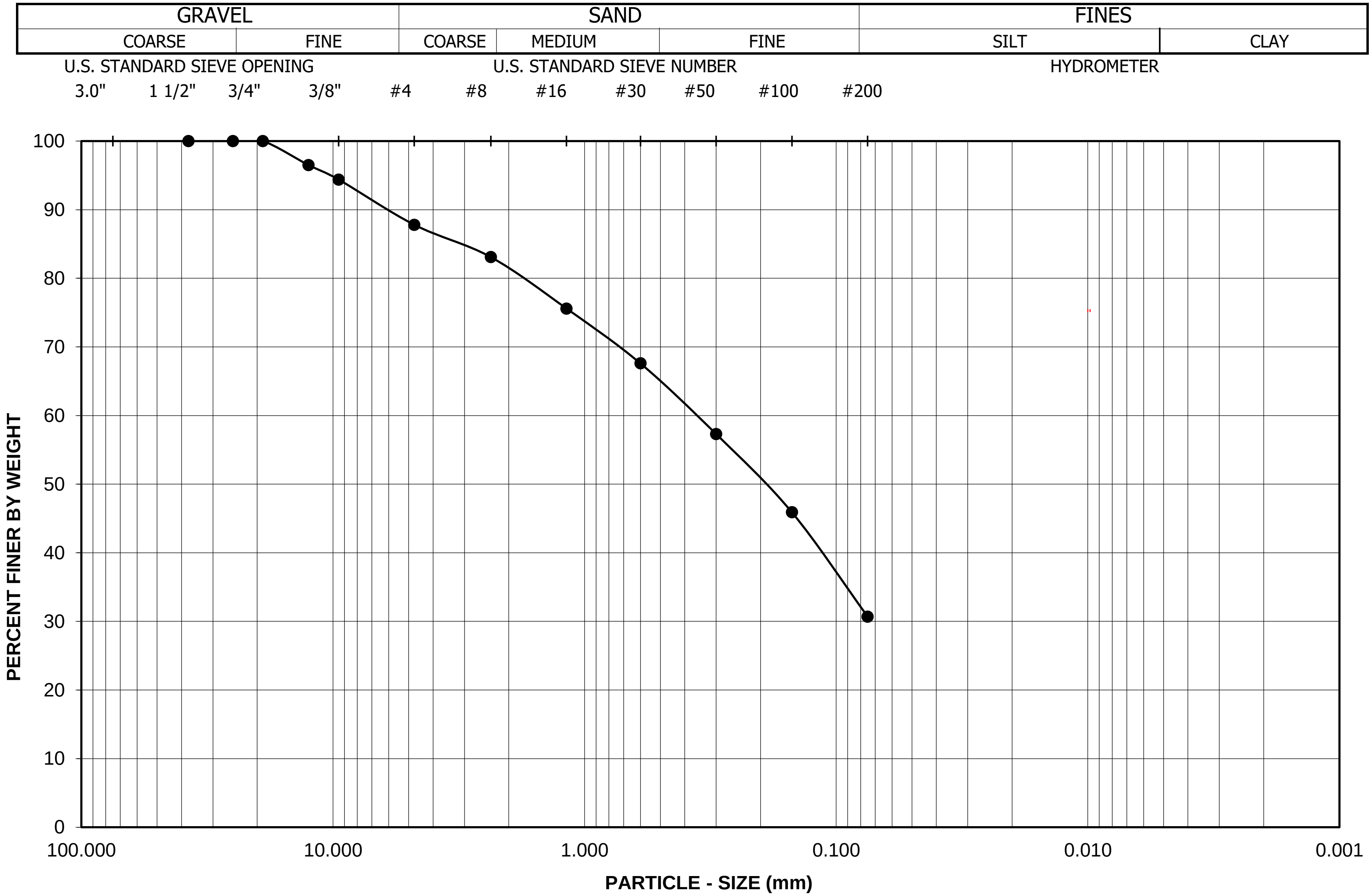
FINES: **31 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = N/A

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

Remarks:



Project Name: VVLIG Apple Valley Cordova Road

Project No.: 13673.003

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-8 Sample No.: B-1

Depth (feet): 0 - 5.0 Soil Type : SM

Soil Identification: Silty Sand (SM), Brown.

GR:SA:FI : (%) 12 : 57 : 31

Sep-22

Leighton

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

ASTM D 6913

Project Name:	VVLIG Apple Valley Cordova Road	Tested By:	MRV	Date:	09/26/22
Project No.:	13673.003	Checked By:	MRV	Date:	09/29/22
Boring No.:	LI-1	Depth (feet):	13.5		
Sample No.:	S-2				
Soil Identification:	Silty Sand (SM), Brown.				

		Moisture Content of Total Air - Dry Soil	
Container No.:	A	Wt. of Air-Dry Soil + Cont. (g)	644.4
Wt. of Air-Dried Soil + Cont.(g)	644.4	Wt. of Dry Soil + Cont. (g)	634.4
Wt. of Container (g)	279.6	Wt. of Container No.______ (g)	279.6
Dry Wt. of Soil (g)	354.8	Moisture Content (%)	2.8

After Wet Sieve	Container No.	A
	Wt. of Dry Soil + Container (g)	577.8
	Wt. of Container (g)	279.6
	Dry Wt. of Soil Retained on # 200 Sieve (g)	298.2

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
3"	75.000		100.0
1"	25.000		100.0
3/4"	19.000		100.0
1/2"	12.500		100.0
3/8"	9.500	0.0	100.0
#4	4.750	5.0	98.6
#8	2.360	27.4	92.3
#16	1.180	84.4	76.2
#30	0.600	158.9	55.2
#50	0.300	225.3	36.5
#100	0.150	267.6	24.6
#200	0.075	294.1	17.1
PAN			

GRAVEL: **1 %**

SAND: **82 %**

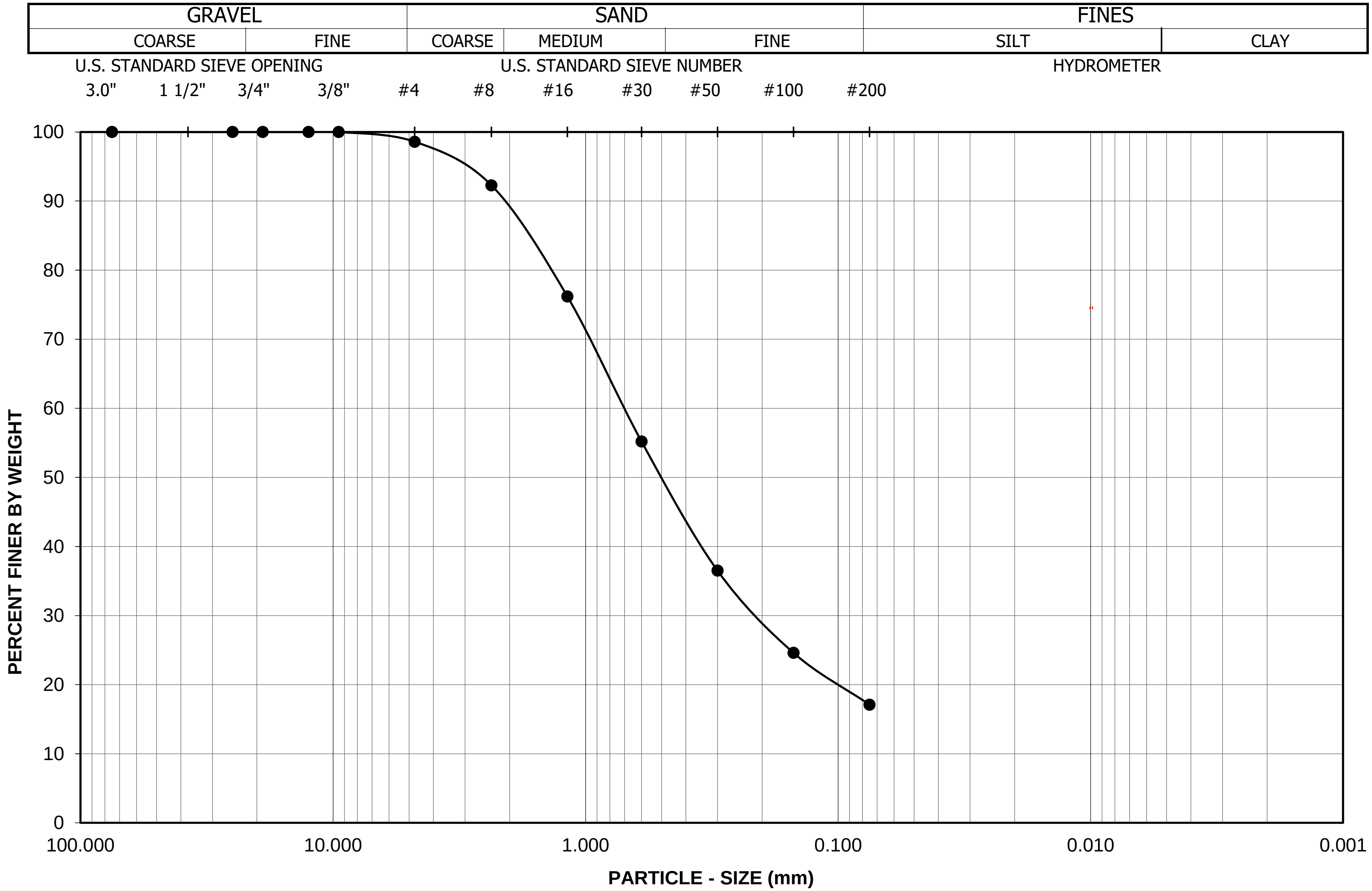
FINES: **17 %**

GROUP SYMBOL: **SM**

$Cu = D60/D10 =$ N/A

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

Remarks:



Project Name: VVLIG Apple Valley Cordova Road

Project No.: 13673.003

Boring No.: LI-1 Sample No.: S-2

Depth (feet): 13.5 Soil Type : SM

Soil Identification: Silty Sand (SM), Brown.

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 82 : 17

Sep-22

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

Leighton

Project Name:	VVLIG Apple Valley Cordova Road	Tested By:	MRV	Date:	09/26/22
Project No.:	13673.003	Checked By:	MRV	Date:	09/29/22
Boring No.:	LI-2	Depth (feet):	13.5		
Sample No.:	S-2				
Soil Identification:	Well-Graded Sand with Silt (SW-SM), Light Brown.				

		Moisture Content of Total Air - Dry Soil	
Container No.:	BA	Wt. of Air-Dry Soil + Cont. (g)	437.7
Wt. of Air-Dried Soil + Cont.(g)	437.7	Wt. of Dry Soil + Cont. (g)	436.8
Wt. of Container (g)	278.0	Wt. of Container No.______ (g)	278.0
Dry Wt. of Soil (g)	158.8	Moisture Content (%)	0.6

After Wet Sieve	Container No.	BA
	Wt. of Dry Soil + Container (g)	423.3
	Wt. of Container (g)	278.0
	Dry Wt. of Soil Retained on # 200 Sieve (g)	145.3

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
3"	75.000		100.0
1"	25.000		100.0
3/4"	19.000		100.0
1/2"	12.500		100.0
3/8"	9.500		100.0
#4	4.750	0.0	100.0
#8	2.360	35.7	77.5
#16	1.180	75.0	52.8
#30	0.600	101.9	35.8
#50	0.300	122.6	22.8
#100	0.150	135.9	14.4
#200	0.075	144.2	9.2
PAN			

GRAVEL: **0 %**

SAND: **91 %**

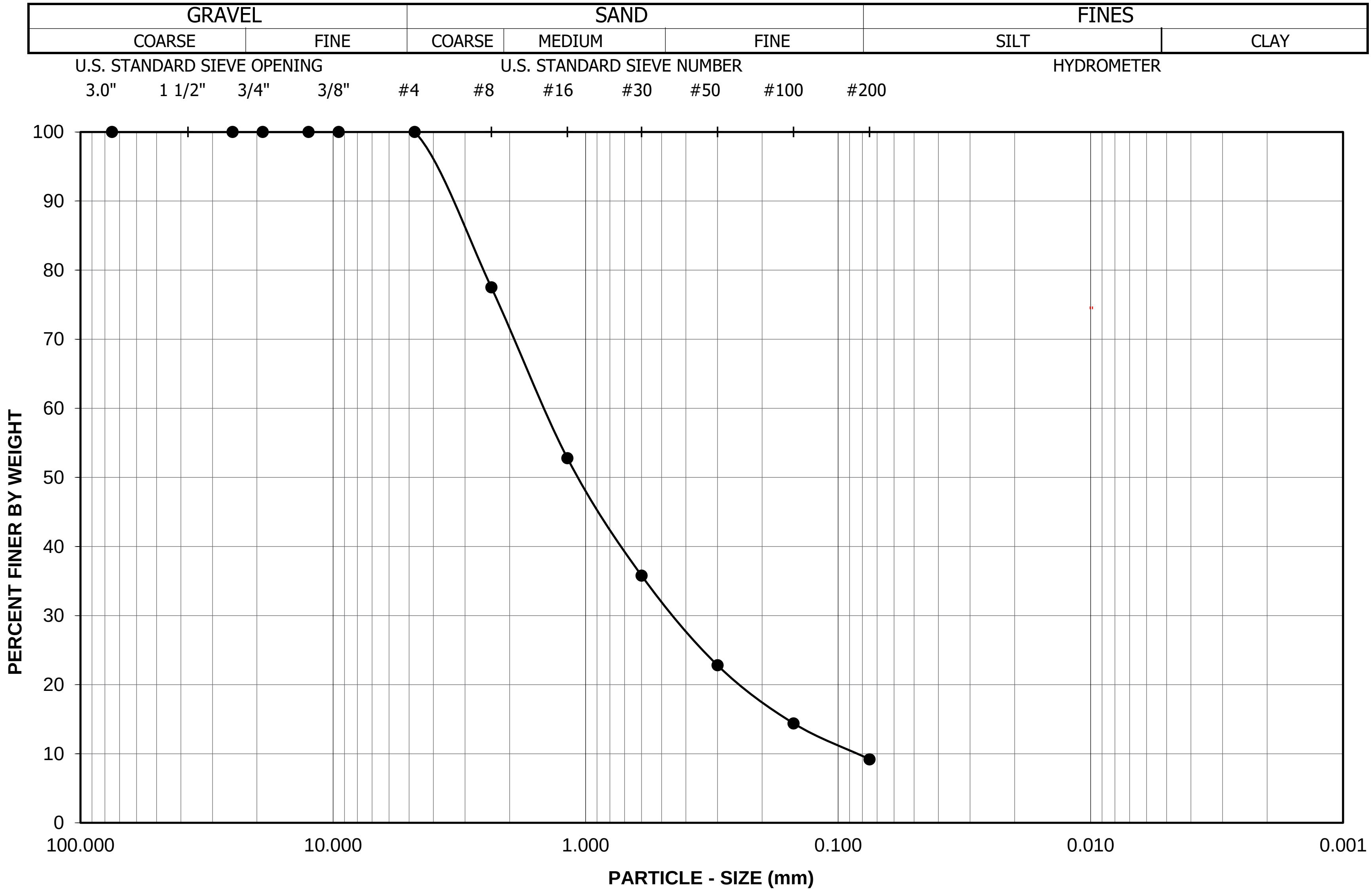
FINES: **9 %**

GROUP SYMBOL: **SW-SM**

Cu = D60/D10 = 18.29

Cc = $(D30)^2/(D60*D10)$ = 1.65

Remarks:



Project Name: VVLIG Apple Valley Cordova Road

Project No.: 13673.003

Boring No.: LI-2

Sample No.: S-2

Depth (feet): 13.5

Soil Type : SW-SM

Soil Identification: Well-Graded Sand with Silt (SW-SM), Light Brown.

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 91 : 9

Sep-22

One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546) -- Method 'B'

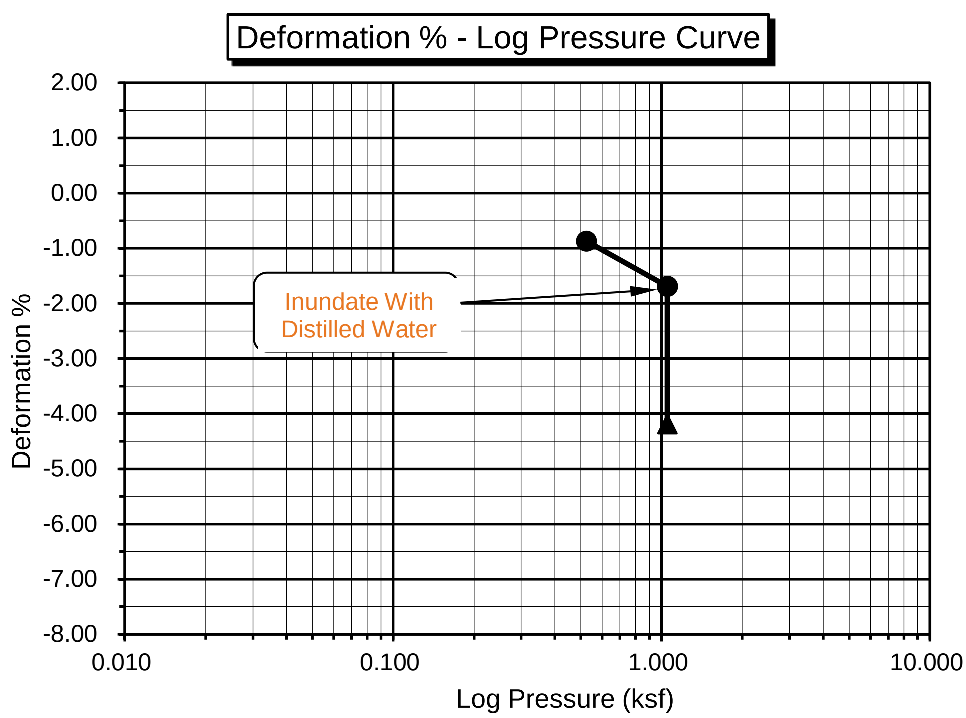
Project Name: VVLIG Apple Valley Cordova Road   Tested By: M. Vinet   Date: 9/26/22
Project No.: 13673.003   Checked By: M. Vinet   Date: 9/29/22
Boring No.: LB-5   Sample Type: IN SITU
Sample No.: R-3   Depth (ft.) 7.5
Sample Description: Poorly Graded Sand with Silt (SP-SM), Brown.
Source and Type of Water Used for Inundation: Arrowhead (Distilled)
** Note: Loading After Wetting (Inundation) not Performed Using this Test Method.

Initial Dry Density (pcf):	90.7
Initial Moisture (%):	4.9
Initial Height (in.):	1.0000
Initial Dial Reading (in):	0.0000
Inside Diameter of Ring (in):	2.416

Final Dry Density (pcf):	94.6
Final Moisture (%) :	23.0
Initial Void ratio:	0.8592
Specific Gravity (assumed):	2.70
Initial Degree of Saturation (%):	15.6

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.525	0.0087	0.9913	0.00	-0.87	0.8430	-0.87
1.050	0.0169	0.9831	0.00	-1.69	0.8278	-1.69
H2O	0.0419	0.9581	0.00	-4.19	0.7813	-4.19

Percent Swell / Settlement After Inundation = -2.54



TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: VVLIG Apple Valley Cordova Road — Tested By : M. Vinet — Date: 09/28/22

Project No. : 13673.003 — Data Input By: M. Vinet — Date: 09/29/22

Boring No.	LB-8			
Sample No.	B-1			
Sample Depth (ft)	0 - 5.0			
Soil Identification:	Silty Sand (SM)			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.0405			
Wt. of Crucible (g)	25.0362			
Wt. of Residue (g) (A)	0.0043			
PPM of Sulfate (A) x 41150	176.95			
PPM of Sulfate, Dry Weight Basis	**177**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**60**			

pH TEST, DOT California Test 643

pH Value	**7.80**			
Temperature °C	20.8			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	VVLIG Apple Valley Cordova Road	Tested By :	M. Vinet	Date:	09/28/22
Project No. :	13673.003	Data Input By:	M. Vinet	Date:	09/29/22
Boring No.:	LB-8	Depth (ft.) :	0 - 5.0		
Sample No. :	B-1				

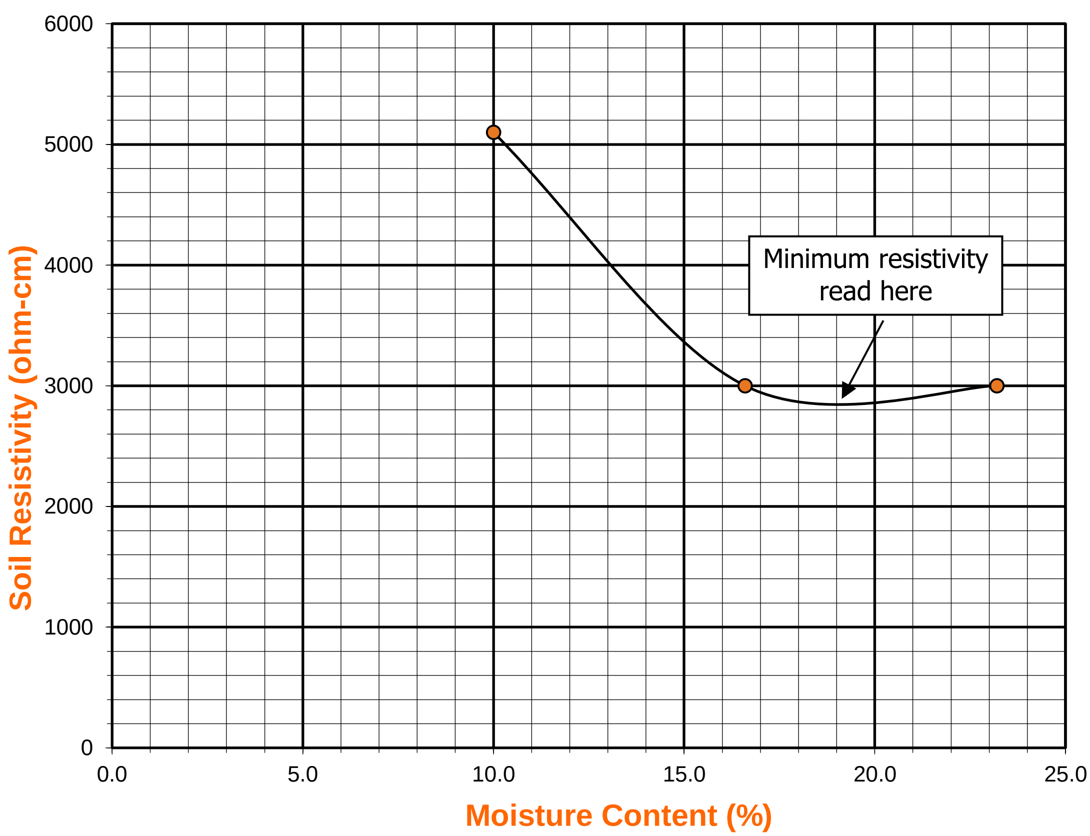
Soil Identification:* Silty Sand (SM)

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	5100	5100
2	83	16.60	3000	3000
3	116	23.20	3000	3000
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2820	**19.0**	**177**	**60**	**7.80**	**20.8**



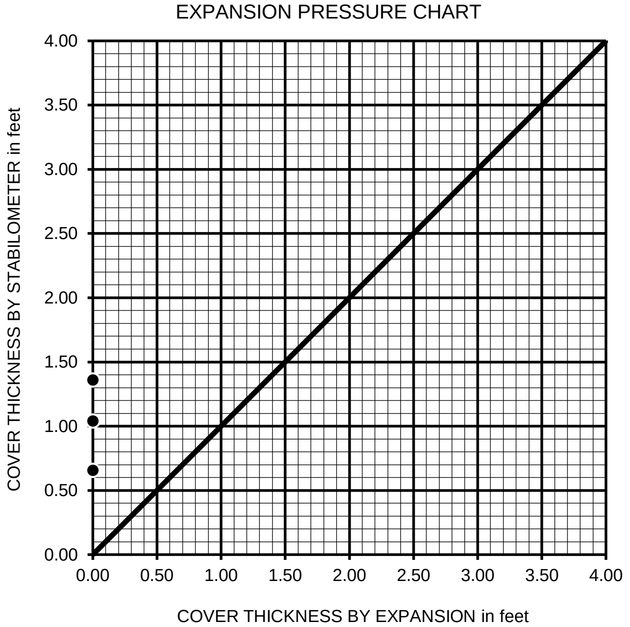
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	VVLIG Apple Valley Cordova Road	PROJECT NUMBER:	13673.003
BORING NUMBER:	LB-8	DEPTH (FT.):	0 - 5.0
SAMPLE NUMBER:	B-1	TECHNICIAN:	F. Mina
SAMPLE DESCRIPTION:	Silty Sand (SM), Brown.	DATE COMPLETED:	9/30/2022

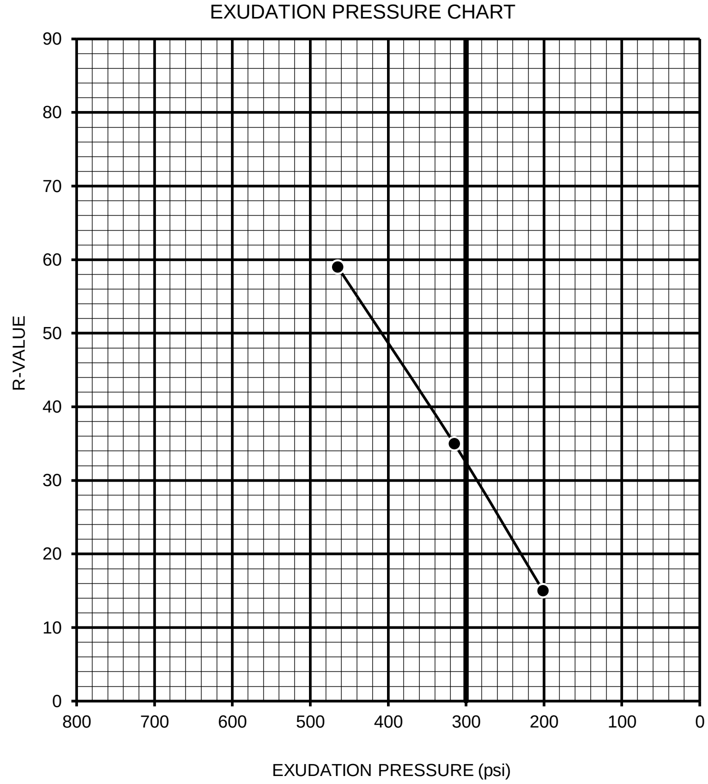
TEST SPECIMEN	**a**	**b**	**c**
MOISTURE AT COMPACTION %	9.0	10.5	11.5
HEIGHT OF SAMPLE, Inches	2.50	2.59	2.57
DRY DENSITY, pcf	120.4	118.3	119.2
COMPACTOR PRESSURE, psi	200	165	140
EXUDATION PRESSURE, psi	465	315	201
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	43	83	124
TURNS DISPLACEMENT	4.65	4.80	4.95
R-VALUE UNCORRECTED	59	33	13
R-VALUE CORRECTED	59	35	15

DESIGN CALCULATION DATA	**a**	**b**	**c**
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.66	1.04	1.36
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	32
EQUILIBRIUM R-VALUE:	32

APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

APPENDIX D

SITE-SPECIFIC SEISMIC ANALYSIS (ASCE 7-16)

VVLIG – Cordova Road Apple Valley
(34.6063, -117.1943)

A site-specific ground motion study was performed in general conformance with Chapters 11, 20 and 21 of ASCE 7-16 and CGS Note 48.

The site is approximately 5.68 km from the surface trace of the closest element of the Helendale-South Lockhart fault zone. A Class C soil profile condition was considered for this site based on the results of our exploratory borings and geophysical survey. The site-specific response spectra in tabular and graphic forms are included herein (see Exhibits C-1 through C-6) and our specific analysis or approach is further discussed below:

Exhibit C-1: The probabilistic MCE spectrum was developed using spectral values obtained from USGS Unified Hazard Maps (UHGM) website, using the factors of ASCE 7-16 Section 21.1. At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum is determined as the product of the risk coefficient, C_R, and the spectral response acceleration from a 5% damped acceleration response spectrum that has a 2% probability of exceedance within a 50-year period.

Exhibit C-2: A deterministic MCE spectrum was based on the maximum values of each period from the three most influential nearby faults. Scenario M8.2, 7.39, and 7.86 events on the Helendale-South Lockhart, San Andreas (San Bernardino section), and the Cucamonga fault zones consistent with the Next Generation West 2 (NGA-West 2) attenuation relations (PEER NGAW2 GMPEs) used for the 2014 USGS seismic source model at fault distances of 5.86, 44, and 45 km, respectively. The equally weighted spectral values from the attenuation relations of Abrahamson and others (ASK 2014), Boore and others (BSSA 2014), Campbell and Borzognia (CB 2014) and Chiou and Youngs (CY 2014) were used for the deterministic MCE spectrum. The MCE spectrum represents 84th-percentile, 5-percent-damped spectral response acceleration in the direction of maximum horizontal response (maximum rotated) for each period. Maximum rotated values were obtained using the scaling factors of ASCE 7-16 Section 21.2. Adjustment to the deterministic limit spectrum was applied as necessary. The Site Class C condition was modeled using Vs30 ≈ 560 meters/second, based on Multichannel Analysis of Surface Wave (MASW) methodology. The depth to bedrock ($Z_{1.0}$ km) was estimated to be around 197 feet (0.06 km), based on our geophysical survey results.

Exhibit C-3: The lesser of the values at any site period from the deterministic MCE_R and MCE_R probabilistic spectra forms the site-specific MCE_R spectrum. For this project site, the site-specific MCER spectrum is equivalent to the risk-modified probabilistic spectrum for all site periods.

Exhibits C-4 through C-6: A design response spectrum was determined according to the procedure outlined in ASCE 7-16, Section 21.3, and is equal to two-thirds of the response spectral accelerations of the site-specific MCE_R. The design spectrum is limited by a "floor" at 80 percent of spectral acceleration determined according to ASCE 7-16, Section 11.4.6. The recommended site-specific design response spectrum is attached in tabular and graphic forms.

PROBABILISTIC RESPONSE SPECTRA

Period (S)	UHGM (g)	C_R	Ordinated Value (g)	Max Dir SF	Max Dir RTGM (g)	Probabilistic Response (g)
0.01	0.512	0.933	0.477	1.1	0.525	0.525
0.10	1.055	0.933	0.985	1.1	1.083	1.083
0.20	1.247	0.933	1.164	1.1	1.280	1.280
0.30	1.125	0.932	1.048	1.124	1.178	1.178
0.50	0.855	0.929	0.795	1.175	0.934	0.934
0.75	0.631	0.926	0.584	1.2375	0.723	0.723
1.00	0.478	0.923	0.441	1.3	0.573	0.573
2.00	0.232	0.923	0.215	1.35	0.290	0.290
3.00	0.153	0.923	0.141	1.4	0.198	0.198
4.00	0.115	0.923	0.106	1.45	0.154	0.154
5.00	0.092	0.923	0.085	1.5	0.128	0.128

Peak Sa		Fa	1.2Fa	Peak Sa < 1.2Fa	Deterministic Needed?
1.280		1.0	1.2	NO	YES

UHGM - Obtained from Unified Hazard Maps
RTGM - Risk Target Ground Motion

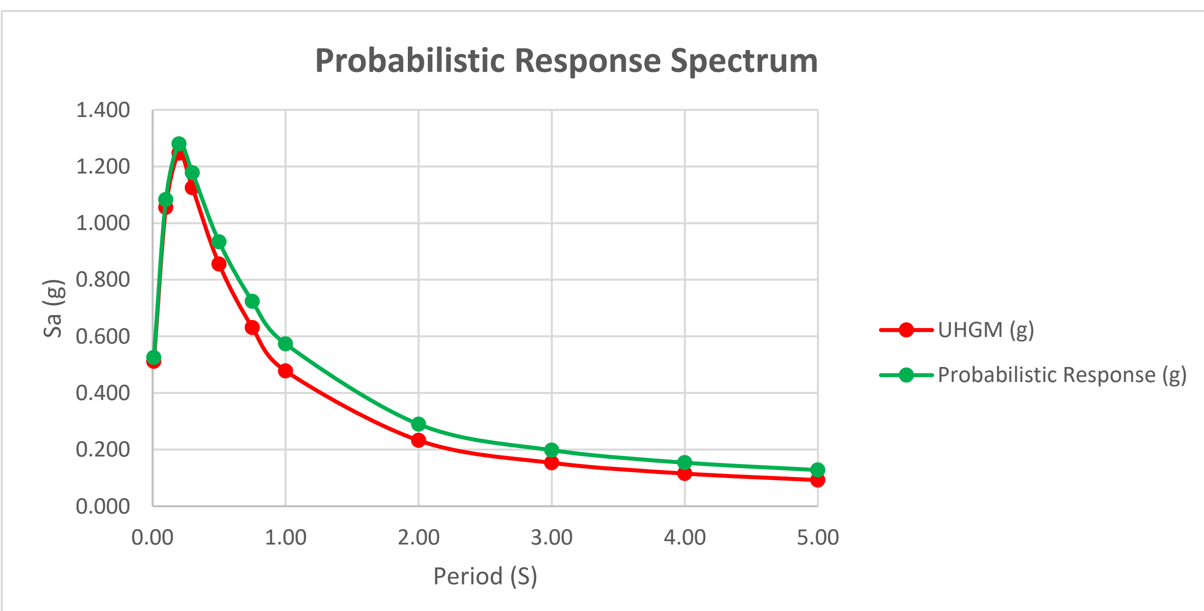


Exhibit C-1

DETERMINISTIC RESPONSE SPECTRUM

Period (S)	84th Percentile for 5% Damping	Max Dir SF	Max Dir Deterministic Sa	Scaled Max Dir Deterministic Sa
0.01	0.758	1.1	0.834	0.834
0.1	1.475	1.1	1.623	1.623
0.2	1.814	1.1	1.996	1.996
0.3	1.641	1.124	1.845	1.845
0.5	1.229	1.175	1.444	1.444
0.75	0.874	1.2375	1.082	1.082
1	0.651	1.3	0.847	0.847
2	0.286	1.35	0.386	0.386
3	0.171	1.4	0.239	0.239
4	0.117	1.45	0.170	0.170
5	0.087	1.5	0.130	0.130

Obtained from NGA West 2 GMPE Worksheet - UCERF3 fault

CALCS

Peak Sa	Fa	1.5Fa	Peak Sa < 1.5Fa	Scaling Factor
1.996	1.0	1.5	NO	1.000

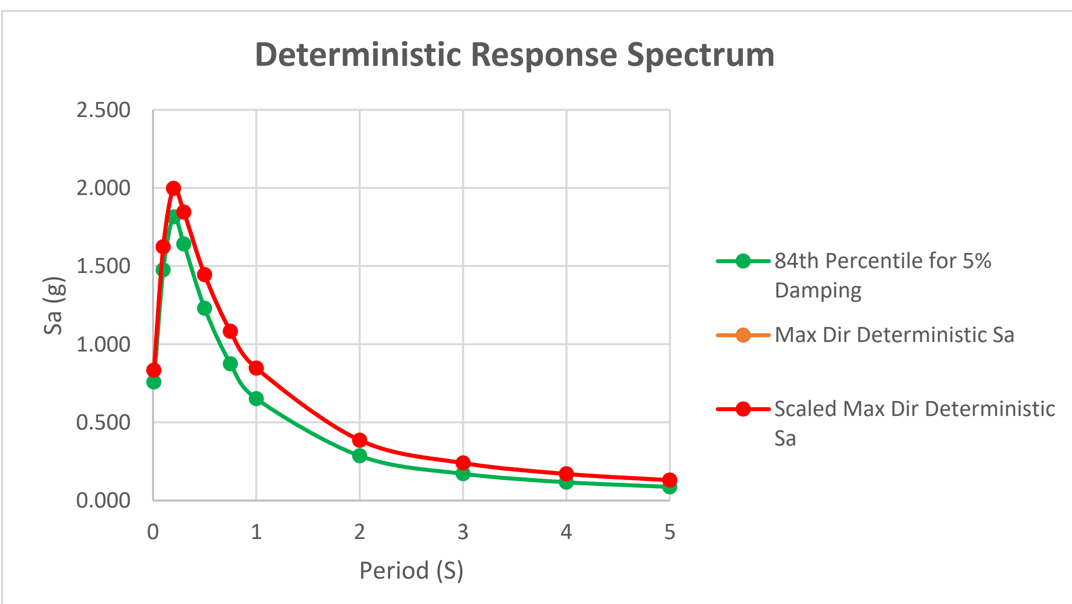


Exhibit C-2

SPECTRA COMPARISION

Period (s)	Probabilistic Response (g)	Scaled Max Dir Deterministic Sa (g)	MCE_{R^*} Response Spectra S_{aM} (g)	2/3 MCER Response Spectra Sa (g)
0.01	0.525	0.834	0.525	0.350
0.1	1.083	1.623	1.083	0.722
0.2	1.280	1.996	1.280	0.853
0.3	1.178	1.845	1.178	0.785
0.5	0.934	1.444	0.934	0.623
0.75	0.723	1.082	0.723	0.482
1	0.573	0.847	0.573	0.382
2	0.290	0.386	0.290	0.193
3	0.198	0.239	0.198	0.132
4	0.154	0.170	0.154	0.103
5	0.128	0.130	0.128	0.085

MCER* is the lesser of the probabilitic and deterministic spectra

CALCS

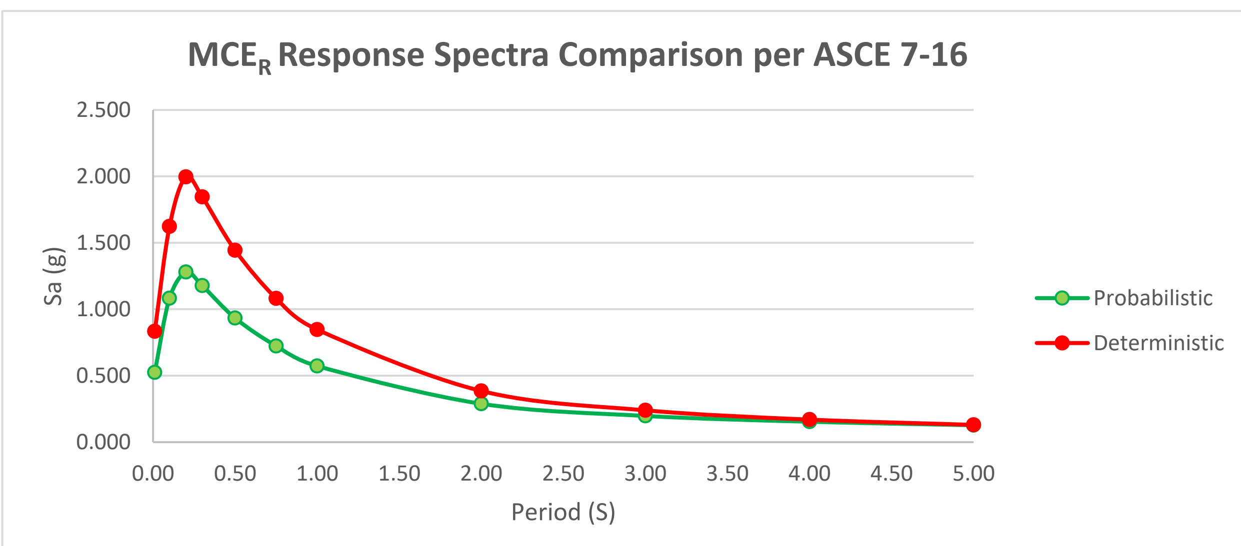


Exhibit C-3

S_s	1.018
S_1	0.391
Fa	1.2
Fv	1.5
S_{MS}	1.222
S_{M1}	0.587
S_{DS}	0.814
S_{D1}	0.391

since S_1 >0.2

T_0	0.100
T_S	0.500

PGA	0.438
PGA_M	0.526

Period (S)	Code-Based Sa (g)	80% Code-Based Sa (g)	2/3 MCER Response Spectra Sa (g)	Design Response Spectra Sa (g)
0.01	0.375	0.300	0.350	0.350
0.10	0.814	0.652	0.722	0.722
0.20	0.814	0.652	0.853	0.853
0.30	0.814	0.652	0.785	0.785
0.50	0.782	0.626	0.623	0.626
0.75	0.521	0.417	0.482	0.482
1.00	0.391	0.313	0.382	0.382
2.00	0.196	0.156	0.193	0.193
3.00	0.130	0.104	0.132	0.132
4.00	0.098	0.078	0.103	0.103
5.00	0.078	0.063	0.085	0.085

FROM SEISMIC MAPS (ATC OR OSHPD)

CALCS

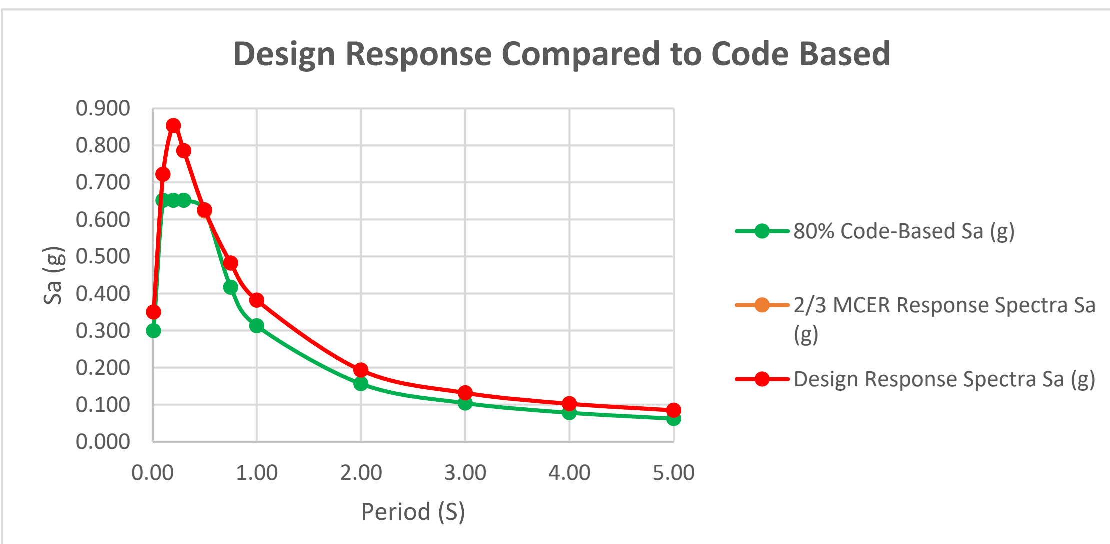


Exhibit C-4

Period (s)	MCER* Response Spectra SaM (g)	Design Response Spectra Sa (g)	Design Values (g)	
0.01	0.525	0.350	0.315	
0.10	1.083	0.722	0.650	
0.20	1.280	0.853	**0.768**	= S_{DS}
0.30	1.178	0.785	0.707	
0.50	0.934	0.626	0.563	
0.75	0.723	0.482	0.434	
1.00	0.573	0.382	0.382	
2.00	0.290	0.193	**0.386**	= S_{D1}
3.00	0.198	0.132	0.395	
4.00	0.154	0.103	0.410	
5.00	0.128	0.085	0.425	

Max Sa between T=0.2s and 5s is **0.853**

S_{DS} = 0.9 X Max Sa = **0.768**
S_{MS} = 1.5*S_{DS} = **1.152**

Short Period Spectrum

V_{S30} = 560 m/s > 365 m/s **Site Class C**

Max T*S_a between T=1s and 2s is **0.386**

Therefore, S_{D1} = **0.386**
S_{M1} = 1.5*S_{D1} = **0.579**

Long Period Spectrum

Probabilistic PGA **0.512**
Deterministic PGA **0.758**
80% Code-Based PGA_M **0.421**
Site-Specific PGA **0.758**

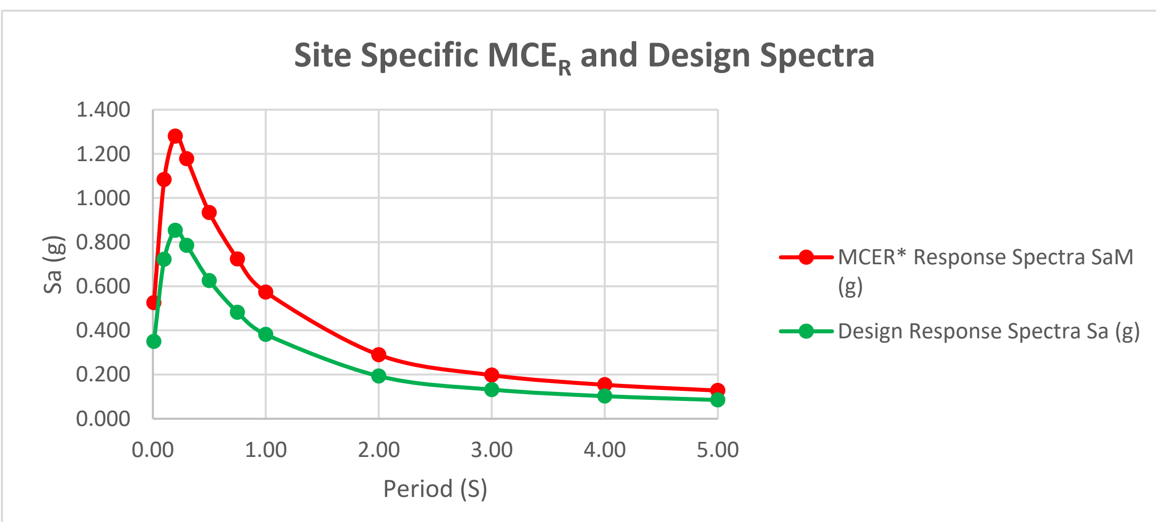


Exhibit C-5

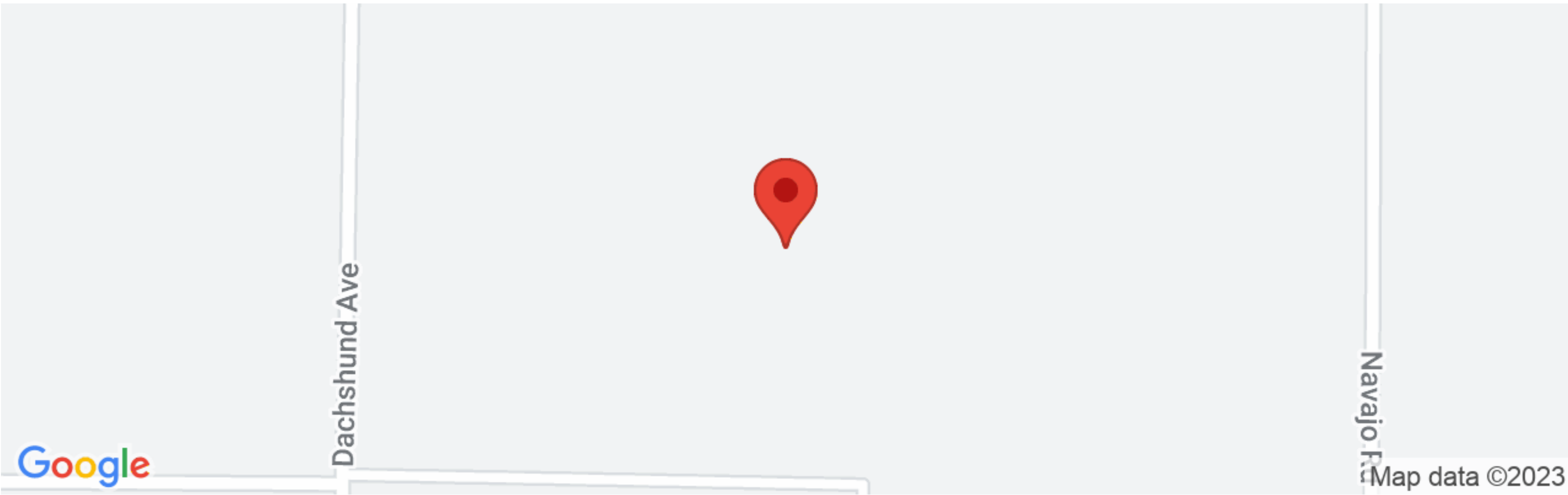
SUMMARY TABLE

Site-Specific Seismic Analysis (per ASCE 7-16)

	Site Seismic Coefficients / Coordinates	Value	
	Latitude	34.6063	
	Longitude	-117.1943	
Mapped Spectra (OSHPD)	Spectral Response – Class C (short), S_S	1.02	**Exhibit C-4**
	Spectral Response – Class C (1 sec), S_1	0.39	**Exhibit C-4**
	Site Modified Peak Ground Acceleration, PGA_M	0.53	**Exhibit C-4**
Site-Specific Response Spectra	Max. Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.15	**Exhibit C-5**
	Max. Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.58	**Exhibit C-5**
	5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.77	**Exhibit C-5**
	5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.39	**Exhibit C-5**
	Maximum Considered Earthqauke Geometric Mean MCE_G PGA	0.76	**Exhibit C-5**

Site 3

Latitude, Longitude: 34.6063, -117.1943



Date	1/6/2023, 11:24:01 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.018	MCE_R ground motion. (for 0.2 second period)
S_1	0.391	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.222	Site-modified spectral acceleration value
S_{M1}	0.587	Site-modified spectral acceleration value
S_{DS}	0.815	Numeric seismic design value at 0.2 second SA
S_{D1}	0.391	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.438	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.526	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.018	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.091	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.723	Factored deterministic acceleration value. (0.2 second)
S1RT	0.391	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.424	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.636	Factored deterministic acceleration value. (1.0 second)
PGAd	0.719	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.438	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.933	Mapped value of the risk coefficient at short periods
C_{R1}	0.923	Mapped value of the risk coefficient at a period of 1 s
C_V	1.104	Vertical coefficient

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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 (update…

Latitude

Decimal degrees

34.6063

Longitude

Decimal degrees, negative values for western longitudes

-117.1943

Site Class

537 m/s (Site class C)

Spectral Period

5.00 Second Spectral Acceleration

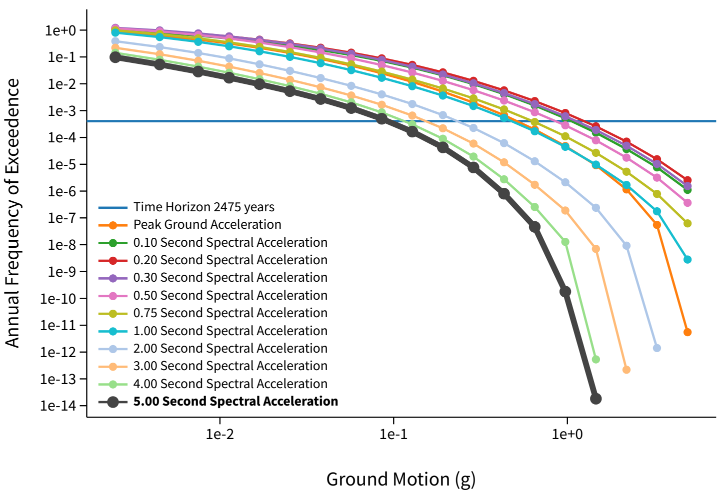
Time Horizon

Return period in years

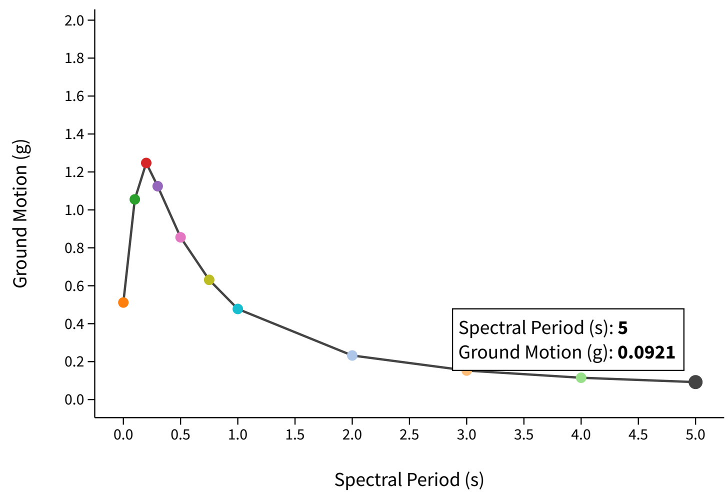
2475

Hazard Curve

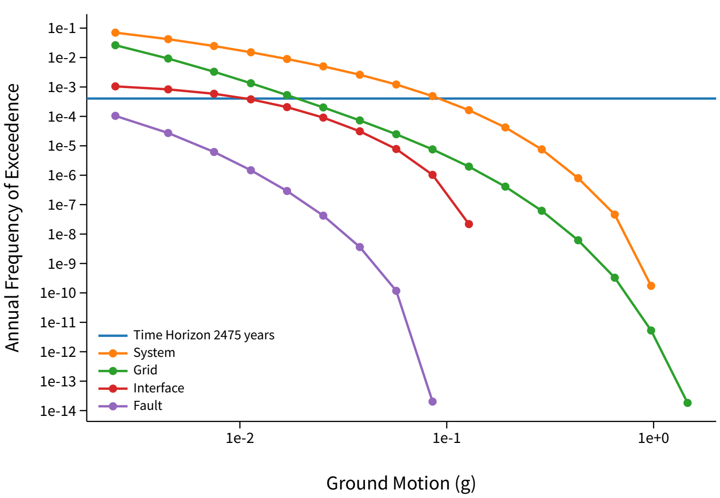
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for 5.00 Second Spectral Acceleration

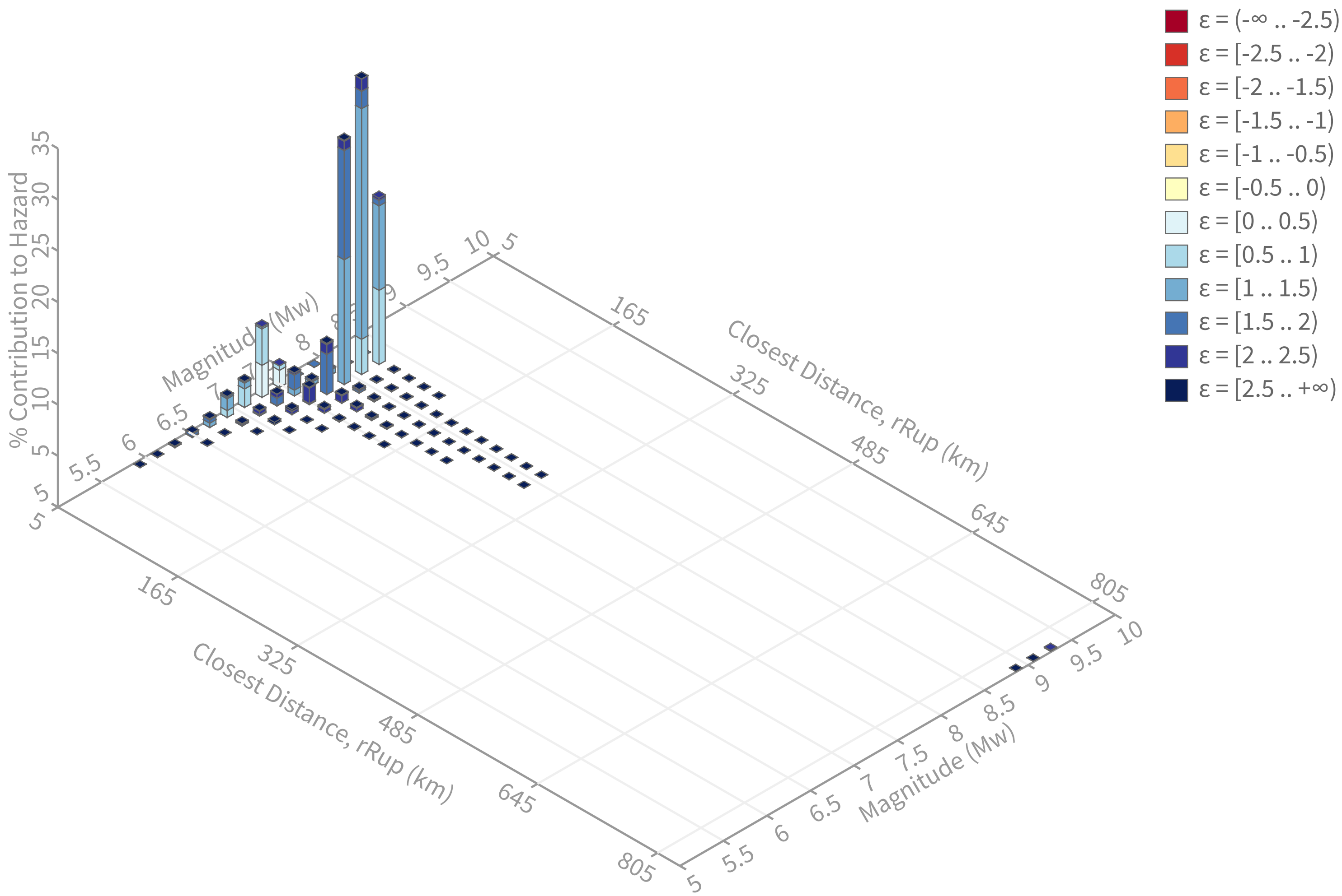


View Raw Data

Deaggregation

Component

Total



ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)
% Contribution to Hazard
Magnitude (Mw)
Closest Distance, rRup (km)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
5.0 s SA ground motion: 0.092147301 g

Recovered targets

Return period: 2728.8802 yrs
Exceedance rate: 0.00036645068 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.17 %

Mean (over all sources)

m: 7.86
r: 40.19 km
ε_0: 1.33 σ

Mode (largest m-r bin)

m: 8.1
r: 44.18 km
ε_0: 1.25 σ
Contribution: 28.82 %

Mode (largest m-r-ε_0 bin)

m: 8.09
r: 44.18 km
ε_0: 1.23 σ
Contribution: 22.5 %

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							49.25
San Andreas (San Bernardino N) [1]		44.13	8.06	1.30	117.456°W	34.273°N	212.99	33.36
Helendale-So Lockhart [7]		5.68	7.23	0.65	117.151°W	34.641°N	46.13	6.14
Cucamonga [0]		45.00	7.86	1.62	117.445°W	34.192°N	206.56	1.04
UC33brAvg_FM32	System							49.22
San Andreas (San Bernardino N) [1]		44.13	8.06	1.30	117.456°W	34.273°N	212.99	33.38
Helendale-So Lockhart [7]		5.68	7.23	0.66	117.151°W	34.641°N	46.13	6.08
Cucamonga [0]		45.00	7.87	1.61	117.445°W	34.192°N	206.56	1.14

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

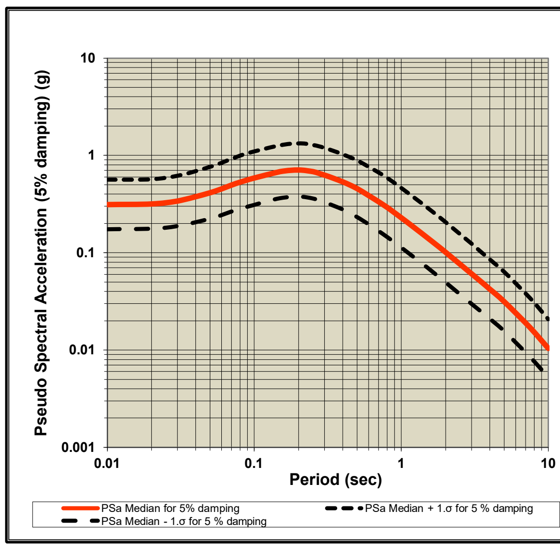
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

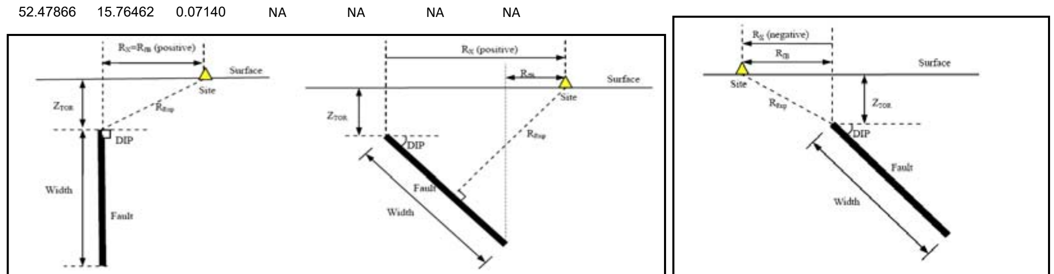
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

RotD50 Horizontal Component of PGA, PGV and *IMs*

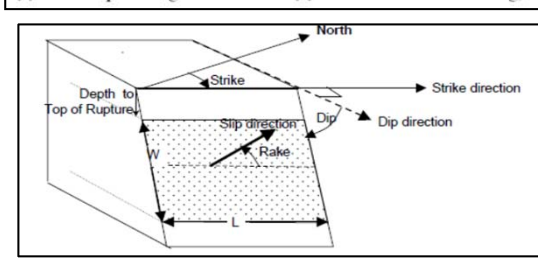
Input variables	Errors and warnings
M_w: 7.39	
R_{RUP} (km): 15	
R_{JB} (km): 12.7	
R_X (km): 12.7	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 560	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 90	
Z_{TOR} (km): 8.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.06	*If unknown use 999*
$Z_{2.5}$ (km): 0.26	*If unknown use 999*
W (km): 11.5	*If unknown use 999*
Vs30Flag: inferred	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

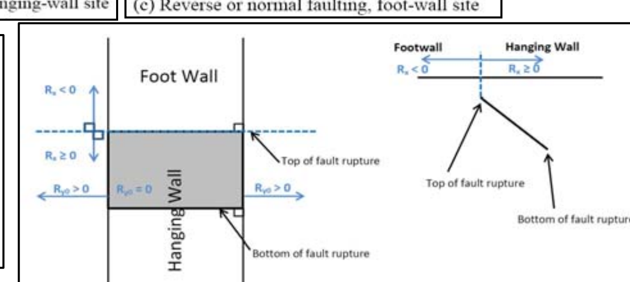
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.31362	0.56333	0.17460	0.00078	0.31362	0.56333	0.17460	0.00078
	0.02	0.31777	0.57160	0.17666	0.00316	0.31777	0.57160	0.17666	0.00316
	0.03	0.34147	0.61891	0.18840	0.00763	0.34113	0.61829	0.18821	0.00762
	0.05	0.41358	0.76252	0.22433	0.02567	0.41358	0.76252	0.22433	0.02567
	0.075	0.51299	0.96120	0.27378	0.07163	0.51452	0.96408	0.27460	0.07184
	0.1	0.58436	1.09984	0.31048	0.14506	0.58611	1.10314	0.31141	0.14550
	0.15	0.67980	1.27315	0.36298	0.37969	0.68116	1.27569	0.36371	0.38045
	0.2	0.70999	1.33272	0.37824	0.70499	0.71141	1.33538	0.37900	0.70640
	0.25	0.68004	1.27968	0.36139	1.05507	0.68140	1.28223	0.36211	1.05718
	0.3	0.62760	1.19268	0.33025	1.40215	0.62886	1.19507	0.33091	1.40495
	0.4	0.53462	1.02737	0.27821	2.12340	0.53516	1.02839	0.27848	2.12553
	0.5	0.45515	0.88720	0.23350	2.82461	0.45560	0.88808	0.23373	2.82744
	0.75	0.31293	0.62724	0.15611	4.36948	0.31293	0.62724	0.15611	4.36948
	1	0.22916	0.46441	0.11308	5.68856	0.22916	0.46441	0.11308	5.68856
	1.5	0.14368	0.29271	0.07053	8.02505	0.14382	0.29300	0.07060	8.03308
	2	0.10116	0.20616	0.04964	10.04436	0.10106	0.20595	0.04959	10.03432
	3	0.06078	0.12375	0.02985	13.57795	0.06078	0.12375	0.02985	13.57795
	4	0.04240	0.08547	0.02104	16.84176	0.04236	0.08539	0.02102	16.82491
	5	0.03167	0.06392	0.01569	19.65199	0.03157	0.06373	0.01564	19.59304
	7.5	0.01707	0.03430	0.00849	23.82858	0.01700	0.03416	0.00846	23.73326
	10	0.01049	0.02084	0.00528	26.03193	0.01044	0.02075	0.00526	25.92781
PGA (g)	0	0.31219	0.56033	0.17394	0.00077	0.31219	0.56033	0.17394	0.00077
PGV (cm/s)	-1	28.76293	52.47866	15.76462	0.07140	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.237	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 1	*auto calculated*
$V_{s30Flag}$: 0	*inferred*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	11.50			15.000		
$Z_{1.0}$ (km)	0.060	0.060			0.162	
$\delta Z_{1.0}$ (km)	-0.102		-0.102			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.260			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.260			0.861		
Z_{hyp} (km)	999.00			10.227		
Z_{tor} (km)	8.20			0.000	0.000	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

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Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	Weighted average of the natural logarithm of the spectral values

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

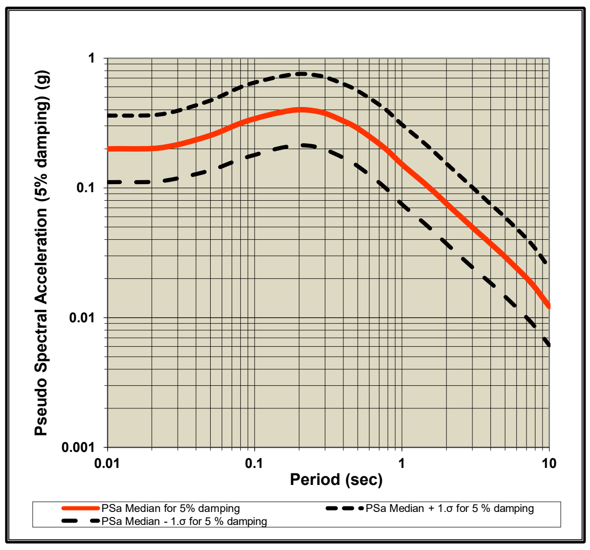
# of std. dev.	1	
Damping ratio (%)	5	Modification factors are calculated in Sheet DSF

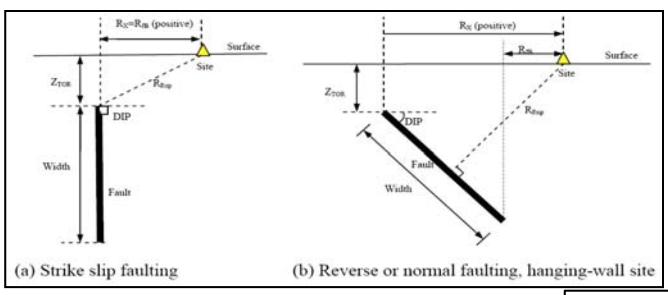
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

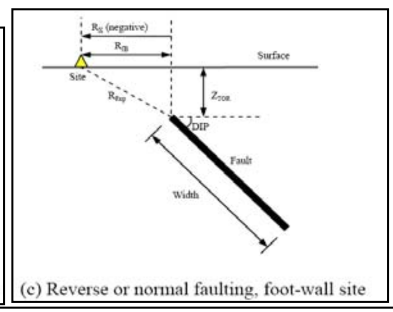
Input variables	Errors and warnings
M_w: 8.2	
R_{RUP} (km): 44.7	
R_{JB} (km): 44	
R_X (km): 44	
Ry0 (km): 999	If unknown use 999
V_{S30} (m/sec): 560	
U (BSSA13): 0	1: Unspecified fault mech.
F_{RV}: 0	1: reverse fault
F_{NM}: 0	1: normal fault
F_{HW}: 0	1: hanging wall side
Dip (deg): 90	
Z_{TOR} (km): 7.68	If unknown use 999
Z_{HYP} (km): 999	If unknown use 999
$Z_{1.0}$ (km): 0.06	If unknown use 999
$Z_{2.5}$ (km): 0.26	If unknown use 999
W (km): 12.8	If unknown use 999
Vs30Flag: inferred	Choose options for V_{s30} from the list
F_{AS}: no	Aftershock effect is not applicable.
Region: California	Choose region from the list

GMP	T (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.20031	0.36123	0.11108	0.00050	0.20031	0.36123	0.11108	0.00050
	0.02	0.20154	0.36403	0.11158	0.00200	0.20154	0.36403	0.11158	0.00200
	0.03	0.21606	0.39371	0.11857	0.00483	0.21584	0.39332	0.11845	0.00482
	0.05	0.25383	0.47166	0.13660	0.01575	0.25408	0.47214	0.13674	0.01577
	0.075	0.30568	0.57802	0.16166	0.04268	0.30660	0.57975	0.16215	0.04281
	0.1	0.34033	0.64661	0.17913	0.08448	0.34169	0.64920	0.17984	0.08482
	0.15	0.38234	0.72179	0.20253	0.21355	0.38349	0.72395	0.20314	0.21419
	0.2	0.39992	0.75407	0.21209	0.39710	0.40112	0.75633	0.21273	0.39829
	0.25	0.39415	0.74353	0.20894	0.61152	0.39415	0.74353	0.20894	0.61152
	0.3	0.37445	0.71248	0.19680	0.83657	0.37557	0.71462	0.19739	0.83908
	0.4	0.32832	0.63128	0.17075	1.30401	0.32897	0.63254	0.17109	1.30661
	0.5	0.28739	0.56045	0.14737	1.78355	0.28739	0.56045	0.14737	1.78355
	0.75	0.20587	0.41275	0.10268	2.87464	0.20608	0.41317	0.10279	2.87751
	1	0.15174	0.30755	0.07486	3.76662	0.15189	0.30786	0.07494	3.77039
	1.5	0.10288	0.20961	0.05050	5.74625	0.10298	0.20982	0.05055	5.75200
	2	0.07586	0.15462	0.03722	7.53256	0.07578	0.15446	0.03718	7.52502
	3	0.04967	0.10115	0.02439	11.09739	0.04972	0.10125	0.02442	11.10848
	4	0.03730	0.07519	0.01851	14.81614	0.03723	0.07504	0.01847	14.78650
	5	0.02947	0.05949	0.01460	18.29153	0.02942	0.05937	0.01457	18.25495
	7.5	0.01869	0.03756	0.00930	26.09496	0.01856	0.03730	0.00923	25.91230
	10	0.01225	0.02435	0.00617	30.41544	0.01218	0.02420	0.00613	30.23294
PGA (g)	0	0.19945	0.35937	0.11069	0.00050	0.19945	0.35937	0.11069	0.00050
PGV (cm/s)	-1	20.87321	38.08669	11.43945	0.05181	NA	NA	NA	NA

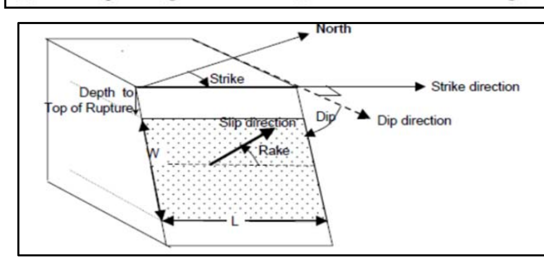


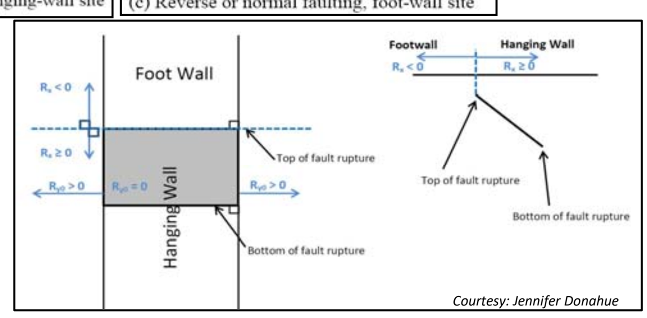


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Calculated Variables/Flags

ΔDPP	0	Always 0 for median calcs.
PGA_r (g)	0.146	
Z_{BOT} (km) (CB14)	15	Enter for default W calcs
SS	1	auto calculated
$V_{s30Flag}$	0	inferred
F_{AS}	0	Aftershock effect is not applicable.
Region	0	California
Option for Sa value	1	Weighted average of the natural logarithm of the spectral values

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	12.80			15.000		
$Z_{1.0}$ (km)	0.060	0.060			0.162	
$\delta Z_{1.0}$ (km)	-0.102		-0.102			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.260			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.260			0.861		
Z_{hyp} (km)	999.00			10.227		
Z_{tor} (km)	7.68			0.000	0.000	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

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Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

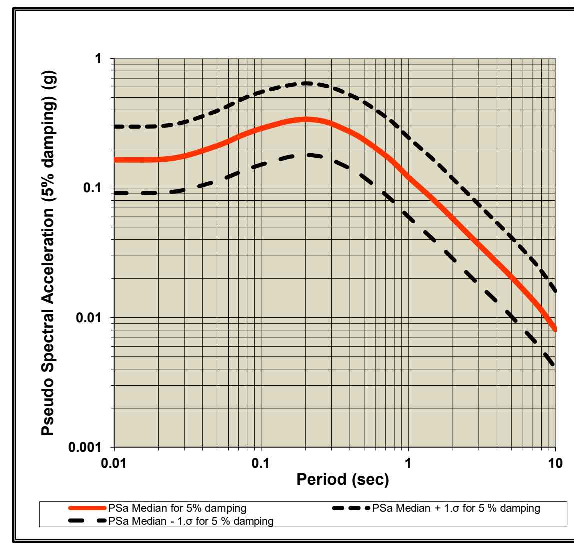
Input variables	Value	Errors and warnings
M_w	7.86	
R_{RUP} (km)	45.7	
R_{JB} (km)	45	
R_X (km)	45	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	560	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	0	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	0	*1: hanging wall side*
Dip (deg)	45	
Z_{TOR} (km)	8.3	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.06	*If unknown use 999*
$Z_{2.5}$ (km)	0.26	*If unknown use 999*
W (km)	16.45	*If unknown use 999*
Vs30Flag	inferred	*Choose options for V_{s30} from the list*
F_{AS}	no	*Aftershock effect is not applicable.*
Region	California	*Choose region from the list*

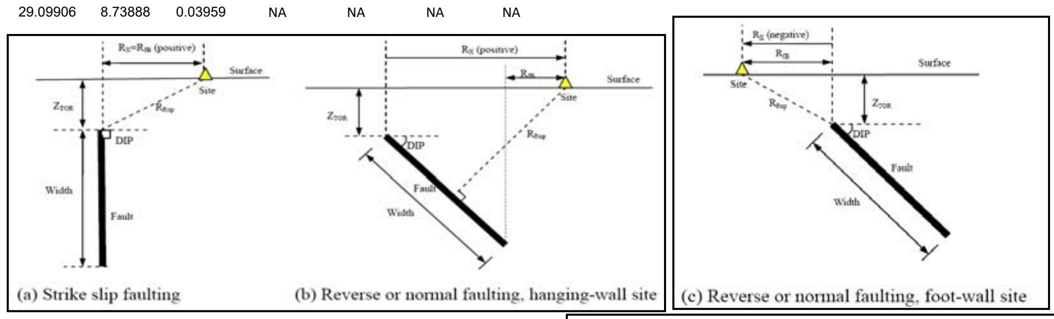
Calculated Variables/Flags

ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.119	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	1	*auto calculated*
$V_{s30Flag}$	0	*inferred*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

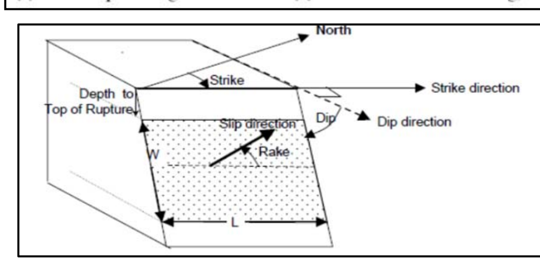
RotD50 Horizontal Component of PGA, PGV and *IMs*

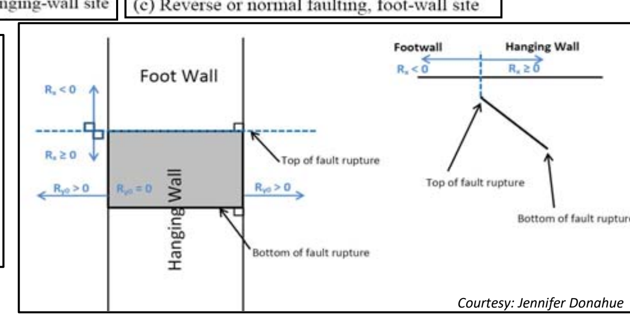
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.16490	0.29774	0.09133	0.00041	0.16490	0.29774	0.09133	0.00041
	0.02	0.16599	0.30022	0.09177	0.00165	0.16599	0.30022	0.09177	0.00165
	0.03	0.17709	0.32330	0.09701	0.00396	0.17692	0.32297	0.09691	0.00395
	0.05	0.21099	0.39314	0.11323	0.01309	0.21120	0.39353	0.11335	0.01311
	0.075	0.25631	0.48624	0.13511	0.03579	0.25708	0.48770	0.13551	0.03590
	0.1	0.28739	0.54786	0.15075	0.07134	0.28854	0.55005	0.15136	0.07163
	0.15	0.32484	0.61495	0.17159	0.18143	0.32581	0.61679	0.17211	0.18198
	0.2	0.33836	0.63892	0.17918	0.33597	0.33937	0.64084	0.17972	0.33698
	0.25	0.33125	0.62536	0.17546	0.51393	0.33125	0.62536	0.17546	0.51393
	0.3	0.31246	0.59480	0.16414	0.69807	0.31339	0.59659	0.16463	0.70016
	0.4	0.27122	0.52165	0.14102	1.07723	0.27176	0.52269	0.14130	1.07939
	0.5	0.23518	0.45873	0.12057	1.45949	0.23518	0.45873	0.12057	1.45949
	0.75	0.16618	0.33323	0.08288	2.32048	0.16635	0.33356	0.08296	2.32280
	1	0.12127	0.24583	0.05983	3.01045	0.12139	0.24608	0.05989	3.01346
	1.5	0.08004	0.16309	0.03928	4.47068	0.08012	0.16325	0.03932	4.47515
	2	0.05797	0.11817	0.02844	5.75656	0.05792	0.11805	0.02841	5.75081
	3	0.03663	0.07459	0.01799	8.18331	0.03663	0.07459	0.01799	8.18331
	4	0.02672	0.05386	0.01326	10.61216	0.02667	0.05375	0.01323	10.59093
	5	0.02064	0.04166	0.01023	12.80979	0.02060	0.04158	0.01021	12.78417
	7.5	0.01258	0.02528	0.00626	17.56412	0.01249	0.02511	0.00621	17.44117
	10	0.00813	0.01616	0.00409	20.18677	0.00809	0.01608	0.00407	20.08583
PGA (g)	**0**	0.16418	0.29621	0.09100	0.00041	0.16418	0.29621	0.09100	0.00041
PGV (cm/s)	**-1**	15.94657	29.09906	8.73888	0.03959	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

- ***Damping ratio*** = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
- ***PSA*** = Pseudo-absolute acceleration response spectrum (g)
- ***PGA*** = Peak ground acceleration (g)
- ***PGV*** = Peak ground velocity (cm/s)
- S_d = Relative displacement response spectrum (cm)
- M_w = Moment magnitude
- R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
- R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
- R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
- R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
- V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
- ***U*** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
- F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
- ***Dip*** = Average dip of rupture plane (degrees)
- Z_{TOR} = Depth to top of coseismic rupture (km)
- Z_{HYP} = Hypocentral depth from the earthquake
- $Z_{1.0}$ = Depth to Vs=1 km/sec
- $Z_{2.5}$ = Depth to Vs=2.5 km/sec
- ***W*** = Fault rupture width (km)
- $V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
- F_{AS} = 0 for mainshock; 1 for aftershock
- ***Region*** = Specific regions considered in the models, Click on Region to see codes
- ***ΔDPP*** = Directivity term, direct point parameter; uses 0 for median predictions
- PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
- Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
- Z_{BOR} (km) = The depth to the bottom of the rupture plane
- ***SS*** = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	16.45			21.213		
$Z_{1.0}$ (km)	0.060	0.060			0.162	
$\delta Z_{1.0}$ (km)	-0.102		**-0.102**			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.260			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.260			0.861		
Z_{hyp} (km)	999.00			**10.227**		
Z_{tor} (km)	8.30			0.000	0.000	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Determination of Site Class and Estimation of Shear Wave Velocity
Project: 13673.003 Cordova Rd

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)								Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2	LB-4	LB-5	LB-6	LB-7	LB-8	LB-10		1.3	
5	7.5	36	60	30	60	60	60	32	60	50	65	0.12
10	5	32	60	60	45	60	60	60	60	55	71	0.07
15	5	36	100	100	60	100	60	100	100	82	100	0.05
20	5	100	60	60	100	60	100	60	60	75	98	0.05
25	5	36				100		100	100	84	100	0.05
30	5					60		60		60	78	0.06
35	5					50				50	65	0.08
40	5					50				50	65	0.08
45	5					50				50	65	0.08
50	7.5					50				50	65	0.12
60	10					50	*Assumed based on blowcount			50	65	0.15
70	10					50				50	65	0.15
80	10					50				50	65	0.15
90	10					50				50	65	0.15
100	5					50				50	65	0.08
Summatioı	100											1.44
										Navg = Sum(di) / Sum(di / Ni) =		**69**

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2019 CBC 1613A.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	**50.001**	**100**	**1200**	**2500**	366	762	**69**	**1705**
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

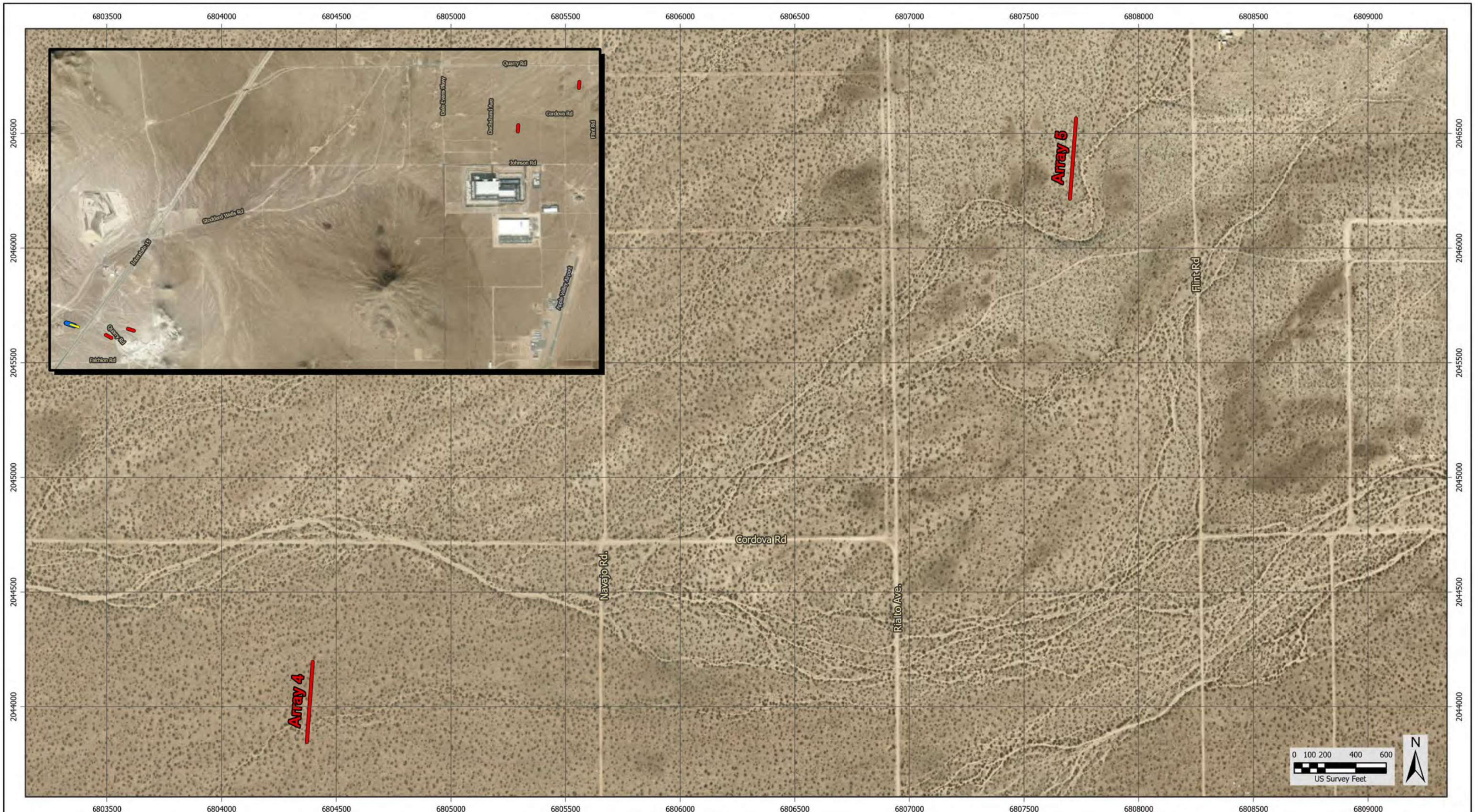
SITE CLASS, Table 20.3-1: $\boxed{C}$

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	0	0
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1325	404
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1100	335
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	1081	329

APPENDIX E

GEOPHYSICAL DATA



MASW Array

Notes:
Coordinate System:NAD 1983 StatePlane California V FIPS 0405 Feet
Base map source: San Bernardino County, Earthstar Geographics, San Bernardino County, Maxar

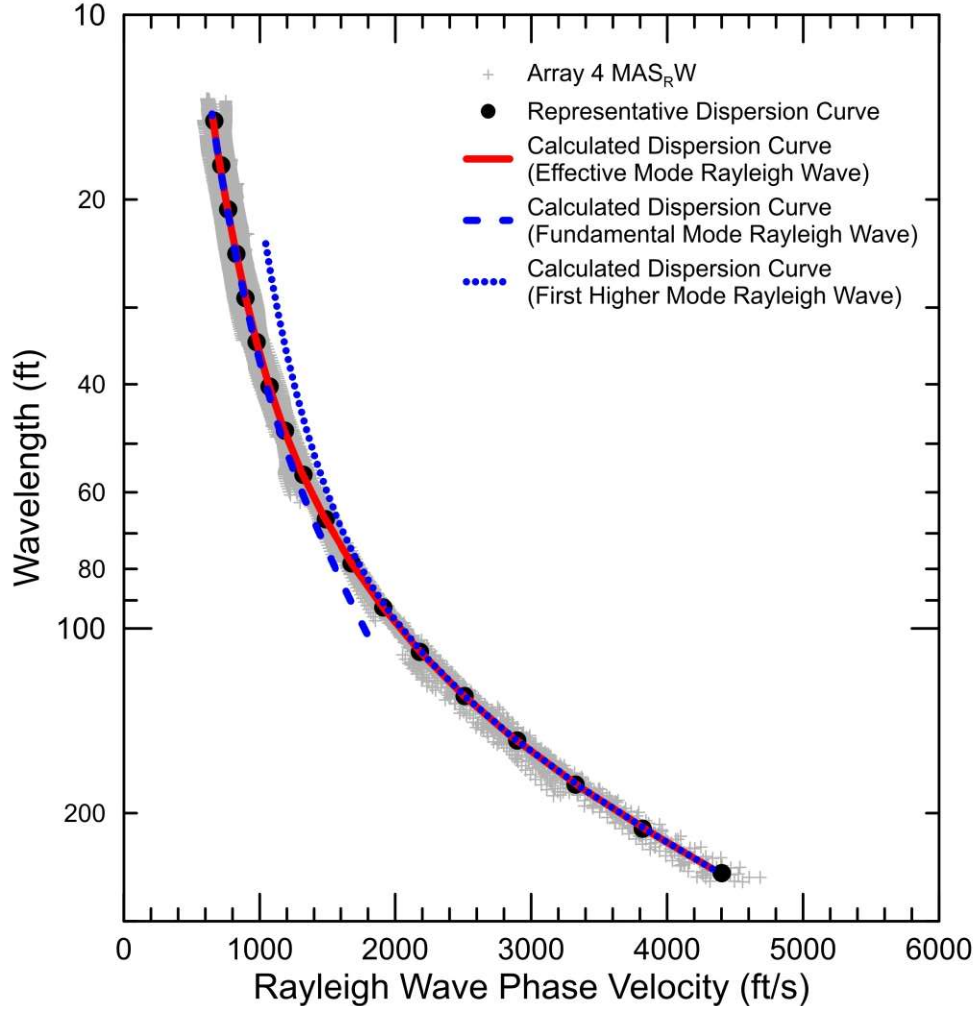
GEOVision geophysical services

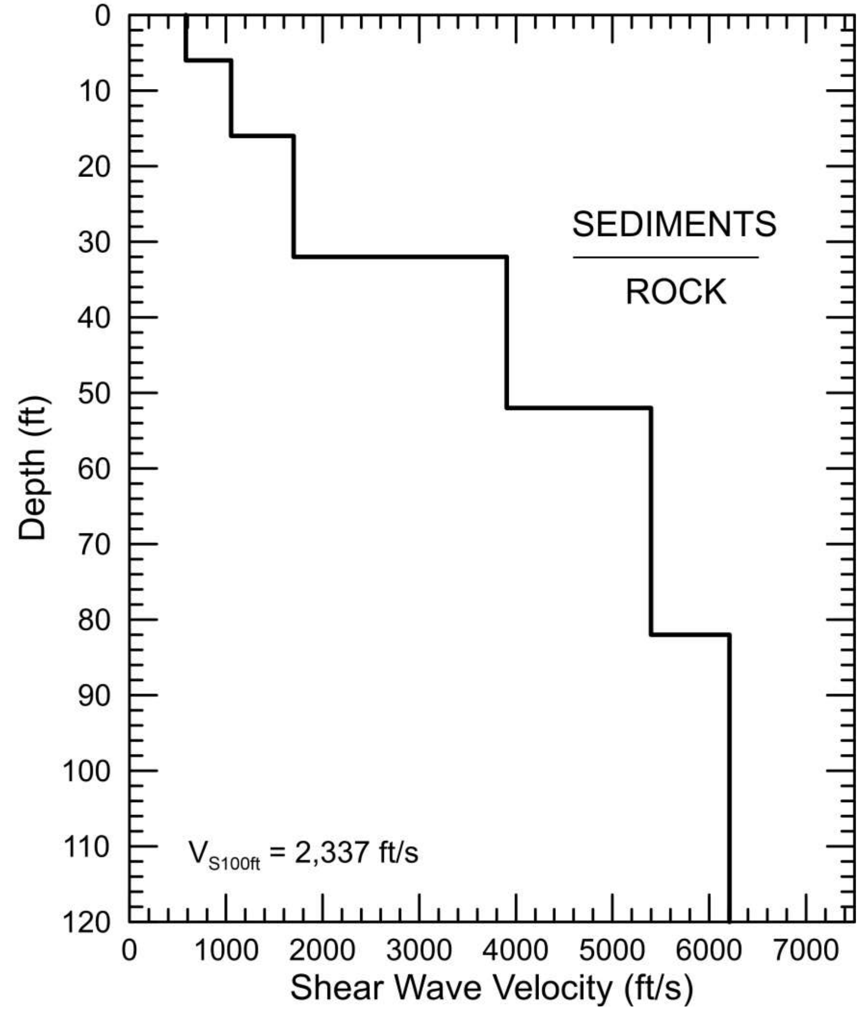
Date:	11/8/2022
GV Project:	22441
Developed by:	A Martin
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	GV-22441

FIGURE 2
SITE MAP

VVLIG CORDOVA ROAD AND QUARRY ROAD WAREHOUSES APPLE VALLEY, CALIFORNIA

PREPARED FOR LEIGHTON CONSULTING, INC.





GEOVision geophysical services	FIGURE 7 SURFACE WAVE MODEL - ARRAY 4
Project No: 22441 Date: NOV 4, 2022	VVLIG CORDOVA ROAD WAREHOUSE APPLE VALLEY, CALIFORNIA
Drawn By: A MARTIN Approved By: Antony J Martin R:\GVProjects\2022\22441 - Leighton\Report\Figure 7.cdr	PREPARED FOR LEIGHTON CONSULTING, INC.

Table 4 Array 3 S-wave Velocity Model (FGFW Parcel A)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Inferred Unit Weight (lb/ft^3)
0.0	4.0	671	1255	0.300	114.0
4.0	6.0	1465	2741	0.300	125.0
10.0	8.0	2369	4430	0.300	131.0
18.0	12.0	2962	5540	0.300	135.0
30.0	16.0	2582	4831	0.300	132.0
46.0	24.0	2397	4486	0.300	131.0
70.0	30.0	2656	4969	0.300	133.0
100.0	Half Space	3037	5679	0.300	135.0

Table 5 Array 4 S-wave Velocity Model (VVLIG Cordova Road Warehouse)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Inferred Unit Weight (lb/ft^3)
0.0	6.0	582	1088	0.300	112.0
6.0	10.0	1054	1972	0.300	120.0
16.0	16.0	1701	3182	0.300	127.0
32.0	20.0	3904	7303	0.300	139.0
52.0	30.0	5399	10100	0.300	144.0
82.0	Half Space	6209	11616	0.300	147.0

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

APPENDIX F

LEIGHTON CONSULTING, INC.

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

D-1.0 GENERAL

D-1.1 <u>Intent</u>

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

D-1.2 <u>Role of Leighton Consulting, Inc.</u>

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

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

D-1.3 <u>The Earthwork Contractor</u>

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

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

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

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

D-2.0 PREPARATION OF AREAS TO BE FILLED

D-2.1 Clearing and Grubbing

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

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D2974). Nesting of the organic materials shall not be allowed.

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

D-2.2 Processing

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

D-2.3 Overexcavation

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

D-2.4 Benching

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

D-2.5 Evaluation/Acceptance of Fill Areas

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

D-3.0 FILL MATERIAL

D-3.1 Fill Quality

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

D-3.2 Oversize

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

D-3.3 Import

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

D-4.0 FILL PLACEMENT AND COMPACTION

D-4.1 Fill Layers

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

D-4.2 Fill Moisture Conditioning

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

D-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D1557 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

D-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D1557 laboratory maximum density.

D-4.5 Compaction Testing

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

D-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document approximate elevation and horizontal coordinates of each density test location, relying on site survey control provided by others. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton Consulting, Inc. can determine test locations with sufficient accuracy. Adequate grade stakes shall be provided.

D-5.0 EXCAVATION

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

D-6.0 TRENCH BACKFILLS

D-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2015 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

D-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least (≥) one-sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

D-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the *Standard Specifications for Public Works Construction* unless the Contractor can demonstrate to Leighton Consulting, Inc. and the Owner that their proposed fill lift can be compacted to the specified relative compaction using the proposed alternative equipment and method; and only if the building official, with the appropriate jurisdiction, approves this proposed lift thickness.

GEOTECHNICAL EXPLORATION PROPOSED INDUSTRIAL WAREHOUSE DEVELOPMENT, ASSESSOR'S PARCEL NUMBER (APN'S) 0463-214-06, 07, 08, AND 09, SOUTHWEST OF QUARRY ROAD AND FLINT ROAD, APPLE VALLEY, SAN BERNARDINO COUNTY, CALIFORNIA

Prepared For **VVLIG HOLDINGS, LLC**
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Project No. 13673.004

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Leighton Consulting, Inc.
A Leighton Group Company

February 1, 2023

Project No. 13673.004

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Attention: Ms. Jessica Haughton

Subject: Geotechnical Exploration
Proposed Industrial Warehouse Development, Assessor's Parcel Number (APN's) 0463-214-06, 07, 08, and 09, Southwest of Quarry Road and Flint Road, Apple Valley, San Bernardino County, California

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has conducted this geotechnical exploration for the proposed industrial warehouse development within Assessor's Parcel Number (APN's) 0463-214-06, 07, 08, 09, located southwest of Quarry Road and Flint Road, in the Town of Apple Valley in San Bernardino County, California. The project site is currently undeveloped, and has an approximate area of 79.4 acres. The purpose of this study has been to collect surface and subsurface geotechnical data at the site with regard to the proposed development, evaluate the proposed development with respect to site geotechnical conditions, and provide geotechnical recommendations for design and construction of the proposed development.

Based on this geotechnical exploration, construction of the proposed warehouse 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 near the surface. 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.

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1.0 INTRODUCTION

1.1 Site Location and Description

The property is approximately 79.4 acres in area and is located southwest of Quarry Road and Flint Avenue, in the Town of Apple Valley, San Bernardino County, California. The project is within Assessor's Parcel Number (APN's) 0463-214-06, 07, 08, and 09.

The site is undeveloped, with vegetation, dirt roads and arroyos transecting the site. Other than a small residential development located directly east towards the northern portion of the site, the surrounding area is also undeveloped with dirt roads present. Quarry Road is at the northern boundary of the site and is the only paved road in the vicinity of the project site. Based on our review of available historical aerial imagery, the area has been undeveloped since 1952, with the residential development to the northeast constructed sometime between 1984 and 1995.

Based on the elevation model of Google Earth and a review of available topographic maps, site elevations (El.) range from approximately El. 3,130 to El. 3,170 feet above mean sea level (msl). The site is relatively flat overall, with local variations in topography from channels and bars typical of this alluvial setting.

1.2 Proposed Development

Our understanding of this project is based on email correspondence with you dated June 17, 2022, and the provided *Overall Site Plan* prepared by LHA Inc., dated June 6, 2022. Based on these, we understand that the proposed warehouse development within the 79.4-acre site consists of a warehouse building with a footprint of 1,540,120 square feet. Also planned are 251 dock doors, 785 auto parking stalls, 615 trailer stalls, drive isles, and underground infiltration facilities. Based on the preliminary grading plans, we understand that the proposed development will include up to 16 feet of fill at the western/southwestern portion of the site and up to 18 feet of design cuts will be required at the eastern portion. Finished pad grade elevation is planned to be approximately El. 3148 feet above msl at the northern end of the building stepping down to approximately 3137 feet above msl at the southern end. A detailed site plan and structural loading were not available at the time of this report. We anticipate that the warehouse will be composed of concrete tilt-up walls.

1.3 Previous Work

Previous geotechnical exploration reports and environmental studies for this site were not available to Leighton for review during the preparation of this report. Leighton is not aware of any previous earthwork activities onsite.

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, infiltration tests, laboratory testing, surface geologic mapping, seismic refraction surveys, 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 and geologic maps and reports and aerial photographs available from our in-house library, available online, or those provided by you.
- Utility Coordination: We contacted Dig Alert (811) prior to excavating borings so that utility companies could mark utilities onsite. We coordinated our work with you and a site representative.
- Field Exploration: A total of eleven (11) hollow-stem auger borings (LB-1 through LB-10 and LB-3A) were logged and sampled onsite on September 19 through 20, 2022 to evaluate subsurface conditions onsite. These borings were drilled by a subcontracted rig to depths ranging from approximately 4 to 50 feet below the existing ground surface (bgs). Relatively undisturbed soil samples were obtained at selected intervals within the borings using a Modified California split barrel sampler lined with rings. Standard Penetration Tests

(SPT) were conducted at selected depths and samples were obtained at those intervals. Representative bulk soil samples were also collected at shallow depths from the borings.

Excavations were backfilled with soil cuttings. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Figure 2, Geotechnical Map.

We conducted well permeameter tests at two locations (LI-1 and LI-2) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. These well permeameter tests were conducted based on the USBR 7300-89 method and in general accordance with San Bernardino County guidelines. Testing consisted of constant head infiltration using a water truck as a source. A 2-inch-diameter slotted PVC pipe was used with sand backfilled around the pipe within the test zone within each boring. LI-1 and LI-2 were conducted at depths ranging from 10 to 15 feet, and 0 to 5 feet bgs, respectively. Boring LI-2 for infiltration testing only extended to a depth of 5 feet due to drilling refusal. Infiltration test logs are included in Appendix B.

Multichannel Analysis of Surface Waves (MASW) was performed along an array within the site to determine the Shear Wave Velocity (V_s) distribution within the subsurface strata.

- 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:

  - Maximum dry density and optimum moisture content
  - In situ moisture content and density
  - Sieve analysis for grain-size distribution
  - Expansion Index
  - Remolded direct shear
  - R-Value
  - Water-soluble sulfate concentration in the soil

- Resistivity, chloride content and pH

Laboratory tests are provided in Appendix C, Laboratory Test Results.

- 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 in the western Mojave Desert, in San Bernardino County California, and is part of the Mojave Desert geomorphic province, a broad interior region of isolated mountain ranges separated by broad desert plains and deep alluvial valleys. The Mojave province is wedged between the Garlock Fault (southern boundary of the Sierra Nevada) and the San Andreas Fault, where it bends northerly from its northwest trend. The northern boundary of the Mojave province is separated from the prominent Basin and Range by the eastern extension of the Garlock Fault.

The geology of the region consists of the following rock groups: i) Surficial sediments (Qa); ii) Older alluvial sediments (Qoa); iii) Granitic and dioritic rocks (qm); iv) Metamorphic rocks (ml, mq, and ms); and v) Metamorphosed quarts latite (mql). The Pre-Tertiary and Tertiary rocks are hard, consolidated materials forming the surrounding mountains and rocky buttes that rise from the valley floors and underlie the alluvium at depths. The valley soil profile consists of up to several hundreds to thousands of feet of fine- to coarse-grained alluvial deposits underlain by consolidated rocks. The alluvial deposits consist of late Pleistocene to Holocene age (5 million years old to recent) fine- to coarse-grained soil layers formed as a result of uplift and erosion of the surrounding mountains. Figure 3, *Regional Geology Map,* presents the site location in relation to the predominate geologic materials (alluvium) of the area. Figure 4, *Regional Fault and Historical Seismicity Map,* presents the site location in relation to active faults and epicenters of relatively large (> M_w 4.0) historical earthquakes.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain mostly by surficial sediments with older alluvial sediments in the center of the site. Artificial fill was not encountered in any of our exploratory borings, which extended to 51.5 feet bgs. Encountered sediment soils of Quaternary Alluvium (Qa) and older alluvium (Qoa) generally consisted of Gravelly Sands (SPg), Silty Sand (SM), Poorly Graded Sand (SP), Sand with Silt (SP-SM), Clayey Sand (SC), and Sandy Silt (ML). At the surface, older alluvium (Qoa) appeared to consist of larger clasts (up to cobble-sized) than younger alluvium (Qa). Several of our borings drilled in older alluvium encountered refusal, which may be an indication that cobbly layers exist buried in that unit. Overall, the soils were very dense based on field Standard Penetration Test (SPT) blowcounts, with a few shallow samples being considered medium dense.

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, native soils found in some of our borings to be medium dense in the upper 2 to 3 feet are considered slightly compressible.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Collapse tests were not performed on the recovered samples due to low sample recovery in very dense soils. Based on the very dense nature of the soils encountered in our borings, onsite soils are not expected to exhibit significant collapse potential.

Soil collapse and consolidation are not a significant issue considering the very dense, granular nature of the onsite soils.

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.

Based on laboratory testing of a representative soil sample and the granular nature of the soils encountered in our borings, the soils onsite are considered to have a "very low" potential for expansion.

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, yielding a sulfate content of less than 0.1 percent by weight. Based on the laboratory test results, the sulfate content of onsite soils is anticipated to be negligible (Exposure Class S0). 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, a representative soil sample was tested during this investigation to determine minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 3,500 ohm-cm, chloride content of 40 ppm, and pH of 7.6. Based on these results, the onsite soil is considered to be moderately corrosive to metals.

2.3 Groundwater

Groundwater was not encountered within our exploratory borings performed on September 19 and 20, 2022.

Review *of California Department of Water Resources Groundwater Wells* included data from multiple groundwater wells approximately 0.1 to 1.2 miles from the site with measurements from 1953 through 1957. Although limited, the readings for the well indicate that groundwater depths have been deeper than 70 feet bgs during the period of groundwater measurements. According to the *Data and Water Table Map of the Mojave River Ground-Water Basin (Stamos and Predmore, 1995), the* groundwater level in 1993 near the project site was deeper than 100 feet.

Based on our review of available groundwater data, groundwater is not expected to be a significant constraint for this project.

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.

The project site has been mapped by the U.S. Geological Society (2022) to be outside of an area of land subsidence from intense removals of significant quantities of water, peat, or oil extraction in the area. Based on this and no known reports indicating land subsidence of the site's area, the potential for ground subsidence is considered very low and less than a significant impact.

2.4 Faulting and Seismicity

In general, 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 towards the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Helendale-South Lockhart fault zone	2.7 miles to the northeast
North Frontal thrust system	11.8 miles to the south
Lenwood-Lockhart fault zone	16.9 miles to the northeast

Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered low.

2.4.2 Seismic Design Parameters

The site has and will 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 seismic parameters should be considered for design under the 2022 edition of the California Building Code (CBC). The following table lists seismic design parameters based on the 2022 CBC and ASCE 7-16 methodology:

Site Seismic Coefficients / Coordinates		Value (g)
Latitude: 34.6122	Longitude: -117.1827	
Site-Specific Analysis (ASCE 7-16)	Spectral Response – Class D (short), S_S	1.03
	Spectral Response – Class D (1 sec), S_1	0.39
	Site Modified Peak Ground Acceleration, PGA_M	0.53
	Max. Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.15
	Max. Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.58
	5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.77
	5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.39
	Maximum Considered Earthquake Geometric Mean MCE_G PGA	0.79

The project structural engineer should review the seismic parameters. Site-Specific analyses output is presented in Appendix D.

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 4.6 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), with a corresponding peak ground acceleration of 0.51g.

2.4.5 Site Class

A geophysical survey line (Array 5) utilizing Multi-channel Analysis of Surface Wave (MASW) methodology was performed towards the central portion of the site (line location shown in Figure 2) and yielded a weighted average shear wave to a depth of 100 feet (V_{S100ft}) of 2,240 ft/s. In addition, we performed an analysis with field Standard Penetration Blowcounts (SPT) from the geotechnical borings that extended to a maximum depth of 50 feet, which yielded a weighted average N-Value of approximately 97 (with blowcount assumptions for soils below 50 feet). In general, SPT blowcounts below 10 feet were 50 blows per less than 4 inches of penetration. Therefore, based on the criteria in the 2022 CBC and ASCE 7-16, the site is classified as Site Class C, very dense soil and soft rock. A summary of Site Class evaluation is included in Appendix D. Geophysical survey data is included in Appendix E.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landslides, 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 due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,
(2) shallow groundwater, and
(3) strong, long-duration ground shaking

The site is mapped within a low liquefaction hazard zone of required investigation on the San Bernardino County General Plan (San Bernardino, 2009).

Due to the very dense nature of the granular soils encountered and lack of shallow groundwater, liquefaction is not a significant hazard at this site.

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.

Based on the very dense nature of the native soils in this area, we believe the onsite soils are susceptible to low seismic settlement (less than 1 inch, with differential settlement of 0.5 inch or less over a horizontal distance of 30 feet based on the MCE).

2.5.3 Lateral Spread

Lateral spread is liquefaction-induced lateral ground movement limited to on the order of several feet, and, thus, smaller than flow failures. A consideration in lateral spread analysis is to evaluate whether laterally continuous liquefiable layers exist. Due to the lack of shallow groundwater (≤50 feet bgs), lateral spread is considered to be less than significant.

2.5.4 Flow Failures

Based on $(N_1)_{60}$ values from the borings and lack of liquefiable soils, the site is not considered susceptible to flow slides (large transitional or rotational failures).

2.5.5 Bearing Failures/Surface Manifestations

We performed an analysis of the potential for bearing failures/structural damage due to liquefaction (surface manifestations) based on the work of Ishihara (1995) and as described in Martin and Lew (1999). This method is based on empirical data and considers the thickness of non-liquefiable soil below the ground surface and foundations, compared to the thickness of underlying liquefiable soils. Due to the lack of liquefiable layers based on our analysis, latera spread is considered to be less than significant.

2.6 Infiltration Testing

Two well permeameter tests (LI-1 and LI-2) were conducted to estimate the infiltration rate at specific locations of the site. Boring LI-1 was located towards the northwest corner of the site, and Boring LI-2 was located towards the southeast region of the site. The locations of the infiltration tests were based on the provided locations of the proposed detention basins in the site plan. The well permeameter tests at LI-1 and LI-2 were conducted inside the drilled borings at depths of 10 to 15 and 0 to 5 feet below the existing ground surface, respectively.

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 is placed in the boring bottom to support temporary perforated well casing pipe. In addition, sand is poured around the outside of the well casing within the test zone to support the boring to reduce caving/collapsing or eroding when water is added. The volume of water percolated during timed intervals is converted into an incremental infiltration rate, which is defined as flow divided by infiltration surface area, in inches per hour. The test was conducted based on the USBR 7300-89 test method.

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)
LI-1	10 to 15	Silty Sand (16% fines)	2.3
LI-2	0 to 5	Silty Sand (24% fines)	1.5

[1] Factor of Safety should be applied to raw rates

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed warehouse 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 warehouses. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking. 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.

We are unaware of environmentally sensitive areas in the project site that would warrant remedial removals from an environmental standpoint. Undocumented fill, if encountered, should be completely removed and properly compacted during earthwork construction. Localized exposures of encountered fill material can be evaluated during grading on a case-by-case basis, and may be left in place if documentation is available and the material appears to be competent based on our field evaluation

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix F, 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.

3.1.2 Removal of Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of the onsite alluvial soil, if any uncontrolled artificial fill is encountered during grading, it should be completely removed and may be used as compacted fill for the project, provided any oversized rock is suitably handled and any deleterious materials are removed from the site. Undocumented fill was not encountered in our subsurface exploration.

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 undocumented artificial fill within the proposed building pad, if encountered during grading, should be removed.

Based on our seismic settlement analysis, we recommend that onsite soils in the proposed building pad area and site walls taller than 8 feet be overexcavated to a minimum depth of 3 feet bgs, or a depth of 2 feet below the bottoms of proposed footings, whichever is deeper.

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. Where this is not achievable, this should be reviewed on a case-by-case basis.

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 proposed structures and planned for new asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), site walls up to 8 feet tall and retaining walls retaining up to 3 feet of soil (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 2 feet 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 D1557 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. The upper 24 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 a laboratory-measured maximum dry density 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 6 +/- 3 percent
Subsidence (overexcavation bottom processing)	Approximately 0.2 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

Although not extracted from our borings, refusal encountered in several of our borings and the existence of cobbles at the surface within areas of mapped older alluvium may indicate that oversized material (rock or rock fragments greater than 8 inches in dimension) may exist in the subsurface. If oversized rock is encountered during grading, it should be placed in deeper fills (deeper than 5 feet below finish grade) or removed from structural fill areas. If encountered, rocks larger than 24 inches in dimension should be placed in windrows, surrounded by sandy soils, and placed with copious amounts of water. The rock windrows should be placed such that individual rocks are not nested and sandy soil can be worked completely around the rocks. It is imperative that the contractor use copious amounts of water.

Excavations for proposed utilities can be very difficult in the presence of large (greater than 24 inches) rocks. To facilitate utility construction (but not a geotechnical requirement), removing rocks larger than 24 inches in the upper 5 feet below the rough graded surface or 1 foot below the deepest utility may be considered.

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 4,500 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 260 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 to be 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 estimated to be 0.5 inch over a horizontal distance of 30 feet for the design-level earthquake, or angular distortion of less than 0.0014L.

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 14 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 1,100 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 Section 2.4.2 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 5 (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	35 pcf
At-Rest	55 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.

For retaining walls with a retained height of more than 6 feet, an incremental seismic load applied as a uniform additive pressure of 17 pcf should be considered for a cantilever (unrestrained) wall with level backfill, and 27 pcf for a basement wall (restrained) with level backfill. This pressure is in addition to the static active earth pressures presented above. Earthquake and at-rest earth pressures need not be combined for analyses.

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 Pavement: Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using a design R-value of 50 based on laboratory testing, 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 (auto access)	3.0	4.0
7 (light truck access)	4.0	4.5
8	5.0	5.0
9	5.5	6.5

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 14 feet on center each way for 7-inch-thick concrete, and 12 feet for 5-inch-thick concrete. 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 Infiltration Recommendations

In general, our geotechnical exploration encountered alluvial soil deposits generally uniform consisting of Gravelly Sands (SPg), Silty Sand (SM), Poorly Graded Sand (SP), and Sand with Silt (SP-SM). Only two borings (LB-7 and LB-8) encountered some Sandy Silt (MLs) within the upper 5 feet. Alluvial soils were relatively uniform throughout the project site. Gravels were observed within the exploratory borings, with variable percentages throughout the site; cobbles are anticipated to be encountered within the mapped older alluvium. At our test locations, sieve analysis tests performed on soil samples from the infiltration test zone generally showed a percent fines (% silt and clay) ranging from 16 to 24 percent.

Based on our infiltration testing, field observations and laboratory testing, the project site is considered to be feasible for groundwater infiltration. A raw infiltration rate of 2.0 inches per hour can be utilized for infiltration system design. As site layout and infiltration system design progresses, supplemental infiltration testing could be performed to further refine our infiltration system recommendations.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with San Bernardino County guidelines, since monitoring of actual facility performance has shown that actual infiltration rates are lower than measured in small-scale tests. Infiltration basins are subject to siltation, which can result in reduced infiltration rates. *This small-scale infiltration rate should be divided by a design factor of at least **3** for buried chambers and at least **4** for open basins; although the design/safety factor may be higher based on project-specific aspects.* It should be noted that during periods of prolonged precipitation, underlying soils tend to become saturated to greater depths/extent. Therefore, infiltration rates tend to decrease with prolonged rainfall.

Some design considerations are presented in the following paragraphs:

- **Adjacent Structure Impact**: As infiltrating water can seep within soil strata partially horizontally, it is important to consider impact that infiltration facilities can play on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these facilities. Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process, but a building setback of at least 15 feet horizontally is initially suggested.

- **Infiltration Basins Type and Geometry**: Further testing may be required depending on final design of infiltration facilities. Infiltration rates are anticipated to vary based on location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. We should review all infiltration plans, including locations and depths of proposed facilities. Further testing may be required depending on infiltration facilities design details, particularly considering type, depth and location.

- **Siltation and Soil Changes**: These infiltration rates are for a clean, un-silted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of soil particles, gradation (uniform versus well graded), particle shape, fines content and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill. For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

- **De-silting Weir/Facilities**: Periodic flow of water carrying sediments into the basin or chamber, plus deposition of fine wind-blown sediments and sediments from erosion of basin side walls, will eventually cause the basin bottom or chamber to accumulate a layer of silt, which has the potential to significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape onsite. 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 be constructed with spillways or other appropriate means that would prevent overfilling that could damage the facility or adjacent improvements.

- **Drainage/Infiltration Time Cycle**: In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with 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. In open basins with compacted or silty bottoms, this could be problematic, in that even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating prolonged open-water safety concern (such as potential for mosquitos and waterborne diseases, algae odor, etc.). In a buried/cover infiltration chamber, these conditions would be of less concern.

- **Maintenance**: Infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented if and as needed. Things to check for include removal of trash or dumping, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained as recommended by the manufacturer or designer. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed.

3.8 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.

3.9 Trench Backfill

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, organic 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 and will allow water to freely permeate. Gravel or rock should not be used for trench backfill without written approval by Leighton. If gravel or open-graded rock is approved and used as bedding or shading, it should be wrapped in Mirafi 140N filter fabric, or equivalent, to prevent surrounding soil from washing into the pore spaces in the gap graded rock. Shading should extend at least 12 inches above the top of the pipe. The bedding/shading materials should be densified in-place by mechanical means, or in accordance with Greenbook specifications.

Subsequent to pipe bedding and shading, backfill soils should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction (ASTMS D1557). The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook). The upper 6 inches in pavement areas should be compacted to 95 percent compaction.

3.10 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.

3.11 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 moderately 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.

3.12 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 VVLIG Holdings, LLC, for application to the design of the proposed warehouse buildings 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

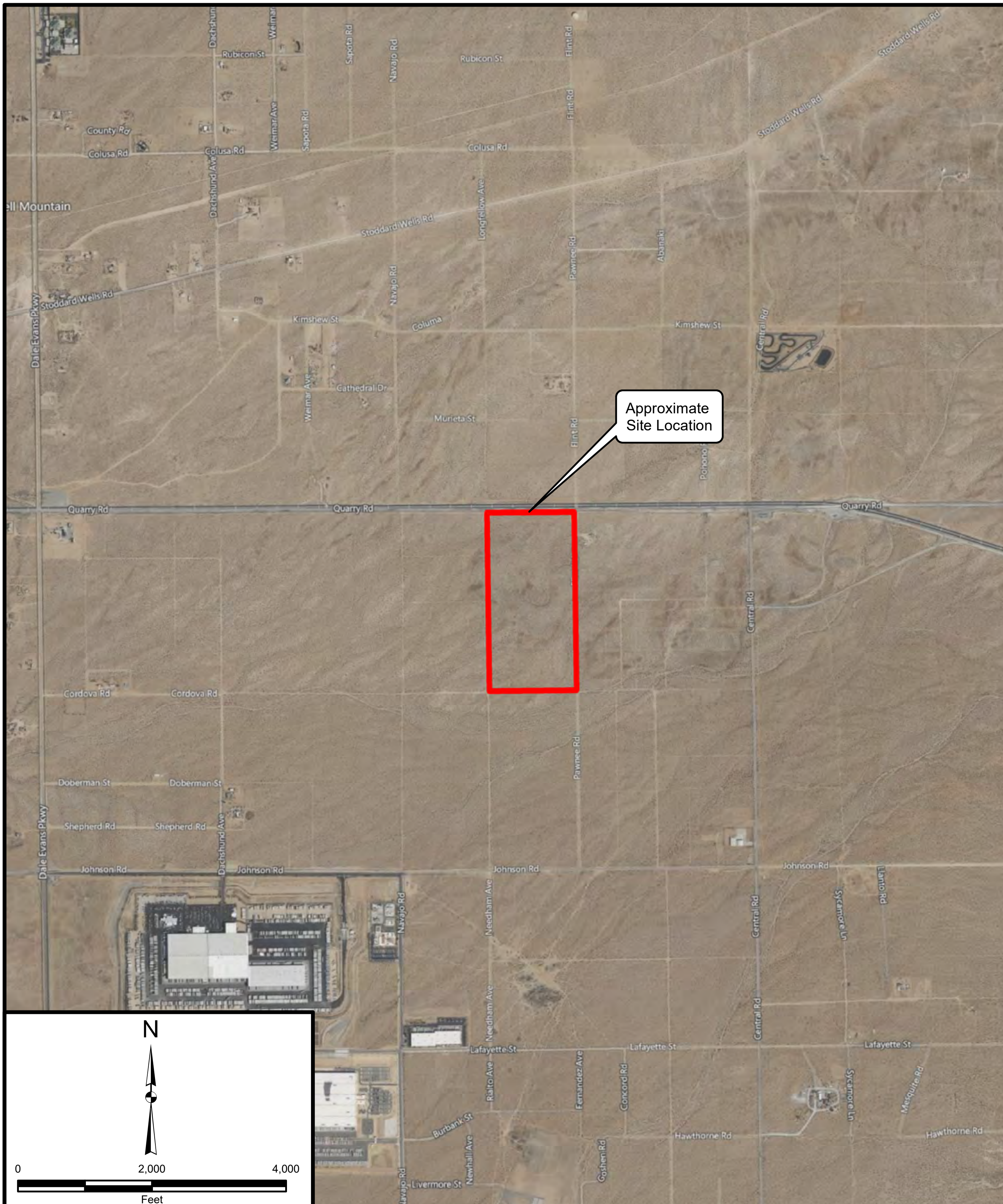
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate 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.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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



Project: 13673.004

Eng/Geol: JDH/SGO

Scale:1 " = 2,000 '

Date: February 2023

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

SITE LOCATION MAP

Proposed Warehouse Building Development
Parcel B APNs 0463-213-05, 07, 08, 09, 16, 33, 34, 35, and 36
Apple Valley, California

FIGURE 1

Leighton

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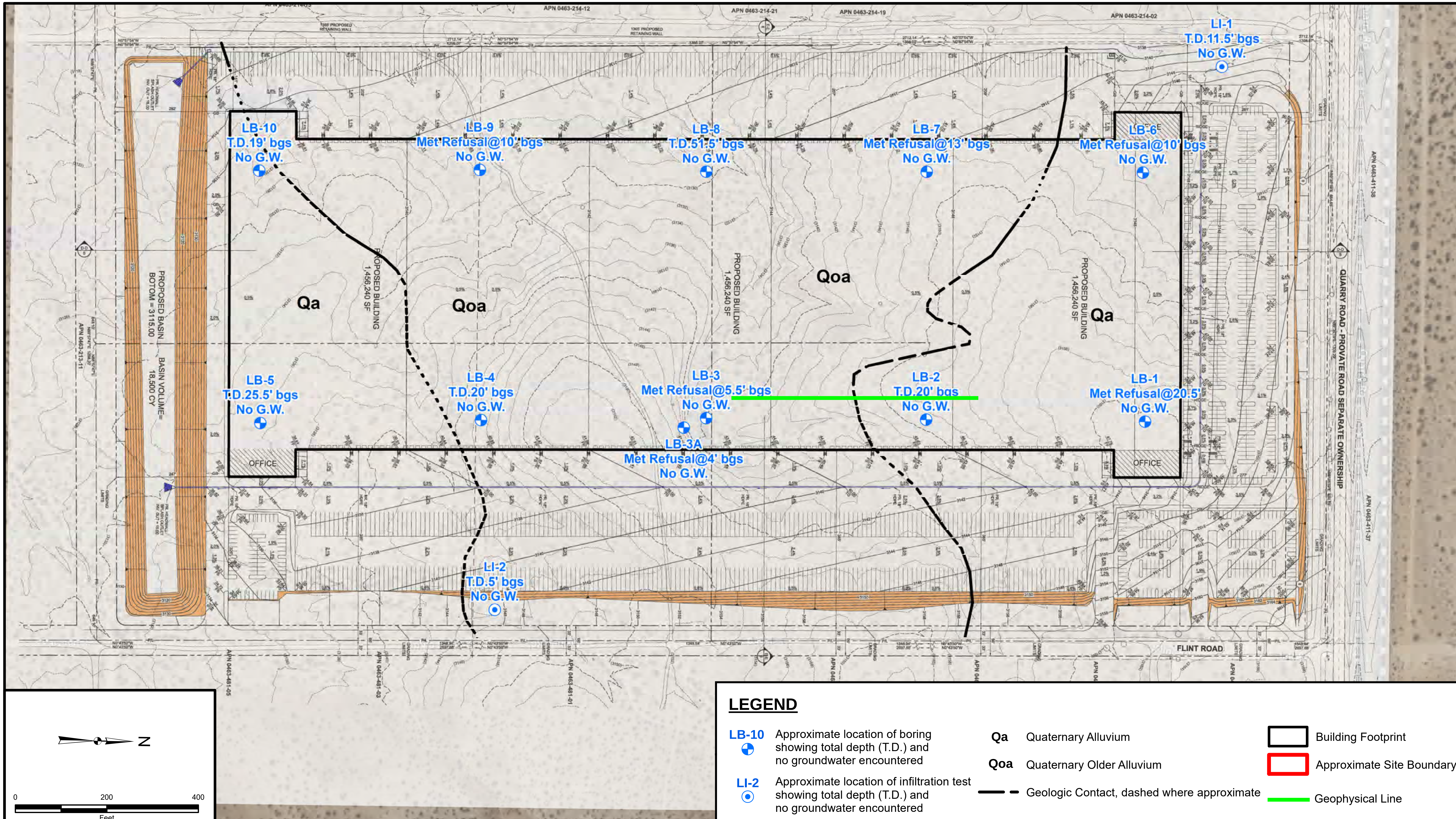

LEGEND

LB-10	Approximate location of boring showing total depth (T.D.) and no groundwater encountered	Qa	Quaternary Alluvium		Building Footprint
LI-2	Approximate location of infiltration test showing total depth (T.D.) and no groundwater encountered	Qoa	Quaternary Older Alluvium		Approximate Site Boundary
			Geologic Contact, dashed where approximate		Geophysical Line

Project: 13673.004 | Eng/Geol: JDH/SGO

Scale: 1 " = 200 ' | Date: February 2023

Base Map: Site Plan Review, Conceptual Grading and Drainage Sheets 2-4 of 9 by David Evans and Associates, Inc.
Author: (btran)

GEOTECHNICAL MAP

Proposed Warehouse Building Development
Parcel B APNs 0463-213-05, 07, 08, 09, 16, 33, 34, 35, and 36
Apple Valley, California

FIGURE 2

Leighton

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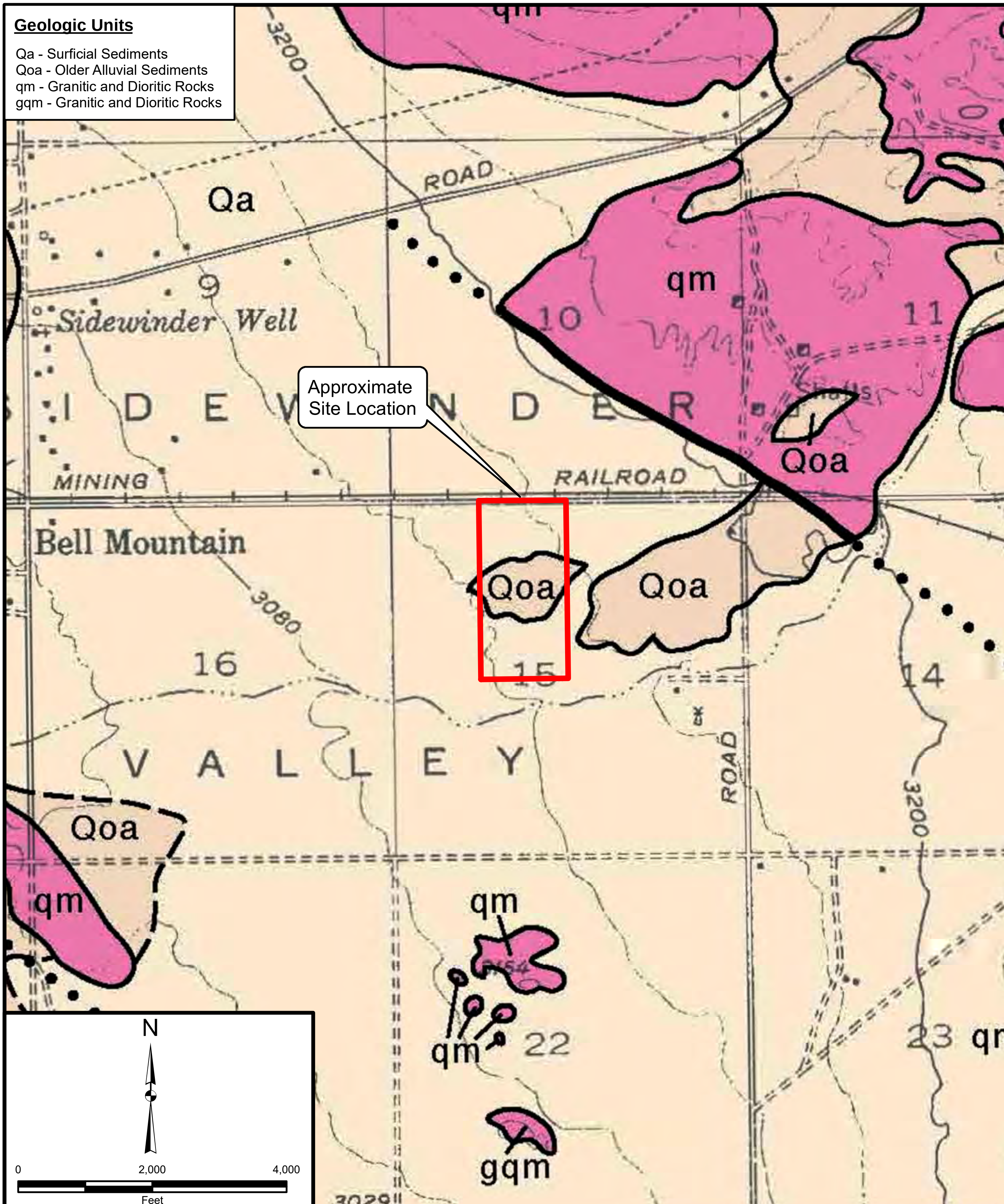

Project: 13673.004	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: February 2023

Reference:
Geologic Map of the Apple Valley & Old Mountains
by Thomas W. Dibblee, JR., 2008

REGIONAL GEOLOGY MAP

Proposed Warehouse Building Development
Parcel B APNs 0463-213-05, 07, 08, 09, 16, 33, 34, 35, and 36
Apple Valley, California

FIGURE 3

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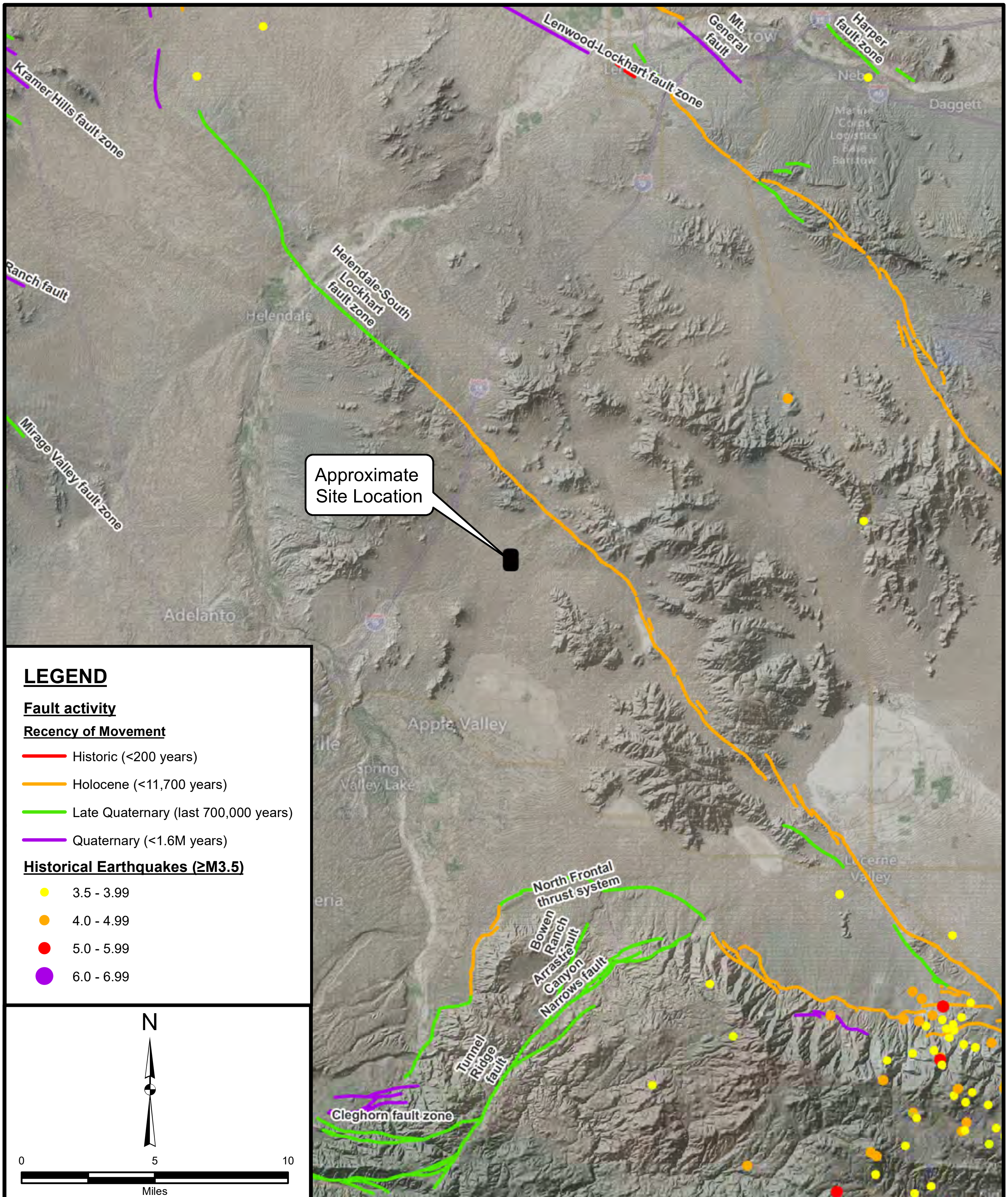
Map Saved as J:\Drafting\13673\004\Maps\13673-004_F03_RGM_2022-09-27.mxd on 9/27/2022 10:50:16 AM Author: KVM (btran)



Project: 13673.004	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: February 2023

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

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Warehouse Building Development
Parcel B APNs 0463-213-05, 07, 08, 09, 16, 33, 34, 35, and 36
Apple Valley, California

FIGURE 4

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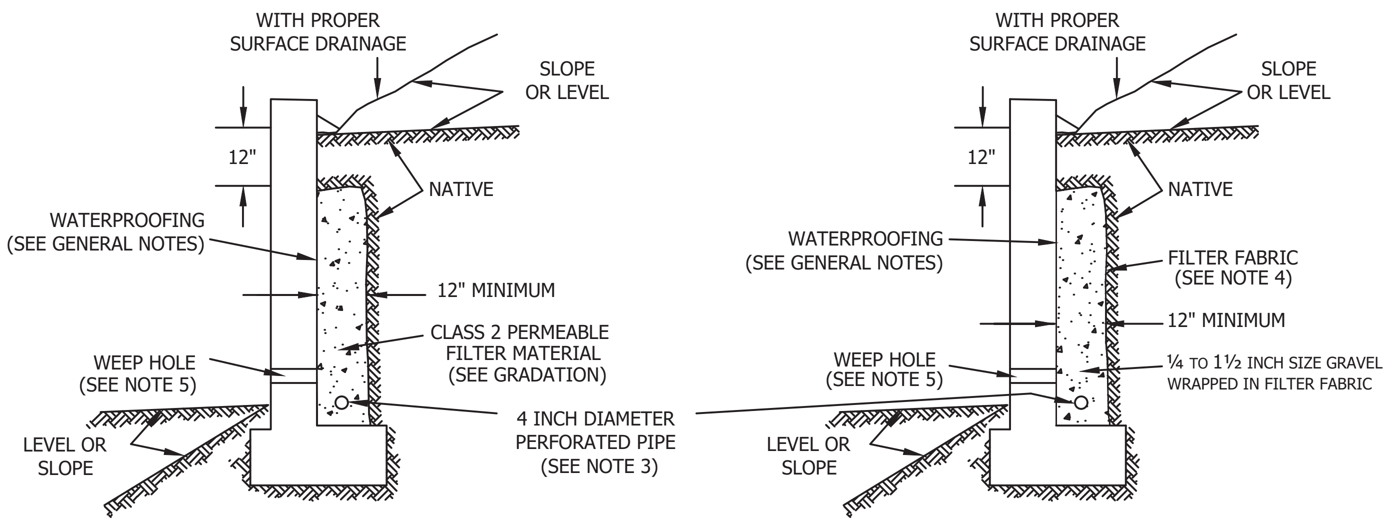
Map Saved as J:\Drafting\13673\004\Maps\13673-004_F04_RFHSM_2022-09-27.mxd on 9/27/2022 10:12:06 AM Author: KVM (btran)

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

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Figure 5

APPENDIX A

REFERENCES

APPENDIX A

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APPENDIX B

GEOTECHNICAL LOGS

APPENDIX B

FIELD EXPLORATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration. Approximate exploration locations are shown on Figure 2, *Geotechnical Map*.

Borings: On September 19 and 20, 2022, 13 hollow-stem-auger borings (LB-1 through LB-10 and IT-1 through IT-2) were drilled, logged and sampled to depths ranging from 4 feet to 50 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 Modified 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.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Quaternary Alluvium (Qa)** @Surface: SILTY SAND with gravel (SM)g, brown, dry, fine to coarse sand, 20% fines (field estimate), 30% gravel (field estimate)	
				R-1	13 17 46			SM	@2.5': SILTY SAND (SM), dense, white, dry, fine to medium sand, 13% fines (field estimate), (lab), loose sample	-200
	5			R-2	23 43 50/2"	99	3	SM	@5': SILTY SAND (SM), very dense, white, dry, fine to medium sand, 13% fines, (lab), auger grinding	
				S-1	18 50/6"			SM	@7.5': SILTY SAND (SM), very dense, white, dry, fine to medium sand, 13% fines (lab), auger grinding @9': Auger grinding heavily, cobble and gravel found in cuttings	
	10			S-2	18 22 34			SPg	@10': Poorly graded GRAVELLY SAND (SPg), very dense, grayish, slightly moist, fine to coarse gravel, slight cementation, auger continues to grind	
	15			S-3	28 50/5"			SPg	@15': Poorly graded GRAVELLY SAND (SPg), very dense, brown, slightly moist, fine to coarse gravel, slight cementation, auger grinding, gravel and cobbles found in cuttings	
	20			S-4	50/5"			SPg	@20': Poorly graded GRAVELLY SAND (SPg), very dense, brown, slightly moist, fine to coarse gravel, slight cementation, auger grinding, met refusal **TOTAL EXPLORED DEPTH = 20.41 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	25									
	30									

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

TYPE OF TESTS:

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

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GEOTECHNICAL BORING LOG LB-2

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

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

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**Quaternary Alluvium (Qa)**	EI, RV
									@Surface: SILTY SAND (SM), brown, dry, fine to coarse sand, trace fine gravel, ~20-25% fines (field estimate)	
				R-1	30 50/6"	107	1	SM	@2.5': SILTY SAND with gravel (SM)g, very dense, orange brown, dry to slightly moist, mostly fine sand, trace medium to coarse, pockets of mostly coarse sand, trace fine subangular gravel, 19% fines (lab)	
	5			R-2	9 50/4"	102	5	SM	@5': SILTY SAND with gravel (SM)g, very dense, pale brown to tan, dry to slightly moist, fine sand, some medium to coarse, 19% fines (lab)	-200
				R-3	50/4"			SM	@7.5': No Recovery	
	10			R-4	50/4"			SW-SM	@10': SAND with silt (SW-SM), very dense, light brown, dry to slightly moist, fine to coarse sand, 8% fines (field estimate), micaceous weathered granite, chunk of intact granite in sampler; grades coarser toward bottom	-200
	15			S-5	50/4"			SM	@15': SILTY SAND (SM), very dense, gray, dry to slightly moist, very fine to coarse sand, ~20% fines (field estimate), micaceous	
	20			S-6	50/2"			SM	@20': No Recovery	
									TOTAL EXPLORED DEPTH = 20.33 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	25									
	30									

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

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

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GEOTECHNICAL BORING LOG LB-3

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								**Quaternary Older Alluvium (Qoa)**	
									@Surface: SILTY SAND (SM), brown, dry, mostly fine sand, some medium to coarse sand, ~30% fines (field estimate)	
				R-1	35 50/1"			SM	@2.5': No Recovery, heavy rig chatter, auger grinding	
									@4': Refusal @4 ft due to gravel/cobbles, stepped out ~5 ft north	
	5			R-2	50/1"			SM	@5': No Recovery, heavy rig chatter, auger grinding	
									TOTAL EXPLORED DEPTH = 5.5 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE<)>>**	
	10									
	15									
	20									
	25									
	30									

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

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

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

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GEOTECHNICAL BORING LOG LB-3A

Project No.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

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			B-1				SP	**Quaternary Older Alluvium (Qoa)** @Surface: Poorly graded SAND with gravel (SP), brown, dry, fine to coarse sand, 40% gravel (field estimate)	SA
				R-1	25 50/5"	103	3	SM	@2.5': SILTY SAND (SM), very dense, white, dry	
	5								**TOTAL EXPLORED DEPTH = 4 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE<)>>**	
	10									
	15									
	20									
	25									
	30									

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

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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

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								**Quaternary Older Alluvium (Qoa)**	
									@Surface: SILTY SAND (SM), orange brown, dry, very fine to fine sand, trace medium to coarse sand, fine gravel, ~30% fines	
				R-1	42 50/5"			SC	@2.5': CLAYEY SAND (SC), very dense, orange to orange brown, dry to slightly moist, mostly coarse sand, trace fine to medium, heavily weathered granite	
	5			R-2	50/5"	103	2	SM	@5': SILTY SAND (SM), very dense, light brown, dry to slightly moist, mostly fine sand, some medium and coarse, ~10-15% fines (field estimate), heavily weathered granite	
				R-3	50/5.5"			SW-SM	@7.5': SAND with silt (SW-SM), very dense, grayish brown, dry to slightly moist, fine to coarse sand, micaceous, grading finer toward bottom of sampler SILTY SAND (SM), 6% fines (lab)	-200
	10			R-4	50/4"			SM	@10': SILTY SAND (SM), very dense, gray, dry to slightly moist, mostly fine sand, some medium to coarse, ~15% fines (field estimate), fine gravel, sized piece of intact, heavily weathered granite	
	15			S-5	50/4"			SW-SM	@15': SAND with silt (SW-SM), very dense, gray, dry to slightly moist, fine to coarse sand, 8% fines (lab)	-200
	20			S-6	50/3"				@20': NO RECOVERY	
									TOTAL EXPLORED DEPTH = 20.25 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE<)>>**	
	25									
	30									

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

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

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GEOTECHNICAL BORING LOG LB-5

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**Quaternary Alluvium (Qa)** @Surface: SILTY SAND (SM), orange, dry, mostly fine sand, some medium to coarse, ~30% fines (field estimate)	
				R-1	50/6"			SP	@2.5': SAND (SP), very dense, orange brown, dry, mostly fine to medium sand, trace coarse, some silt, portions slightly cemented	
	5			R-2	17 31 48	109	3	SM	@5': SILTY SAND (SM), very dense, yellowish brown to tan, dry to slightly moist, mostly fine sand, trace medium to coarse, trace fine gravel	
				R-3	30 50/5"	114	2	SM	@7.5': SILTY SAND (SM), very dense, pinkish tan, dry, fine to coarse sand, ~10-15% fines (field estimate)	
	10			R-4	50/5"			SP-SM	@10': SAND with silt (SP-SM), very dense, pinkish tan, dry, fine to coarse sand, ~5-10% fines (field estimate), grading finer to SILTY SAND (SM), very dense, tan, very fine to fine sand, trace medium to coarse, ~35% fines (field estimate)	
	15			S-5	21 50/6"			SM	@15': SILTY SAND (SM), very dense, whitish tan, dry, mostly fine sand, trace medium to coarse, weathered granite, ~23% fines (lab), some CaO2 lenses	-200
	20			R-6	50/3"	107	5	SM	@20': SILTY SAND with gravel (SM)g, very dense, tan, dry, mostly fine sand, trace medium to coarse sand, trace fine gravel, weathered granite, ~15% fines (field estimate)	
	25			S-7	50/5"			SM	@25': SILTY SAND (SM), very dense, tan, dry, mostly fine sand, trace medium to coarse sand, trace fine gravel, weathered granite, ~15% fines (field estimate), poor recovery	
	30									

TOTAL EXPLORED DEPTH = 25.41 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH SOIL CUTTINGS TO SURFACE<)>>

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

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

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GEOTECHNICAL BORING LOG LB-6

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				ML	**Quaternary Alluvium (Qa)** @Surface: SANDY SILT with gravel (ML), loose, light brown, slightly moist, fine sand, some fine gravel, subangular	
				R-1	37 50/6"			ML	@2.5': SANDY SILT (ML), hard, yellow brown to tan, dry to slightly moist, fine to coarse sand, trace fine gravel, subangular, ~75% fines (field estimate)	
	5			R-2	28 50/6"	115	3	SW-SM	@5': SAND with gravel (SW-SM)g, very dense, yellow brown to tan, dry to slightly moist, fine to coarse sand, fine gravel, subangular, 7% fines (lab), grading to SILTY SANDY GRAVEL (GM), very dense, light to red brown, fine to coarse sand, fine gravel	-200
				R-3	45 50/3"			SM	@7.5': SILTY SAND with gravel (SM)g, very dense, light brown, dry to slightly moist, fine to coarse sand, fine gravel, subrounded, angular, broken bits of gravel	
	10				50/0				@10': NO RECOVERY **TOTAL EXPLORED DEPTH = 30.25 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE<)>>**	
	15									
	20									
	25									
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-7

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

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								**Quaternary Older Alluvium (Qoa)**	
									@Surface: SILTY SAND with gravel (SM)g, light brown, slightly moist, mostly fine sand, trace medium to coarse, some fine gravel, subangular	
				R-1	24 22 21			ML	@2.5': SANDY SILT (ML), very stiff, tan, dry to slightly moist, fine to coarse sand, fine gravel, subrounded, rig chatter	
	5			R-2	16 22 28	100	3	SW-SM	@5': SAND with silt (SW-SM), dense, light brown to orangish brown, dry to slightly moist, fine to coarse sand, trace silt, trace fine gravel, some portions slightly cemented, 6% fines (lab), rig chatter	-200
				R-3	50/6"	106	2	SM	@7.5': SILTY SAND (SM), very dense, light brown, dry to slightly moist, fine sand, some medium to coarse, ~15% fines (field estimate), large coarse gravel at bottom of sampler, sample disturbed, heavy rig chatter	
	10			R-4	41 50/2"			SP	@10': Poorly graded SAND (SP), very dense, orange brown, slightly moist, mostly fine sand, trace medium to coarse, trace fine gravel, some CaO2 lenses, friable, heavy rig chatter	-200
	15								**TOTAL EXPLORED DEPTH = 13 FEET** **REFUSAL DUE TO COBBLES/GRAVEL** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	20									
	25									
	30									

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

TYPE OF TESTS:

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

Leighton

GEOTECHNICAL BORING LOG LB-8

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Quaternary Older Alluvium (Qoa)** @Surface: SILTY SAND (SM)	SA, MD, DS, CR
				R-1	27 21 18	119	1	SM	@2.5': SILTY SAND (SM), medium dense, orange brown, dry to slightly moist, fine to coarse sand, trace fine gravel, subrounded, ~15% fines (field estimate)	
	5			R-2	50/5"			SM	@5': No Recovery	
				R-3	50/5"			SM	@7.5': SILTY SAND (SM), very dense, light brown, fine to coarse sand, trace fine gravel, subrounded, ~15% fines (field estimate), No Recovery	
	10			R-4	50/6"	110	2	SM	@10': SILTY SAND (SM), very dense, light brown to grayish brown, mostly fine sand, trace medium to coarse sand, few silt, trace fine gravel, grading coarser to mostly medium to coarse sand, trace fine sand, trace silt, chunks of weathered granite	
	15			S-5	31 50/1"			SW-SM	@15': SAND with silt (SW-SM), very dense, light brown to tan, dry to slightly moist, mostly fine sand, trace medium to coarse, 12% fines (lab), trace CaO2 spots, friable	-200
	20			R-6	50/6"	111	2	SM	@20': SILTY SAND (SW-SM), very dense, light brown to tan, dry to slightly moist, mostly fine sand, trace medium to coarse, ~15% fines (field estimate), trace CaO2 spots, friable, slightly coarser (localized)	
	25			S-7	50/5"			SM	@25': SILTY SAND (SM), very dense, light brown to tan, dry to slightly moist, mostly fine sand, trace medium to coarse, ~15% fines (field estimate), trace CaO2 spots, friable, slightly coarser (localized), No Recovery	
	30									

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

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

Leighton

GEOTECHNICAL BORING LOG LB-8

Project No.	13673.004	**Date Drilled**	9-19-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	BTM

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			R-8	50/1"			SM	@30': SILTY SAND (SM), very dense, light brown, dry to slightly moist, mostly fine sand, trace medium to coarse, ~20% fines (field estimate), trace fine gravel, chunks of heavily weathered granite, micaceous	
	35			S-9	50/3"			SM	@35': SILTY SAND (SM), very dense, light brown, dry to slightly moist, mostly fine sand, trace medium to coarse, ~20% fines (field estimate), trace fine gravel, chunks of heavily weathered granite, micaceous	
	40			R-10	50/3"			SW-SM	@40': SAND with silt (SW-SM), very dense, brown to grayish brown, slightly moist, fine to coarse sand, heavily weathered granite, 6% fines (lab), micaceous	-200
	45			S-11	50/3"			SW-SM	@45': No Recovery	
	50			R-12	50/1"			SW-SM	@50': No Recovery	
									TOTAL EXPLORED DEPTH = 50.08 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	55									
	60									

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

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

Leighton

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GEOTECHNICAL BORING LOG LB-9

Project No.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

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								**Quaternary Older Alluvium (Qoa)**	
									@Surface: Poorly graded SAND with silt (SP-SM), dry, brown, medium to coarse sand, 10% fines (field estimate), 25% gravel (field estimate)	
				R-1	50/3"			SP-SM	@2.5': No Recovery, slight auger chatter	
	5			R-2	50/4"			SP-SM	@5': Cuttings; Poorly graded SAND with silt and gravel (SP-SM), very dense, gray, slightly moist, medium and coarse, 10% fines (field estimate), 40% gravel (field estimate), slight rig chatter	
				S-1	50/3"			SP-SM	@7.5': Poorly graded SAND with silt and gravel (SP-SM), very dense, gray, slightly moist, medium and coarse, 10% fines (field estimate), 40% gravel (field estimate)	
	10			S-2	50/1"			SP-SM	@10': Poorly graded SAND with silt and gravel (SP-SM), very dense, gray, slightly moist, fine and coarse, 10% fines (field estimate), 40% gravel (field estimate) 1 inch recovery **MET REFUSAL at 10 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	15									
	20									
	25									
	30									

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

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

Leighton

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GEOTECHNICAL BORING LOG LB-10

Project No.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

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			B-1				SP	**Quaternary Alluvium (Qa)** @Surface: Poorly graded SAND with gravel (SP), brown, dry, fine to coarse, 20% gravel (field estimate)	
				R-1	43 50/6"	97	3	SM	@2.5': SILTY SAND with gravel (SM)g, very dense, white, dry, fine to coarse sand, 15% fines (field estimate), 20% gravel (field estimate)	
	5			R-2	27 50/5"	94	3	SW-SM	@5': SAND with silt (SW-SM), very dense, white to gray, slightly moist, coarse sand, 9% fines (lab)	-200
					50/4"				@7.5': No Recovery	
	10			S-1	50/6"			SP-SM	@10': Poorly graded SAND with silt and gravel (SP-SM), very dense, grayish brown, slightly moist, medium to coarse sand, 7% fines (field estimate), 35% gravel (field estimate), poor recovery	
	15			S-2	50/3"			SP-SM	@15': Poorly graded SAND with silt and gravel (SP-SM), very dense, grayish brown, slightly moist, medium to coarse sand, 7% fines (field estimate), 35% gravel (field estimate), auger grinding, partial recovery (4-inches)	
									@19': met refusal @ 19 ft	
	20								**TOTAL EXPLORED DEPTH = 19 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	25									
	30									

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

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

GEOTECHNICAL BORING LOG LI-1

Project No.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

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								**Quaternary Alluvium (Qa)**	
									@Surface: Poorly graded SAND with silt and gravel (SP-SM), brown, dry, fine to coarse sand, 10% fines (field estimate), 40% gravel (field estimate)	
	5									
	10			S-1	15 27 50/4"			SM	@10.5': SILTY SAND with gravel (SM)g, very dense, grayish brown, slightly moist, fine to coarse sand, 16% fines, 20% gravel (lab)	SA
									TOTAL EXPLORED DEPTH = 11.5 FEET **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @ 10FT**	
	15									
	20									
	25									
	30									

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

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

Leighton

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GEOTECHNICAL BORING LOG LI-2

Project No.	13673.004	**Date Drilled**	9-20-22
Project	Synergy Warehouses - Quarry Road	**Logged By**	AA
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	AA

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			B-1				SM	**<u>Quaternary Older Alluvium (Qoa)</u>**	SA
									@Surface: Poorly graded SAND with gravel (SP), brown, dry, medium and coarse sand, 40% gravel (field estimate)	
								SM	@2.5': SILTY SAND (SM), light brown, dry, fine to coarse sand, trace gravel, 24% fines (lab)	
	5								**TOTAL EXPLORED DEPTH = 5 FEET** **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @5 FT**	
	10									
	15									
	20									
	25									
	30									

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

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

Leighton

Results of Well Permeameter, from USBR 7300-89 Method

Leighton 60 Anniversary

Project: 13673.004
Exploration #/Location: *LI-1*
Depth Boring drilled, bgs (ft): 10.5
Tested by: AA
USCS Soil Type in test zone: SM / SP-SM
Weather (start to finish): Sunny
Water Source/pH: H2O
Measured boring diameter: 8 in. — 4 in. Well Radius
Depth to GW or aquitard, bgs: 100 ft
Well Prep: Drill to 5', hit refusal, set 5' screen, sand backfilll in test zone

Initial estimated Depth to Water Surface (in.): 95
Average depth of water in well, "h" (in.): 31
approx. h/r: 7.6
Tu (Fig. 8) (ft): 92.0
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity: 0.4
Casing outer diameter, in.: 2.3
Casing inner diameter, in.: 2.1
Cross-sectional area, in.^2: 21.9

Use of Barrels: No
Use of Flow Meter: Yes
Test Type: Constant Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surfa	10. ft	0. in.	120
Casing stickup measured above top of auger (or ground surface) (+ i	0. ft	-6. in.	-6
Depth to top of sand from top of casing			

Depth of well bottom below top of casing (in): 114

Flow Meter ID: 2497 Meter Units: Gallons 0.05 gallons/pulse
Data logger ID:

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	Depth to WL in Boring in.	Water Temp (deg F)	Refilled? (or 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)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/29/2022	Start time: 14:29	Gallons		ft	in.																	
9/29/22	14:29	1736.91		7.03					0	90.4	29.6											
9/29/22	14:31	1737.22		7.07				2	2	90.8	29.2	-0.48	29	72	11	82	41	2464	789	0.9	0.77	2.88
9/29/22	14:35	1737.84		7.11				4	6	91.3	28.7	-0.48	29	143	11	154	38	2306	777	0.9	0.74	2.74
9/29/22	14:45	1739.35		7.07				10	16	90.8	29.2	0.48	29	349	-11	338	34	2030	777	0.9	0.63	2.41
9/29/22	14:55	1740.87		7				10	26	90.0	30.0	0.84	30	351	-18	333	33	1996	794	0.9	0.59	2.32
9/29/22	15:05	1742.39		6.95				10	36	89.4	30.6	0.6	30	351	-13	338	34	2028	812	0.9	0.59	2.30
9/29/22	15:15	1743.9		6.92				10	46	89.0	31.0	0.36	31	349	-8	341	34	2046	824	0.9	0.58	2.29
9/29/22	15:25	1745.4		6.9				10	56	88.8	31.2	0.24	31	347	-5	341	34	2047	831	0.9	0.58	2.27
9/29/22	15:35	1746.92		6.87				10	66	88.4	31.6	0.36	31	351	-8	343	34	2059	839	0.9	0.57	2.26
9/29/22	15:45	1748.43		6.86				10	76	88.3	31.7	0.12	32	349	-3	346	35	2077	845	0.9	0.57	2.27
9/29/22	15:55	1749.94		6.85				10	86	88.2	31.8	0.12	32	349	-3	346	35	2077	848	0.9	0.57	2.26
9/29/22	16:05	1751.48		6.83				10	96	88.0	32.0	0.24	32	356	-5	350	35	2103	853	0.9	0.57	2.27
																			Minimum Rate:			2.3
														Raw Rate for design, prior to application of adjustment factors:								2.3

Results of Well Permeameter, from USBR 7300-89 Method

Project: 13673.004

Exploration #/Location:	LI-2	Initial estimated Depth to Water Surface (in.):	21
Depth Boring drilled, bgs (ft):	5	Average depth of water in well, "h" (in.):	39
Tested by:	AA	approx. h/r:	9.7
USCS Soil Type in test zone:	SM / SP-SM	Tu (Fig. 8) (ft):	98.2
Weather (start to finish):	Sunny	Tu>3h?:	yes, OK
Water Source/pH:	H2O		
Measured boring diameter:	8 in.	4 in. Well Radius	
Depth to GW or aquitard, bgs:	100 ft		
Well Prep:	Drill to 5', hit refusal, set 5' screen, sand backfilll in test zone		

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity	0.4
Casing outer diameter, in.	2.3
Casing inner diameter, in.	2.1
Cross-sectional area, in.^2	21.9

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surfa	5. ft	0. in.	60
Casing stickup measured above top of auger (or ground surface) (+ i	0. ft	0. in.	0
Depth to top of sand from top of casing			

Depth of well bottom below top of casing (in): 60

Use of Barrels: No
Use of Flow Meter: Yes
Test Type: Constant Head

Flow Meter ID: 2497 Meter Units: Gallons 0.05 gallons/pulse Data logger ID:

Field Data | **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or 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)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date: 9/29/2022	Start time: 11:53	Gallons																				
9/29/22	11:53	1712.22		2.72					0	32.6	27.4											
9/29/22	11:55	1712.73		2.44				2	2	29.3	30.7	3.36	29	118	-74	44	22	1326	780	0.9	0.37	1.57
9/29/22	12:00	1713.52		2.25				5	7	27.0	33.0	2.28	32	182	-50	133	27	1590	851	0.9	0.40	1.72
9/29/22	12:05	1714.22		2.11				5	12	25.3	34.7	1.68	34	162	-37	125	25	1499	901	0.9	0.36	1.53
9/29/22	12:15	1715.62		1.89				10	22	22.7	37.3	2.64	36	323	-58	266	27	1593	955	0.9	0.34	1.54
9/29/22	12:25	1717.05		1.74				10	32	20.9	39.1	1.8	38	330	-39	291	29	1745	1011	0.9	0.34	1.59
9/29/22	12:35	1718.42		1.66				10	42	19.9	40.1	0.96	40	316	-21	295	30	1773	1046	0.9	0.34	1.56
9/29/22	12:45	1719.81		1.6				10	52	19.2	40.8	0.72	40	321	-16	305	31	1832	1067	0.9	0.34	1.58
9/29/22	12:55	1721.22		1.54				10	62	18.5	41.5	0.72	41	326	-16	310	31	1860	1085	0.9	0.34	1.58
9/29/22	13:05	1722.62		1.52				10	72	18.2	41.8	0.24	42	323	-5	318	32	1909	1097	0.9	0.34	1.60
9/29/22	13:18	1724.43		1.47				13	85	17.6	42.4	0.6	42	418	-13	405	31	1869	1107	0.9	0.33	1.56
9/29/22	13:29	1725.91		1.45				11	96	17.4	42.6	0.24	42	342	-5	337	31	1836	1118	0.9	0.32	1.51
9/29/22	13:40	1727.5		1.45				11	107	17.4	42.6	0	43	367	0	367	33	2003	1121	0.9	0.35	1.65
9/29/22	13:50	1728.9		1.44				10	117	17.3	42.7	0.12	43	323	-3	321	32	1925	1122	0.9	0.33	1.58
9/29/22	14:00	1730.28		1.42				10	127	17.0	43.0	0.24	43	319	-5	314	31	1881	1127	0.9	0.32	1.54

Minimum Rate: 1.5

Raw Rate for design, prior to application of adjustment factors: **1.5**

APPENDIX C

LABORATORY TEST RESULTS

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.

Expansion Index: An Expansion Index (EI) test was performed on a bulk sample of the site soils, in general accordance with the ASTM D 4829 Standard Test Method. Results of this test are presented on the "Expansion Index" sheet in this appendix.

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.

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.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of grain size analyses, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our boring logs.

R-value Test: One R-value test was performed on collected bulk soil sample to evaluate pavement support characteristics of the near-surface soils. R-value test was performed in accordance with Caltrans Standard Test Method 301. The test result is presented in this appendix.

Remolded Direct Shear: One Remolded Direct Shear test was performed on a collected bulk soil sample to determine the shear strength of soils at sloped areas. Direct Shear test was performed in accordance with ASTM D3080-04. The test result is presented 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.

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	M. Vinet	Date:	10/03/22
Project No.:	13673.004	Input By:	M. Vinet	Date:	10/04/22
Boring No.:	LB-8	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Reddish Brown.				

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

Mold Volume (ft³) 0.03340

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

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5567	5686	5694	5635		
Weight of Mold (g)	3529	3529	3529	3529		
Net Weight of Soil (g)	2038	2157	2165	2106		
Wet Weight of Soil + Cont. (g)	1074.6	1077.1	927.4	1012.2		
Dry Weight of Soil + Cont. (g)	1053.5	1044.2	893.0	962.0		
Weight of Container (g)	419.6	420.7	421.1	420.8		
Moisture Content (%)	3.3	5.3	7.3	9.3		
Wet Density (pcf)	134.5	142.4	142.9	139.0		
Dry Density (pcf)	130.2	135.2	133.2	127.2		

Maximum Dry Density (pcf) 135.3 **Optimum Moisture Content (%)** 5.5

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

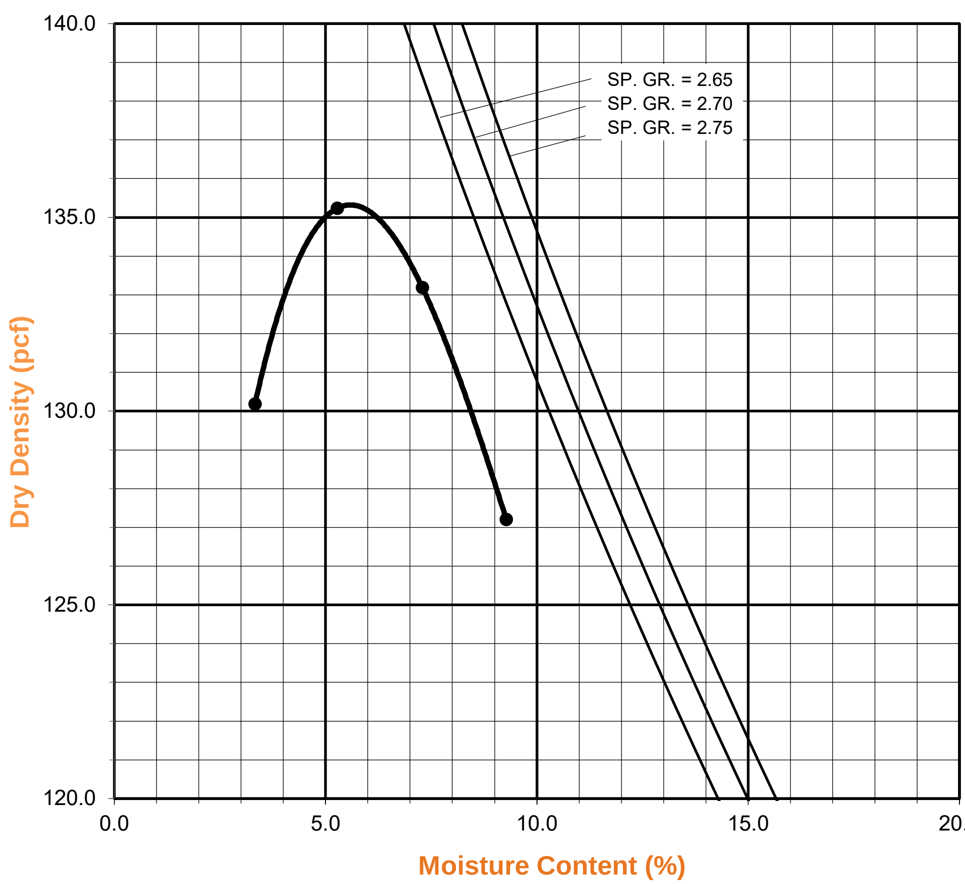
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: VVLIG Apple Valley Quarry Rd

Project No. : 13673.004

Tested By : M. Vinet Date: 10/05/22

Data Input By: M. Vinet Date: 10/05/22

Boring No.	LB-8			
Sample No.	B-1			
Sample Depth (ft)	0 - 5.0			
Soil Identification:	Silty Sand with Gravel (SM)g			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.0400			
Wt. of Crucible (g)	25.0363			
Wt. of Residue (g) (A)	0.0037			
PPM of Sulfate (A) x 41150	152.25			
PPM of Sulfate, Dry Weight Basis	**152**			

CHLORIDE CONTENT, DOT California Test 422

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

pH TEST, DOT California Test 643

pH Value	**7.60**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST
DOT CA TEST 643

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By :	M. Vinet Date: 10/05/22
Project No. :	13673.004	Data Input By:	M. Vinet Date: 10/05/22
Boring No.:	LB-8	Depth (ft.) :	0 - 5.0
Sample No. :	B-1		

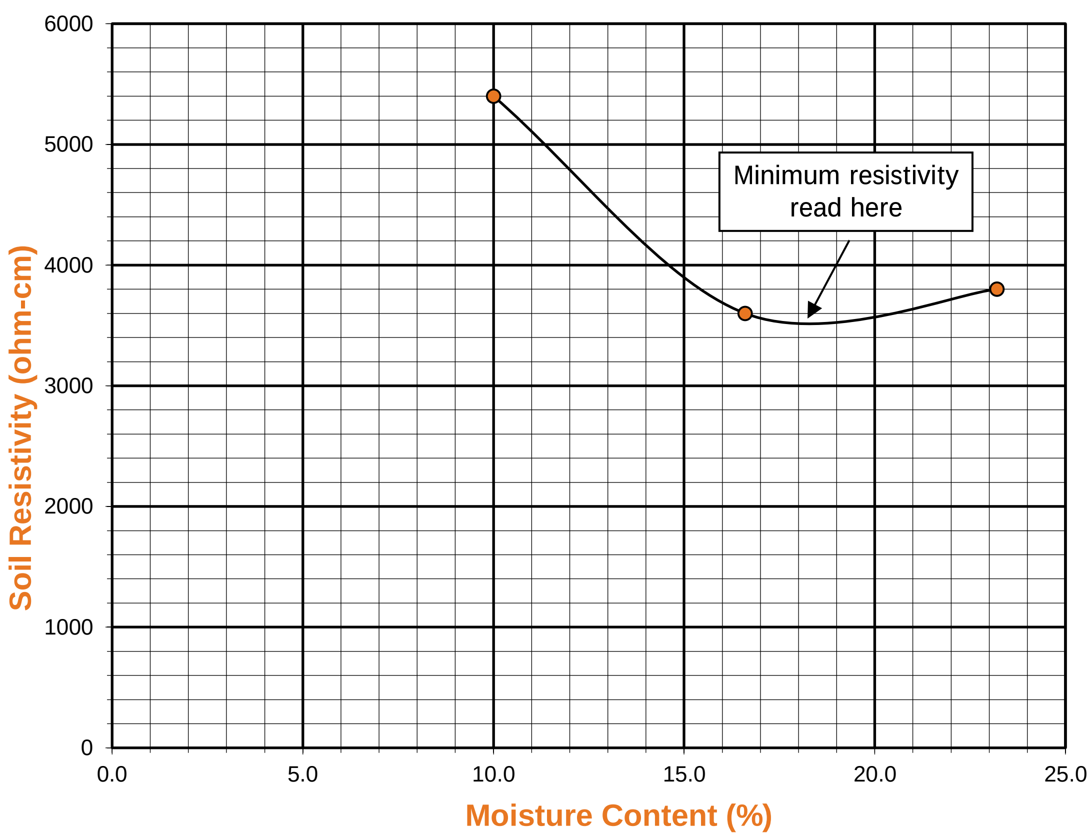
Soil Identification:* Silty Sand with Gravel (SM)g

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	5400	5400
2	83	16.60	3600	3600
3	116	23.20	3800	3800
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
3500	**18.0**	**152**	**40**	**7.60**	**21.0**



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By: M. Vinet	Date: 10/3/22
Project No. :	13673.004	Checked By: M. Vinet	Date: 10/4/22
Boring No.:	LB-2	Depth: 0 - 5.0	
Sample No. :	B-1	Location: N/A	
Sample Description:	Well-Graded Sand with Silt (SW-SM), Brown.		

Dry Wt. of Soil + Cont. (gm.)	3793.5
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	3793.5
Weight Soil Retained on #4 Sieve	465.1
Percent Passing # 4	87.7

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	0.9987
Wt. Comp. Soil + Mold (gm.)	610.0	611.1
Wt. of Mold (gm.)	199.0	199.0
Specific Gravity (Assumed)	2.70	2.70
Container No.	9	9
Wet Wt. of Soil + Cont. (gm.)	350.4	611.1
Dry Wt. of Soil + Cont. (gm.)	326.9	378.8
Wt. of Container (gm.)	50.4	199.0
Moisture Content (%)	8.5	8.8
Wet Density (pcf)	124.0	124.5
Dry Density (pcf)	114.3	114.4
Void Ratio	0.475	0.473
Total Porosity	0.322	0.321
Pore Volume (cc)	66.7	66.4
Degree of Saturation (%) [S meas]	**48.3**	**50.1**

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.)
10/3/22	12:30	1.0	0	0.5000
10/3/22	12:40	1.0	10	0.5000
	Add Distilled Water to the Specimen			
10/4/22	8:00	1.0	1160	0.4987
10/4/22	9:00	1.0	1220	0.4987

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	-1.3
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Heigh	**0**

Rev. 03-08

Boring No.	LB-1	LB-2	LB-2	LB-3A	LB-4	LB-4	LB-5	LB-5
Sample No.	R-2	R-1	R-2	R-1	R-1	R-2	R-2	R-3
Depth (ft.)	5.0	2.5	5.0	2.5	2.5	5.0	5.0	7.5
Sample Type	RING	RING	RING	RING	RING	RING	RING	RING
Visual Soil Classification	**TOP:** **SM** **BOTTOM:** **SM**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **SM** **BOTTOM:** **SM**	**TOP:** **SC** **BOTTOM:** **SC**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **SM** **BOTTOM:** **SM**	**TOP:** **SM** **BOTTOM:** **SM**
Pocket Penetrometer								
Weight Soil + Rings / Tube (gm.)	1005.2	874.5	1037.3	1035.4	1093.8	1024.3	1072.2	1105.3
Weight of Rings / Tube (gm.)	267.0	222.5	267.0	267.0	267.0	267.0	267.0	267.0
Average Length (in.)	6.0	5.0	6.0	6.0	6.0	6.0	6.0	6.0
Average Diameter (in.)	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Wet. Wt. of Soil + Cont. (gm.)	372.0	330.6	648.4	287.9	359.2	391.2	313.8	448.0
Dry Wt. of Soil + Cont. (gm.)	361.9	328.8	629.2	281.8	347.8	383.5	306.0	438.7
Weight of Container (gm)	36.7	49.6	277.5	38.5	49.5	32.6	35.7	36.8
Container No.:	A-21	29	K2	A-10	M	A-13	A-12	A-1
Wet Density (pcf)	102	108	107	106	115	105	112	116
Moisture Content (%)	**3**	**1**	**5**	**3**	**4**	**2**	**3**	**2**
Dry Density (pcf)	**99**	**108**	**101**	**104**	**110**	**103**	**108**	**113**
Degree of Saturation (%)	12	3	22	11	20	9	14	13

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MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: VVLIG Apple Valley Quarry Rd
Project No.: 13673.004
Client Name: VVLIG Holdings, LLC
Tested By: M. Vinet Date: 10/03/22

Boring No.	LB-5	LB-6	LB-7	LB-7	LB-8	LB-8	LB-8	LB-10
Sample No.	R-6	R-2	R-2	R-3	R-1	R-4	R-6	R-1
Depth (ft.)	20.0	5.0	5.0	7.5	2.5	10.0	20.0	2.5
Sample Type	RING	RING	RING	RING	RING	RING	RING	RING
Visual Soil Classification	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **(SW-SM)g** **BOTTOM:** **(SW-SM)g**	**TOP:** **SW-SM** **BOTTOM:** **SW-SM**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **SM** **BOTTOM:** **SM**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **(SM)g** **BOTTOM:** **(SM)g**	**TOP:** **(ML)s** **BOTTOM:** **(ML)s**
Pocket Penetrometer								
Weight Soil + Rings / Tube (gm.)	1077.3	1116.4	1011.2	1047.2	945.4	1074.8	1082.6	987.3
Weight of Rings / Tube (gm.)	267.0	267.0	267.0	267.0	222.5	267.0	267.0	267.0
Average Length (in.)	6.0	6.0	6.0	6.0	5.0	6.0	6.0	6.0
Average Diameter (in.)	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Wet. Wt. of Soil + Cont. (gm.)	375.9	880.4	631.7	369.0	334.2	394.9	334.7	292.5
Dry Wt. of Soil + Cont. (gm.)	361.7	864.3	620.8	362.0	331.4	388.2	329.0	284.9
Weight of Container (gm)	50.4	279.3	279.6	39.4	50.3	38.4	50.4	50.4
Container No.:	VW	PO	A	76	BB	A-19	HH	JJ
Wet Density (pcf)	112	118	103	108	120	112	113	100
Moisture Content (%)	**5**	**3**	**3**	**2**	**1**	**2**	**2**	**3**
Dry Density (pcf)	**107**	**114**	**100**	**106**	**119**	**110**	**111**	**97**
Degree of Saturation (%)	22	16	13	10	6	10	11	12

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MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name:	VVLIG Apple Valley Quarry Rd
Project No.:	13673.004
Client Name:	VVLIG Holdings, LLC
Tested By:	M. Vinet
Date:	10/03/22

Boring No.	LB-10								
Sample No.	R-2								
Depth (ft.)	5.0								
Sample Type	RING								
Visual Soil Classification	**TOP: (SW-SM)g BOTTOM: (SW-SM)g**								
Pocket Penetrometer									
Weight Soil + Rings / Tube (gm.)	964.5								
Weight of Rings / Tube (gm.)	267.0								
Average Length (in.)	6.0								
Average Diameter (in.)	2.416								
Wet. Wt. of Soil + Cont. (gm.)	610.4								
Dry Wt. of Soil + Cont. (gm.)	600.8								
Weight of Container (gm)	279.5								
Container No.:	B-1								
Wet Density (pcf)	97								
Moisture Content (%)	**3**								
Dry Density (pcf)	**94**								
Degree of Saturation (%)	10								

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MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: VVLIG Apple Valley Quarry Rd
Project No.: 13673.004
Client Name: VVLIG Holdings, LLC
Tested By: M. Vinet Date: 10/03/22

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DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	M. Vinet	Date:	10/04/22
Project No.:	13673.004	Checked By:	M. Vinet	Date:	10/05/22
Boring No.:	LB-8	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0 - 5.0		
Soil Identification:	Silty Sand (SM), Reddish Brown.				

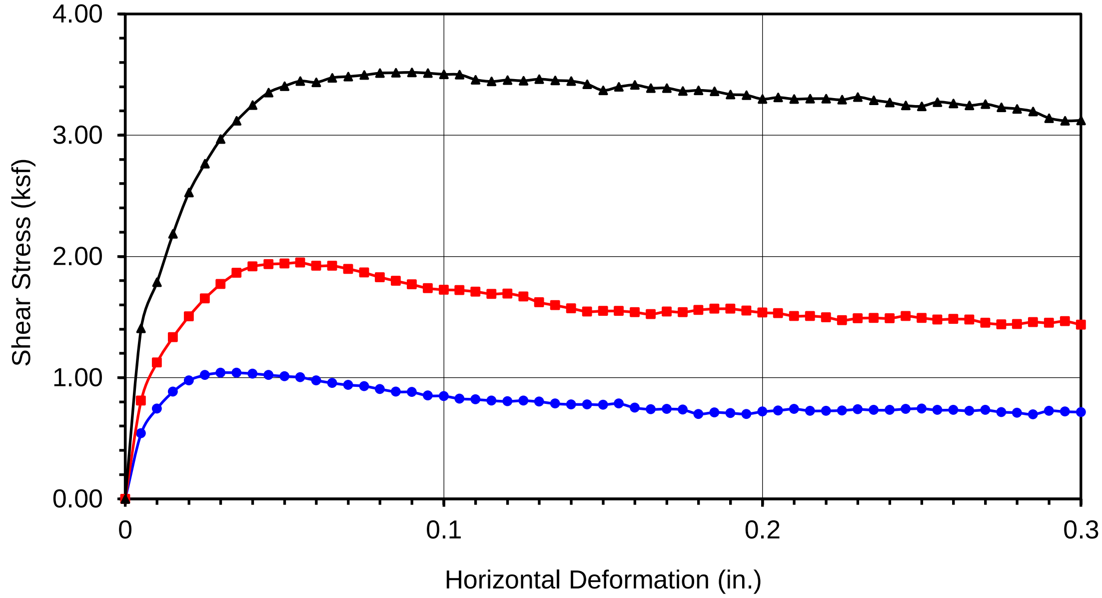
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	198.15	197.76	198.09
Weight of Ring(gm):	43.72	43.72	43.72

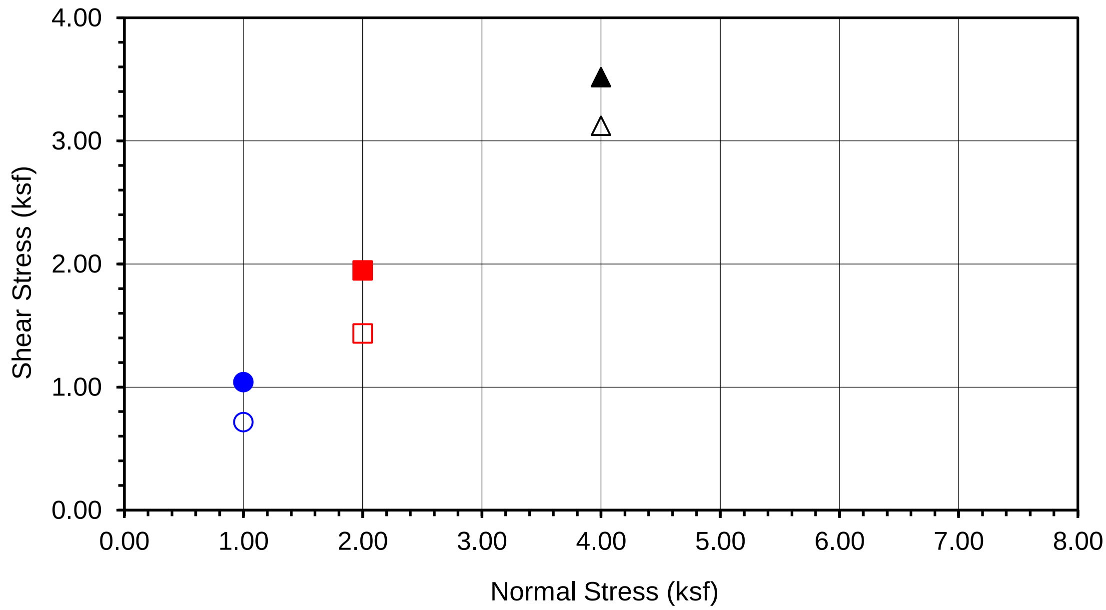
Before Shearing

Weight of Wet Sample+Cont.(gm):	253.94	253.94	253.94
Weight of Dry Sample+Cont.(gm):	244.11	244.11	244.11
Weight of Container(gm):	50.51	50.51	50.51
Vertical Rdg.(in): Initial	0.0000	0.2558	0.2476
Vertical Rdg.(in): Final	-0.0027	0.2621	0.2568

After Shearing

Weight of Wet Sample+Cont.(gm):	212.83	212.43	213.54
Weight of Dry Sample+Cont.(gm):	193.18	193.96	194.40
Weight of Container(gm):	50.41	50.43	50.40
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-8**
Sample No.	**B-1**
Depth (ft)	**0 - 5.0**

Sample Type:

90% Remold

Soil Identification:
Silty Sand (SM), Reddish Brown.

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.040	■ 1.948	▲ 3.518
Shear Stress @ End of Test (ksf)	○ 0.716	□ 1.436	△ 3.122
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	5.08	5.08	5.08
Dry Density (pcf)	122.2	121.9	122.2
Saturation (%)	36.2	35.8	36.1
Soil Height Before Shearing (in.)	0.9973	0.9937	0.9908
Final Moisture Content (%)	13.8	12.9	13.3

Leighton

DIRECT SHEAR TEST RESULTS

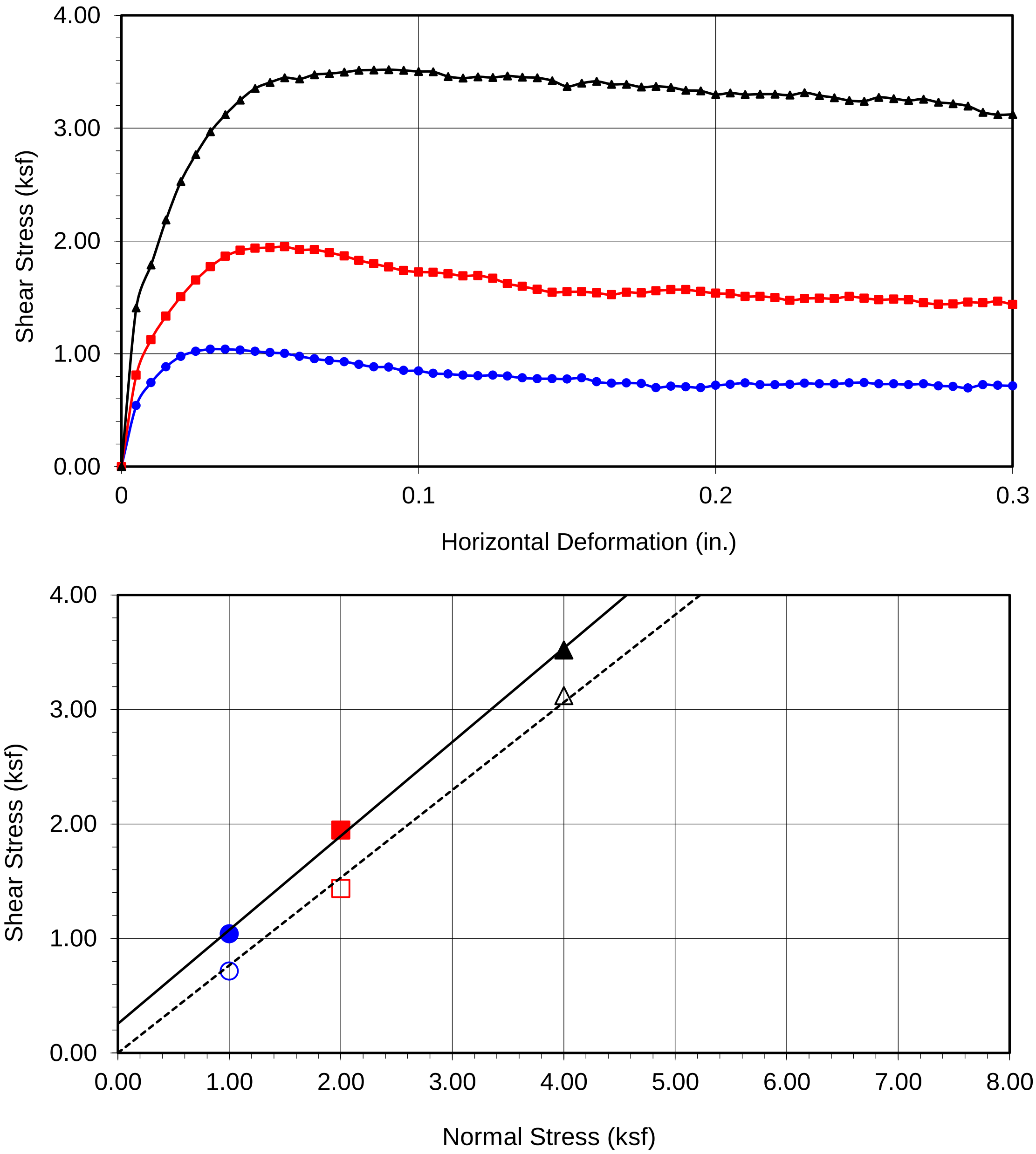
Consolidated Drained - ASTM D 3080

Project No.: 13673.004

VVLIG Apple Valley Quarry Rd

10-22

Remold Shear; LB-8, B-1 (09-20-22)



Boring No.	**LB-8**
Sample No.	**B-1**
Depth (ft)	**0 - 5.0**
Sample Type:	90% Remold
Soil Identification: Silty Sand (SM), Reddish Brown.	

Strength Parameters

	C (psf)	ϕ (°)
Peak	255	39
Ultimate	0	37

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.040	■ 1.948	▲ 3.518
Shear Stress @ End of Test (ksf)	○ 0.716	□ 1.436	△ 3.122
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	5.08	5.08	5.08
Dry Density (pcf)	122.2	121.9	122.2
Saturation (%)	36.2	35.8	36.1
Soil Height Before Shearing (in.)	0.9973	0.9937	0.9908
Final Moisture Content (%)	13.8	12.9	13.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13673.004

VVLIG Apple Valley Quarry Rd

10-22

Remold Shear; LB-8, B-1 (09-20-22)

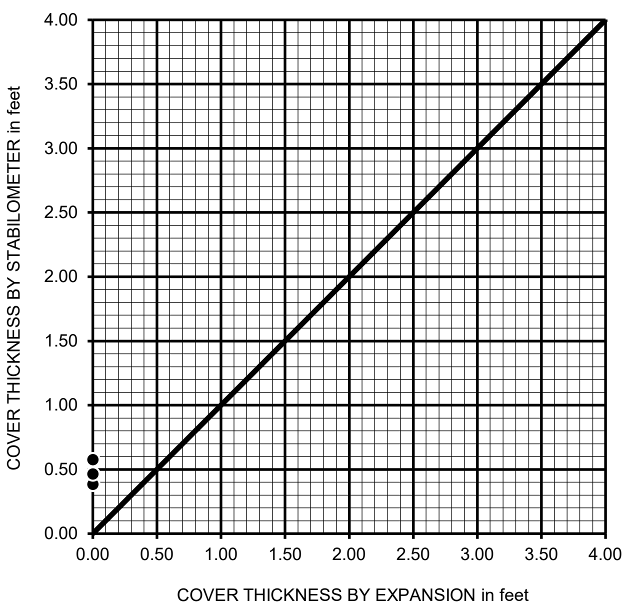
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	VVLIG Apple Valley Quarry Rd	PROJECT NUMBER:	13673.004
BORING NUMBER:	LB-2	DEPTH (FT.):	0 - 5.0
SAMPLE NUMBER:	B-1	TECHNICIAN:	F. Mina
SAMPLE DESCRIPTION:	Well-Graded Sand with Silt (SW-SM), Brown.	DATE COMPLETED:	10/4/2022

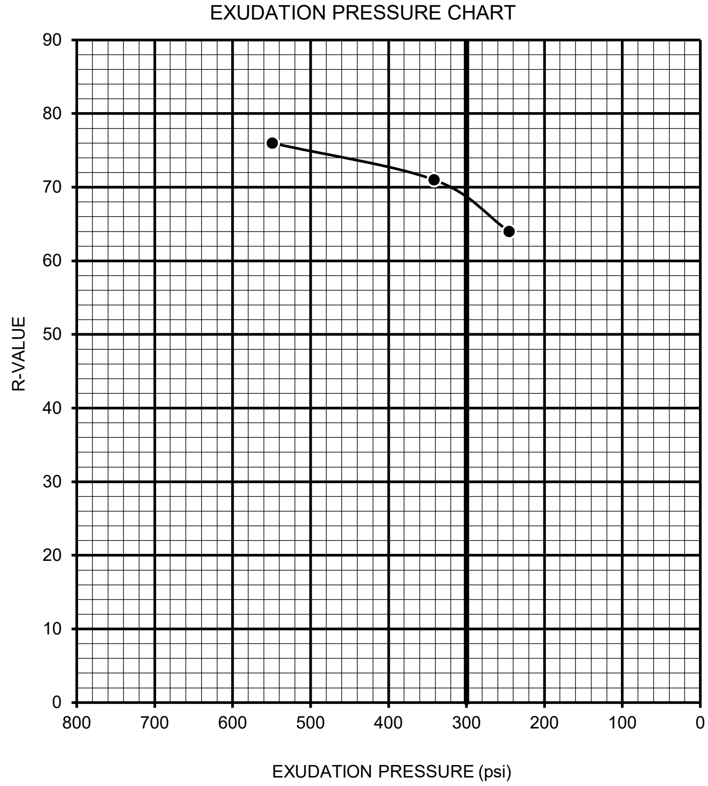
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	7.2	7.7	8.2
HEIGHT OF SAMPLE, Inches	2.57	2.52	2.49
DRY DENSITY, pcf	119.4	120.1	119.9
COMPACTOR PRESSURE, psi	350	350	350
EXUDATION PRESSURE, psi	549	341	245
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	23	27	33
TURNS DISPLACEMENT	4.94	5.10	5.30
R-VALUE UNCORRECTED	75	71	64
R-VALUE CORRECTED	76	71	64

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.38	0.46	0.58
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	69
EQUILIBRIUM R-VALUE:	69

Leighton

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

ASTM D 6913

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	MRV	Date:	10/03/22
Project No.:	13673.004	Checked By:	MRV	Date:	10/04/22
Boring No.:	LB-3A	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand with Gravel (SM)g, Yellowish Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	FL	FL	Wt. of Air-Dry Soil + Cont.(g)	1928.5	592.1
Wt. Air-Dried Soil + Cont.(g)	1928.5	592.1	Wt. of Dry Soil + Cont. (g)	1900.8	592.1
Wt. of Container (g)	277.9	277.9	Wt. of Container No.______(g)	277.9	277.9
Dry Wt. of Soil (g)	1623.0	314.2	Moisture Content (%)	1.7	0.0

Passing #4 Material After Wet Sieve		
	Container No.	FL
	Wt. of Dry Soil + Container (g)	526.0
	Wt. of Container (g)	277.9
	Dry Wt. of Soil Retained on # 200 Sieve (g)	248.1

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500			100.0
1"	25.000	0.0		100.0
3/4"	19.000	11.4		99.3
1/2"	12.500	72.9		95.5
3/8"	9.500	132.4		91.8
#4	4.750	292.9		82.0
#8	2.360		56.2	67.3
#16	1.180		110.8	53.1
#30	0.600		152.6	42.2
#50	0.300		188.2	32.9
#100	0.150		216.5	25.5
#200	0.075		241.3	19.0
PAN				

GRAVEL: **18 %**

SAND: **63 %**

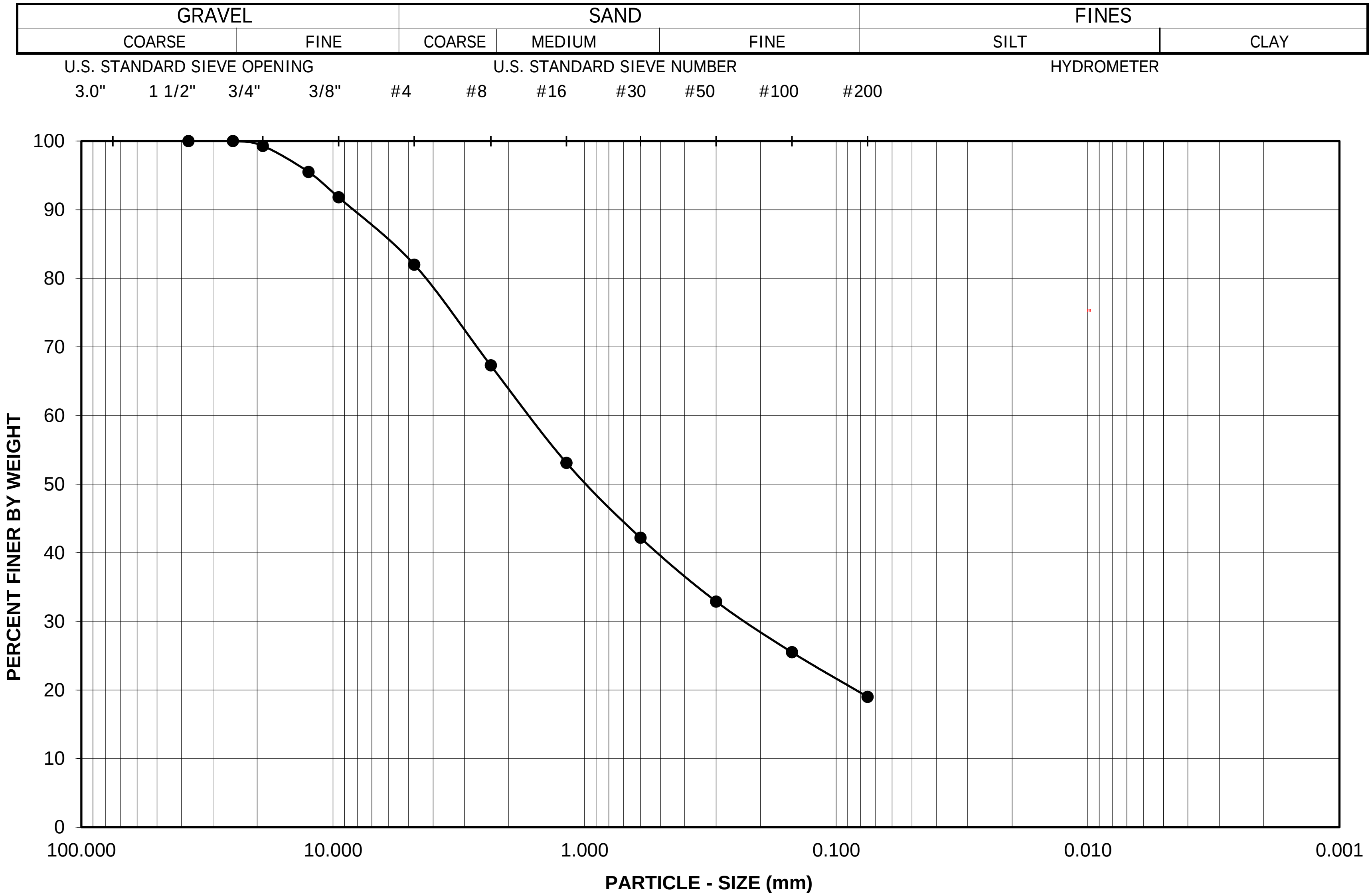
FINES: **19 %**

GROUP SYMBOL: **(SM)g**

Cu = D60/D10 = N/A

Cc = (D30)²/(D60*D10) = N/A

Remarks:



Project Name: VVLIG Apple Valley Quarry Rd

Project No.: 13673.004

Boring No.: LB-3A

Sample No.: B-1

Depth (feet): 0 - 5.0

Soil Type : (SM)g

Soil Identification: Silty Sand with Gravel (SM)g, Yellowish Brown.

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 18 : 63 : 19

Oct-22

Leighton

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

ASTM D 6913

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	MRV	Date:	10/03/22
Project No.:	13673.004	Checked By:	MRV	Date:	10/04/22
Boring No.:	LB-8	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand with Gravel (SM)g, Yellowish Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	W	W	Wt. of Air-Dry Soil + Cont.(g)	1970.0	609.4
Wt. Air-Dried Soil + Cont.(g)	1970.0	609.4	Wt. of Dry Soil + Cont. (g)	1946.7	609.4
Wt. of Container (g)	279.1	279.1	Wt. of Container No.______(g)	279.1	279.1
Dry Wt. of Soil (g)	1667.6	330.3	Moisture Content (%)	1.4	0.0

Passing #4 Material After Wet Sieve		
	Container No.	W
	Wt. of Dry Soil + Container (g)	557.0
	Wt. of Container (g)	279.1
	Dry Wt. of Soil Retained on # 200 Sieve (g)	277.9

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500			100.0
1"	25.000	0.0		100.0
3/4"	19.000	45.9		97.2
1/2"	12.500	83.0		95.0
3/8"	9.500	103.3		93.8
#4	4.750	241.3		85.5
#8	2.360		84.7	63.6
#16	1.180		166.5	42.4
#30	0.600		213.1	30.3
#50	0.300		240.5	23.2
#100	0.150		257.3	18.9
#200	0.075		274.3	14.5
PAN				

GRAVEL: **15 %**

SAND: **70 %**

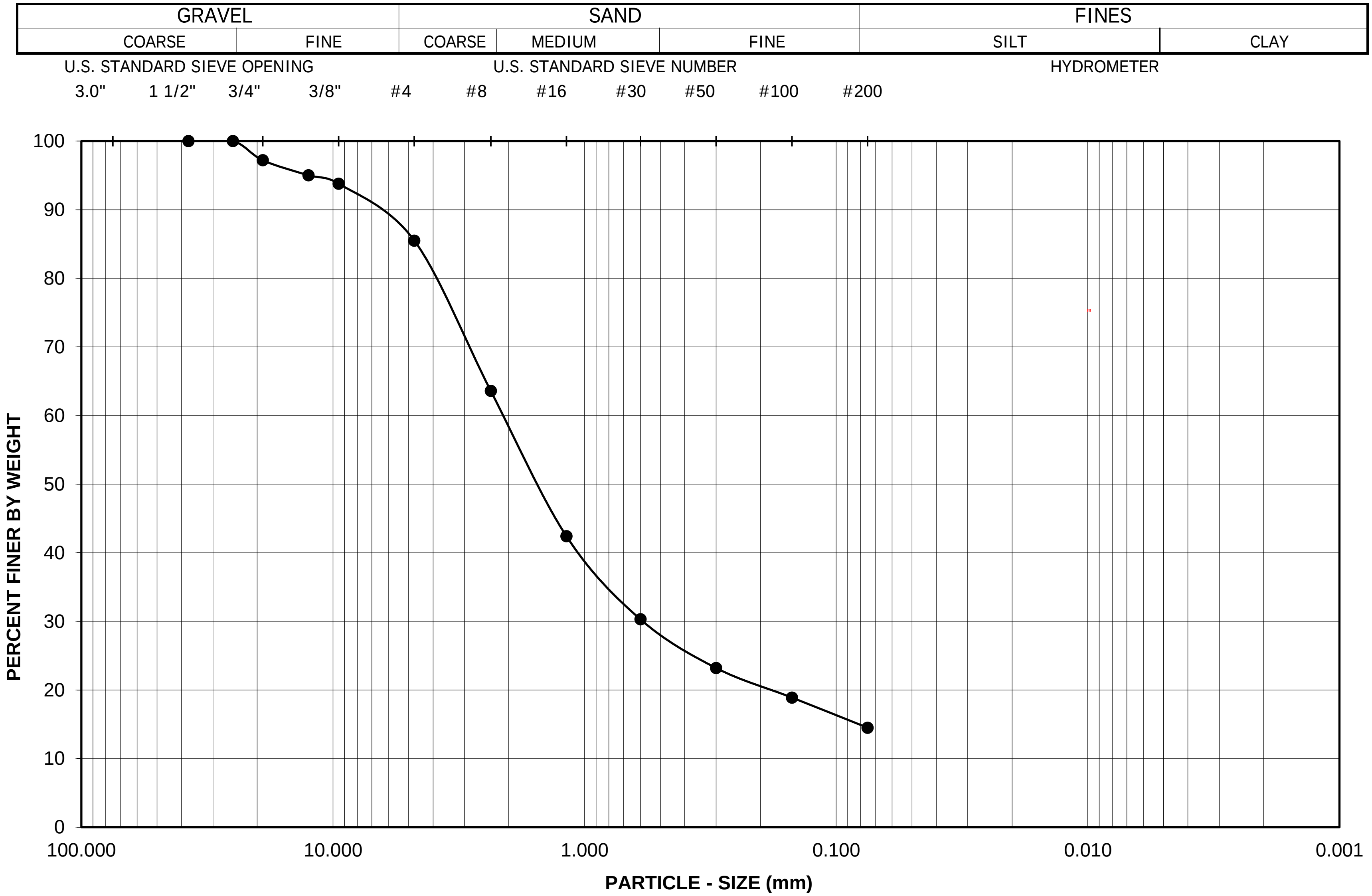
FINES: **15 %**

GROUP SYMBOL: **(SM)g**

Cu = D60/D10 = N/A

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

Remarks:



Project Name: VVLIG Apple Valley Quarry Rd

Project No.: 13673.004

Boring No.: LB-8 | Sample No.: B-1

Depth (feet): 0 - 5.0 | Soil Type : (SM)g

Soil Identification: Silty Sand with Gravel (SM)g, Yellowish Brown.

GR:SA:FI : (%) 15 : 70 : 15

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Oct-22

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

Leighton

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	MRV	Date:	10/03/22
Project No.:	13673.004	Checked By:	MRV	Date:	10/04/22
Boring No.:	LI-2	Depth (feet):	10.5		
Sample No.:	S-1				
Soil Identification:	Silty Sand with Gravel (SM), Yellowish Brown.				

		Moisture Content of Total Air - Dry Soil	
Container No.:	AB	Wt. of Air-Dry Soil + Cont. (g)	948.6
Wt. of Air-Dried Soil + Cont.(g)	948.6	Wt. of Dry Soil + Cont. (g)	913.8
Wt. of Container (g)	277.7	Wt. of Container No.______ (g)	277.7
Dry Wt. of Soil (g)	636.1	Moisture Content (%)	5.5

After Wet Sieve	Container No.	AB
	Wt. of Dry Soil + Container (g)	814.0
	Wt. of Container (g)	277.7
	Dry Wt. of Soil Retained on # 200 Sieve (g)	536.3

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
3"	75.000	0.0	100.0
1"	25.000	21.4	96.6
3/4"	19.000	32.9	94.8
1/2"	12.500	63.1	90.1
3/8"	9.500	79.9	87.4
#4	4.750	129.4	79.7
#8	2.360	238.8	62.5
#16	1.180	347.6	45.4
#30	0.600	419.7	34.0
#50	0.300	477.1	25.0
#100	0.150	511.0	19.7
#200	0.075	531.8	16.4
PAN			

GRAVEL: **20 %**

SAND: **64 %**

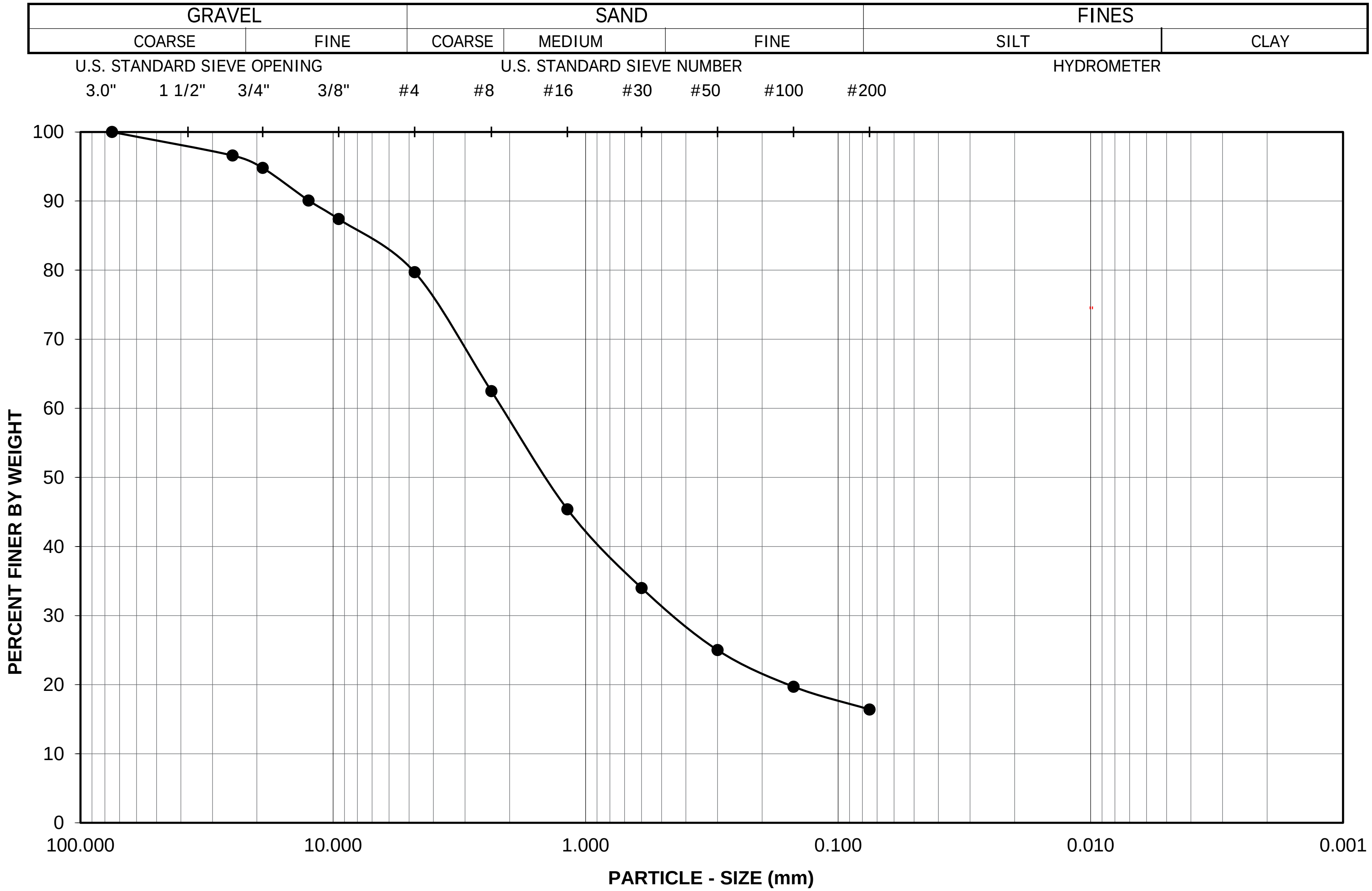
FINES: **16 %**

GROUP SYMBOL: **(SM)g**

Cu = D60/D10 = N/A

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

Remarks:



Project Name: VVLIG Apple Valley Quarry Rd

Project No.: 13673.004

Boring No.: LI-2 Sample No.: S-1

Depth (feet): 10.5 Soil Type : (SM)g

Soil Identification: Silty Sand with Gravel (SM), Yellowish Brown.

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 20 : 64 : 16

Oct-22

Sieve; LI-1, S-1 (09-20-22)

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

Leighton

Project Name:	VVLIG Apple Valley Quarry Rd	Tested By:	MRV	Date:	10/03/22
Project No.:	13673.004	Checked By:	MRV	Date:	10/04/22
Boring No.:	LI-2	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Yellowish Brown.				

		Moisture Content of Total Air - Dry Soil	
Container No.:	CC	Wt. of Air-Dry Soil + Cont. (g)	806.0
Wt. of Air-Dried Soil + Cont.(g)	806.0	Wt. of Dry Soil + Cont. (g)	800.0
Wt. of Container (g)	276.9	Wt. of Container No.______ (g)	276.9
Dry Wt. of Soil (g)	523.1	Moisture Content (%)	1.1

After Wet Sieve	Container No.	CC
	Wt. of Dry Soil + Container (g)	691.0
	Wt. of Container (g)	276.9
	Dry Wt. of Soil Retained on # 200 Sieve (g)	414.1

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
3"	75.000		100.0
1"	25.000		100.0
3/4"	19.000		100.0
1/2"	12.500		100.0
3/8"	9.500	0.0	100.0
#4	4.750	18.7	96.4
#8	2.360	84.5	83.8
#16	1.180	179.6	65.7
#30	0.600	254.0	51.4
#50	0.300	310.5	40.6
#100	0.150	350.4	33.0
#200	0.075	397.5	24.0
PAN			

GRAVEL: **4 %**

SAND: **72 %**

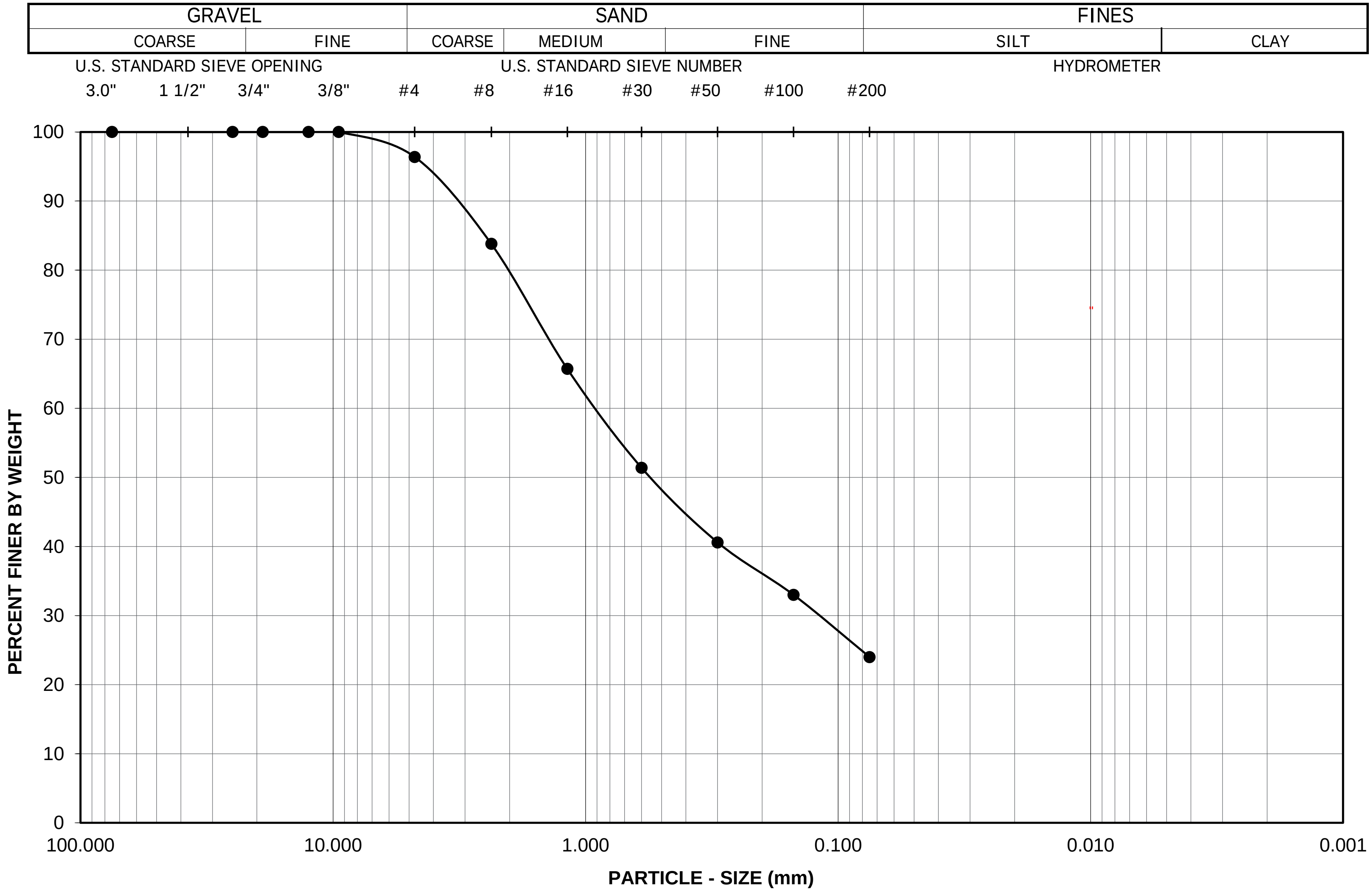
FINES: **24 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = N/A

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

Remarks:



Project Name: VVLIG Apple Valley Quarry Rd

Project No.: 13673.004

Boring No.: LI-2 Sample No.: B-1

Depth (feet): 0 - 5.0 Soil Type : SM

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

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 4 : 72 : 24

Oct-22

Sieve; LI-2, B-1 (09-20-22)

APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

APPENDIX D

SITE-SPECIFIC SEISMIC ANALYSIS (ASCE 7-16)

VVLIG – Quarry Road Apple Valley
(34.6122, -117.1827)

A site-specific ground motion study was performed in general conformance with Chapters 11, 20 and 21 of ASCE 7-16 and CGS Note 48.

The site is approximately 4.5 km from the surface trace of the closest element of the Helendale-South Lockhart fault zone. A Class C soil profile condition was considered for this site based on the results of our exploratory borings and geophysical survey. The site-specific response spectra in tabular and graphic forms are included herein (see Exhibits C-1 through C-6) and our specific analysis or approach is further discussed below:

Exhibit C-1: The probabilistic MCE spectrum was developed using spectral values obtained from USGS Unified Hazard Maps (UHGM) website, using the factors of ASCE 7-16 Section 21.1. At each spectral response period for which the acceleration is computed, ordinates of the probabilistic ground motion response spectrum is determined as the product of the risk coefficient, C_R, and the spectral response acceleration from a 5% damped acceleration response spectrum that has a 2% probability of exceedance within a 50-year period.

Exhibit C-2: A deterministic MCE spectrum was based on the maximum values of each period from the two most influential nearby faults. Scenario M7.39, and 8.2 events on the Helendale-South Lockhart and San Andreas (San Bernardino section) fault zones consistent with the Next Generation West 2 (NGA-West 2) attenuation relations (PEER NGAW2 GMPEs) used for the 2014 USGS seismic source model at fault distances of 4.5 and 45 km, respectively. The equally weighted spectral values from the attenuation relations of Abrahamson and others (ASK 2014), Boore and others (BSSA 2014), Campbell and Borzognia (CB 2014) and Chiou and Youngs (CY 2014) were used for the deterministic MCE spectrum. The MCE spectrum represents 84th-percentile, 5-percent-damped spectral response acceleration in the direction of maximum horizontal response (maximum rotated) for each period. Maximum rotated values were obtained using the scaling factors of ASCE 7-16 Section 21.2. Adjustment to the deterministic limit spectrum was applied as necessary. The Site Class C condition was modeled using Vs30 ≈ 560 meters/second, based on Multichannel Analysis of Surface Wave (MASW) methodology. The depth to bedrock ($Z_{1.0}$ km) was estimated to be around 197 feet (0.06 km), based on our geophysical survey results.

Exhibit C-3: The lesser of the values at any site period from the deterministic MCE_R and MCE_R probabilistic spectra forms the site-specific MCE_R spectrum. For this project site, the site-specific MCER spectrum is equivalent to the risk-modified probabilistic spectrum for all site periods.

Exhibits C-4 through C-6: A design response spectrum was determined according to the procedure outlined in ASCE 7-16, Section 21.3, and is equal to two-thirds of the response spectral accelerations of the site-specific MCE_R. The design spectrum is limited by a "floor" at 80 percent of spectral acceleration determined according to ASCE 7-16, Section 11.4.6. The recommended site-specific design response spectrum is attached in tabular and graphic forms.

PROBABILISTIC RESPONSE SPECTRA

Period (S)	UHGM (g)	C_R	Ordinated Value (g)	Max Dir SF	Max Dir RTGM (g)	Probabilistic Response (g)
0.01	0.509	0.935	0.476	1.1	0.524	0.524
0.10	1.051	0.935	0.982	1.1	1.081	1.081
0.20	1.243	0.935	1.162	1.1	1.278	1.278
0.30	1.122	0.934	1.048	1.124	1.178	1.178
0.50	0.854	0.931	0.795	1.175	0.934	0.934
0.75	0.631	0.928	0.585	1.2375	0.724	0.724
1.00	0.478	0.925	0.442	1.3	0.575	0.575
2.00	0.232	0.925	0.215	1.35	0.290	0.290
3.00	0.153	0.925	0.141	1.4	0.198	0.198
4.00	0.115	0.925	0.106	1.45	0.154	0.154
5.00	0.092	0.925	0.085	1.5	0.127	0.127

Peak Sa		Fa	1.2Fa	Peak Sa < 1.2Fa	Deterministic Needed?
1.278		1.0	1.2	NO	YES

UHGM - Obtained from Unified Hazard Maps
RTGM - Risk Target Ground Motion

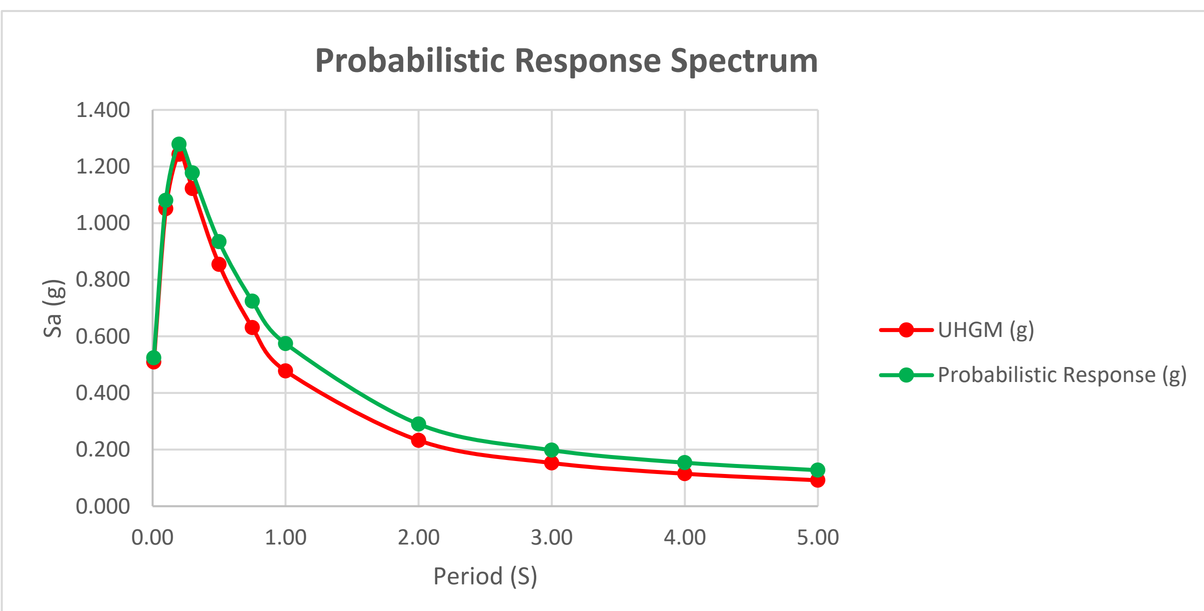


Exhibit C-1

DETERMINISTIC RESPONSE SPECTRUM

Period (S)	84th Percentile for 5% Damping	Max Dir SF	Max Dir Deterministic Sa	Scaled Max Dir Deterministic Sa
0.01	0.793	1.1	0.872	0.872
0.1	1.541	1.1	1.696	1.696
0.2	1.899	1.1	2.089	2.089
0.3	1.719	1.124	1.932	1.932
0.5	1.290	1.175	1.516	1.516
0.75	0.919	1.2375	1.137	1.137
1	0.685	1.3	0.891	0.891
2	0.299	1.35	0.404	0.404
3	0.179	1.4	0.251	0.251
4	0.122	1.45	0.177	0.177
5	0.091	1.5	0.136	0.136

Obtained from NGA West 2 GMPE Worksheet - UCERF3 fault

CALCS

Peak Sa	Fa	1.5Fa	Peak Sa < 1.5Fa	Scaling Factor
2.089	1.0	1.5	NO	1.000

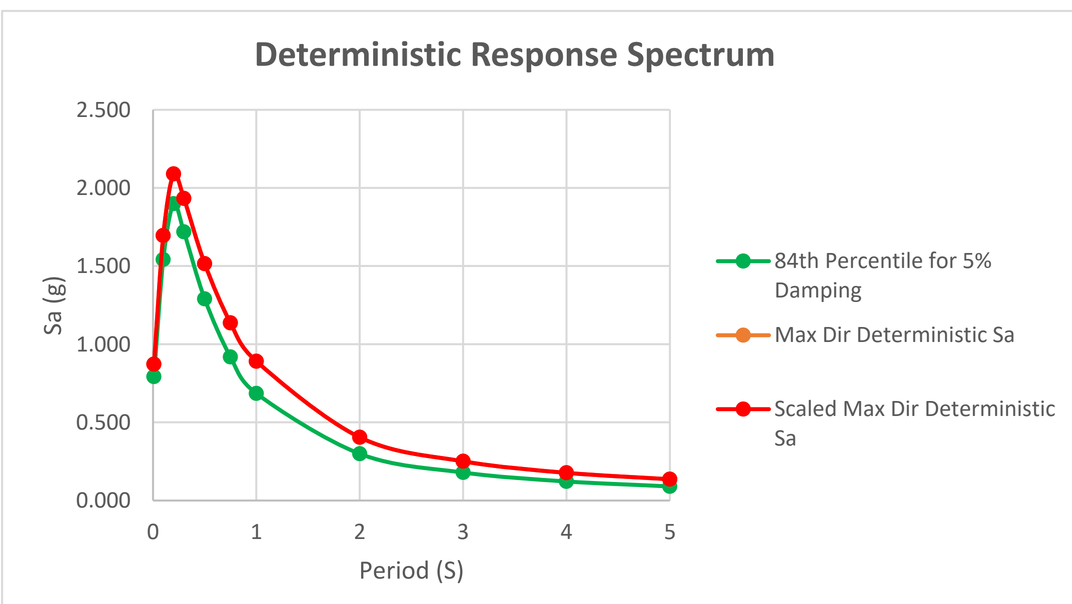


Exhibit C-2

SPECTRA COMPARISION

Period (s)	Probabilistic Response (g)	Scaled Max Dir Deterministic Sa (g)	MCE_{R*} Response Spectra S_{aM} (g)	2/3 MCER Response Spectra Sa (g)
0.01	0.524	0.872	0.524	0.349
0.1	1.081	1.696	1.081	0.720
0.2	1.278	2.089	1.278	0.852
0.3	1.178	1.932	1.178	0.785
0.5	0.934	1.516	0.934	0.623
0.75	0.724	1.137	0.724	0.483
1	0.575	0.891	0.575	0.383
2	0.290	0.404	0.290	0.193
3	0.198	0.251	0.198	0.132
4	0.154	0.177	0.154	0.102
5	0.127	0.136	0.127	0.085

MCER* is the lesser of the probabilistic and deterministic spectra

CALCS

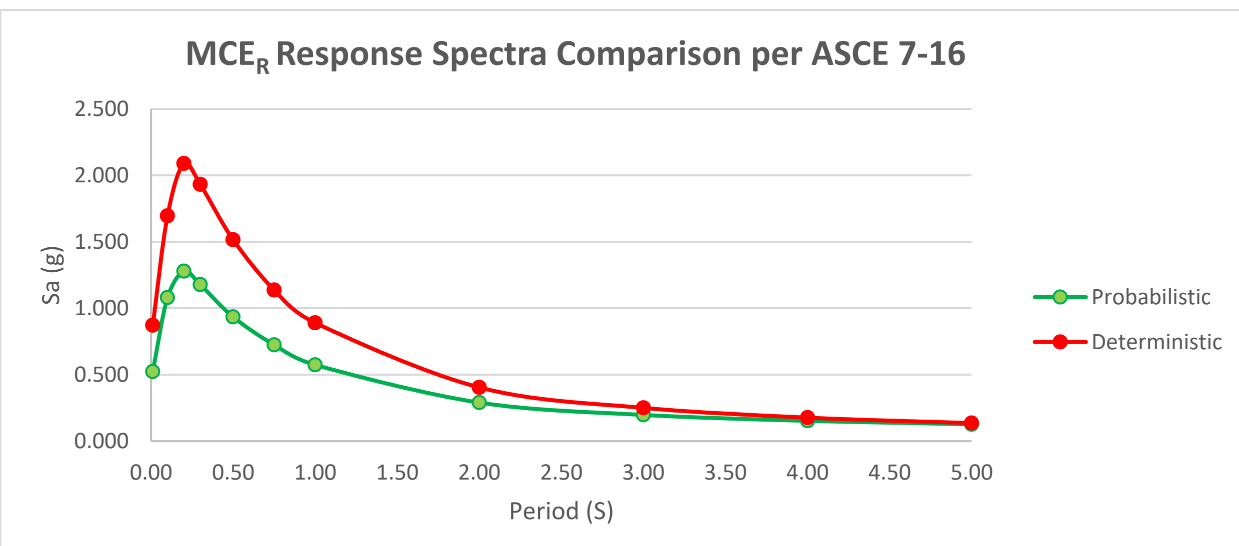


Exhibit C-3

S_S	1.025
S_1	0.393
Fa	1.2
Fv	1.5
S_{MS}	1.230
S_{M1}	0.590
S_{DS}	0.820
S_{D1}	0.393

since S_1 >0.2

T_0	0.100
T_S	0.500

PGA	0.440
PGA_M	0.528

Period (S)	Code-Based Sa (g)	80% Code-Based Sa (g)	2/3 MCER Response Spectra Sa (g)	Design Response Spectra Sa (g)
0.01	0.377	0.302	0.349	0.349
0.10	0.820	0.656	0.720	0.720
0.20	0.820	0.656	0.852	0.852
0.30	0.820	0.656	0.785	0.785
0.50	0.786	0.629	0.623	0.629
0.75	0.524	0.419	0.483	0.483
1.00	0.393	0.314	0.383	0.383
2.00	0.197	0.157	0.193	0.193
3.00	0.131	0.105	0.132	0.132
4.00	0.098	0.079	0.102	0.102
5.00	0.079	0.063	0.085	0.085

FROM SEISMIC MAPS (ATC OR OSHPD)

CALCS

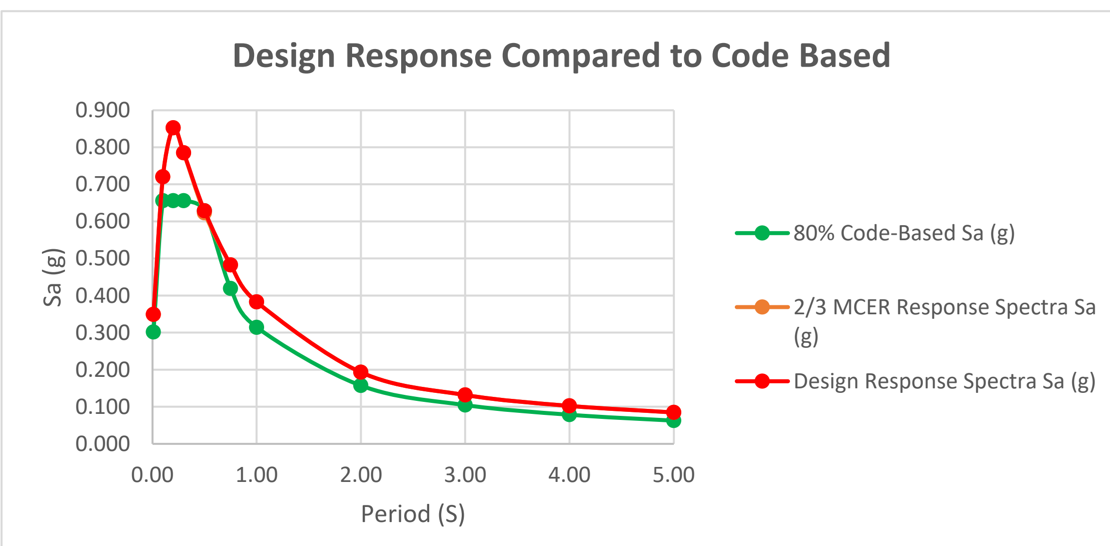


Exhibit C-4

Period (s)	MCER* Response Spectra SaM (g)	Design Response Spectra Sa (g)	Design Values (g)	
0.01	0.524	0.349	0.314	
0.10	1.081	0.720	0.648	
0.20	1.278	0.852	**0.767**	= S_{DS}
0.30	1.178	0.785	0.707	
0.50	0.934	0.629	0.566	
0.75	0.724	0.483	0.435	
1.00	0.575	0.383	0.383	
2.00	0.290	0.193	**0.387**	= S_{D1}
3.00	0.198	0.132	0.395	
4.00	0.154	0.102	0.410	
5.00	0.127	0.085	0.424	

Max Sa between T=0.2s and 5s is **0.852**

S_{DS} = 0.9 X Max Sa = **0.767**
S_{MS} = 1.5*S_{DS} = **1.151**

Short Period Spectrum

V_{S30} = 560 m/s > 365 m/s **Site Class C**

Max T*S_a between T=1s and 2s is **0.387**

Therefore, S_{D1} = **0.387**
S_{M1} = 1.5*S_{D1} = **0.580**

Long Period Spectrum

Probabilistic PGA	**0.509**
Deterministic PGA	**0.793**
80% Code-Based PGA_M	**0.422**
Site-Specific PGA	**0.793**

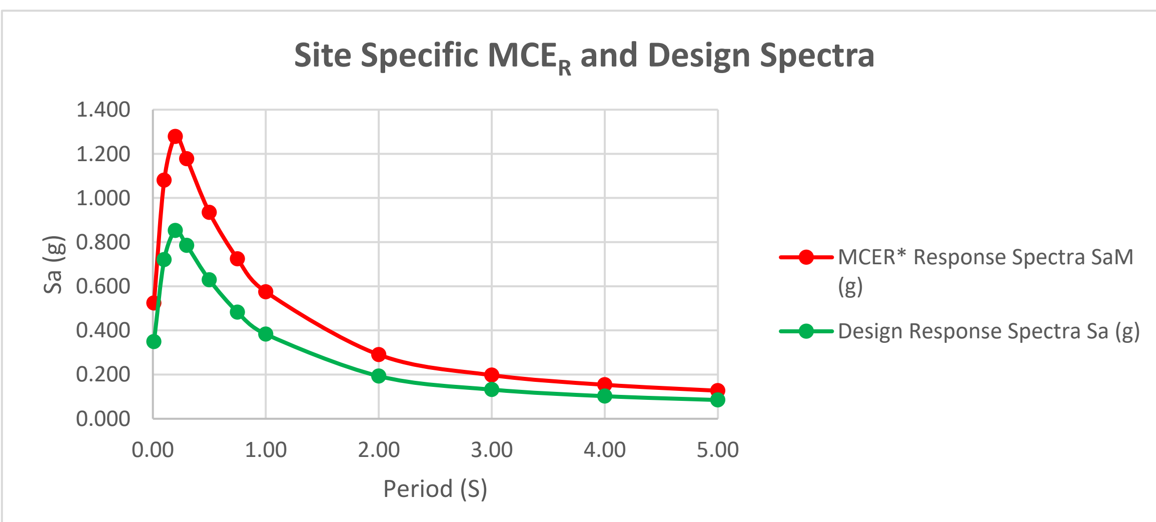


Exhibit C-5

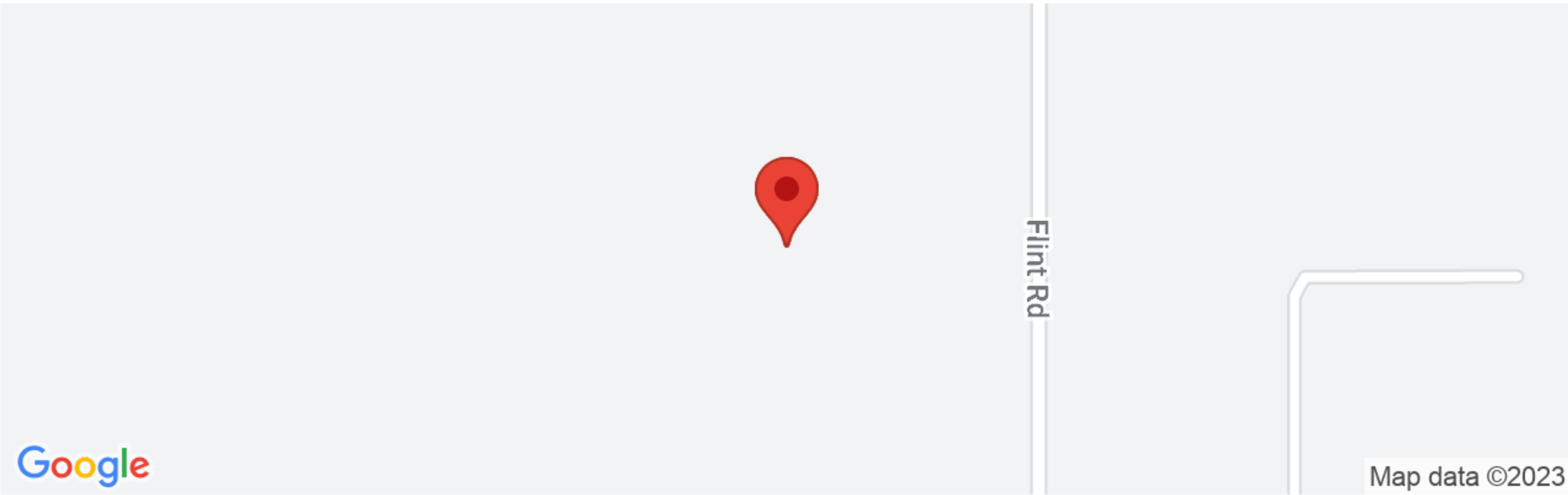
SUMMARY TABLE

Site-Specific Seismic Analysis (per ASCE 7-16)

	Site Seismic Coefficients / Coordinates	Value	
	Latitude	34.6122	
	Longitude	-117.1827	
Mapped Spectra (OSHPD)	Spectral Response – Class C (short), S_S	1.03	**Exhibit C-4**
	Spectral Response – Class C (1 sec), S_1	0.39	**Exhibit C-4**
	Site Modified Peak Ground Acceleration, PGA_M	0.53	**Exhibit C-4**
Site-Specific Response Spectra	Max. Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.15	**Exhibit C-5**
	Max. Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.58	**Exhibit C-5**
	5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.77	**Exhibit C-5**
	5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.39	**Exhibit C-5**
	Maximum Considered Earthqauke Geometric Mean MCE_G PGA	0.79	**Exhibit C-5**

Site 4

Latitude, Longitude: 34.6122, -117.1827



Date	1/6/2023, 12:52:05 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.025	MCE_R ground motion. (for 0.2 second period)
S_1	0.393	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.229	Site-modified spectral acceleration value
S_{M1}	0.59	Site-modified spectral acceleration value
S_{DS}	0.82	Numeric seismic design value at 0.2 second SA
S_{D1}	0.393	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.44	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.528	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.025	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.096	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.874	Factored deterministic acceleration value. (0.2 second)
S1RT	0.393	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.425	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.696	Factored deterministic acceleration value. (1.0 second)
PGAd	0.782	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.44	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.935	Mapped value of the risk coefficient at short periods
C_{R1}	0.925	Mapped value of the risk coefficient at a period of 1 s
C_V	1.105	Vertical coefficient

DISCLAIMER

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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 (update…

Latitude

Decimal degrees

34.6122

Longitude

Decimal degrees, negative values for western longitudes

-117.1827

Site Class

537 m/s (Site class C)

Spectral Period

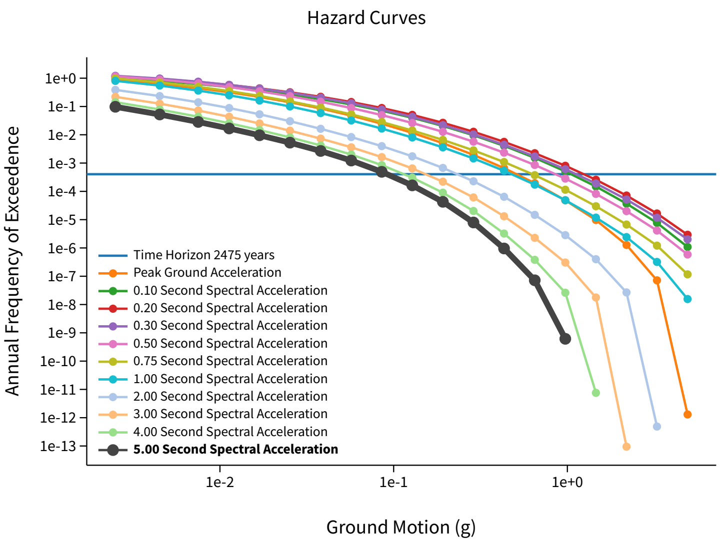
5.00 Second Spectral Acceleration

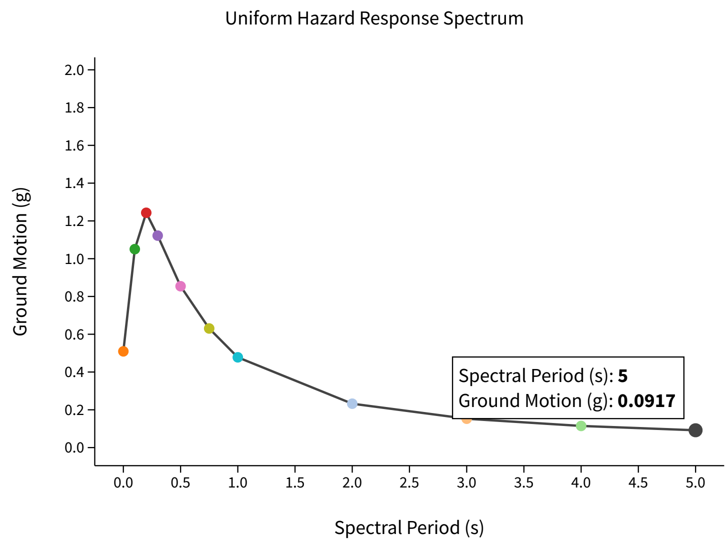
Time Horizon

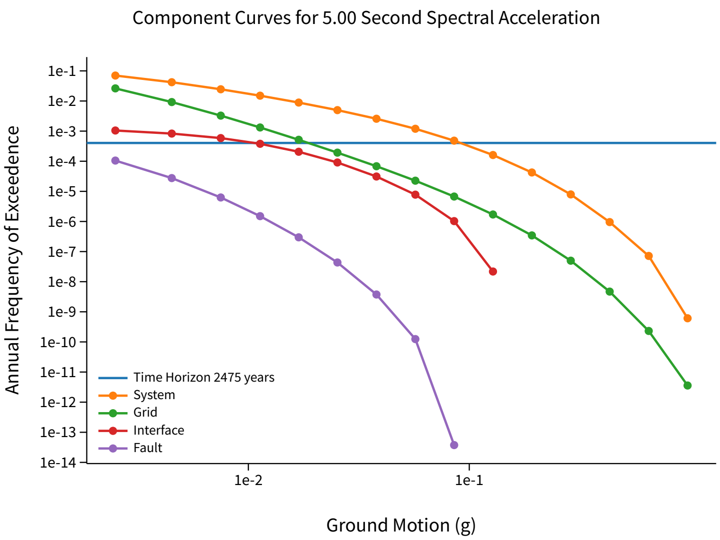
Return period in years

2475

Hazard Curve





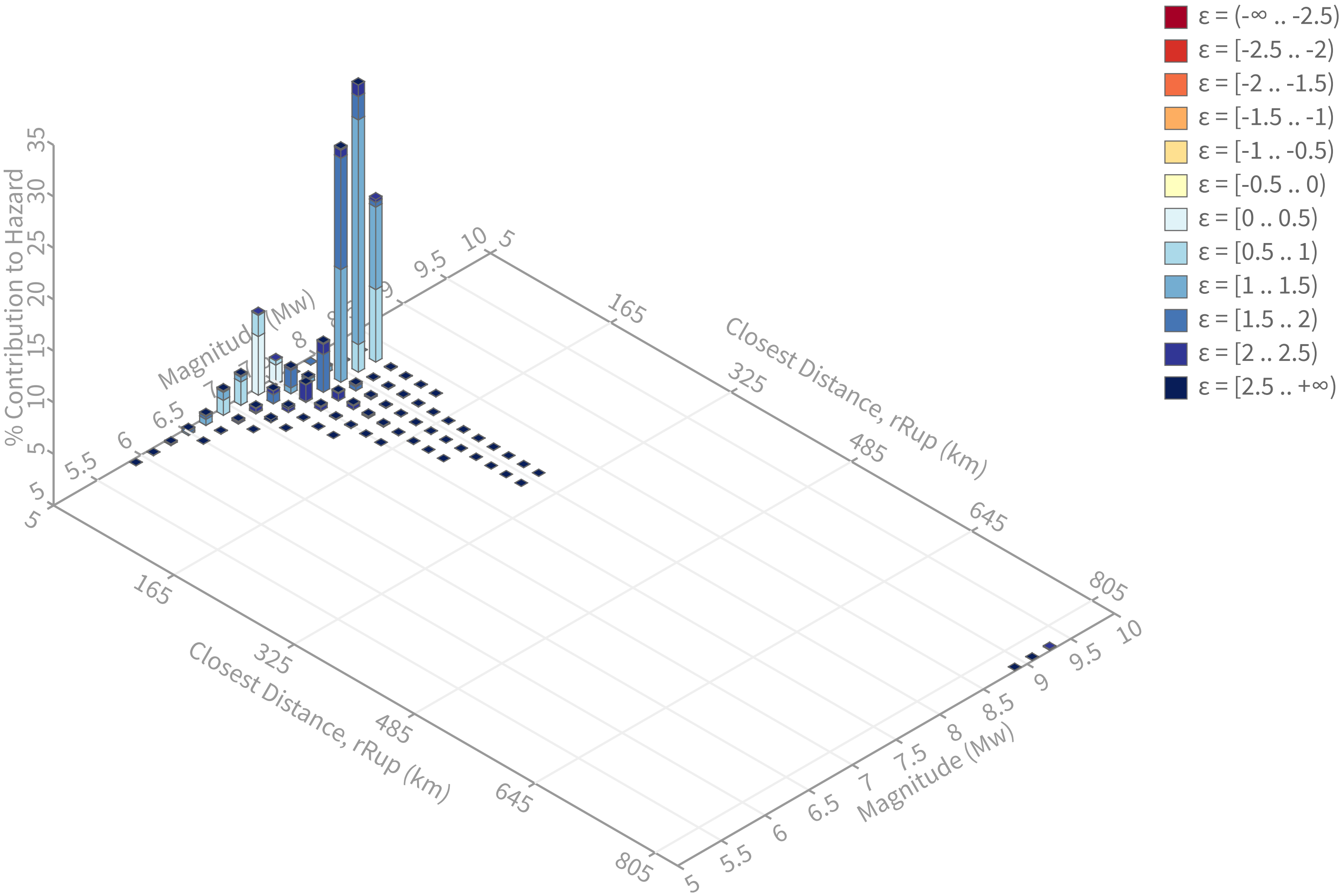


View Raw Data

Deaggregation

Component

Total



% Contribution to Hazard
35
30
25
20
15
10
5
Magnitude (Mw)
5
5.5
6
6.5
7
7.5
8
8.5
9
9.5
10
Closest Distance, rRup (km)
5
165
325
485
645
805
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
5.0 s SA ground motion: 0.091659512 g

Recovered targets

Return period: 2730.4819 yrs
Exceedance rate: 0.00036623572 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.17 %

Mean (over all sources)

m: 7.84
r: 40 km
ε_0: 1.32 σ

Mode (largest m-r bin)

m: 8.1
r: 45.3 km
ε_0: 1.27 σ
Contribution: 27.89 %

Mode (largest m-r-ε_0 bin)

m: 8.09
r: 45.3 km
ε_0: 1.23 σ
Contribution: 21.83 %

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							49.33
San Andreas (San Bernardino N) [1]		45.26	8.07	1.32	117.447°W	34.268°N	212.43	32.24
Helendale-So Lockhart [7]		4.50	7.22	0.55	117.151°W	34.641°N	42.65	7.13
UC33brAvg_FM32	System							49.28
San Andreas (San Bernardino N) [1]		45.26	8.06	1.32	117.447°W	34.268°N	212.43	32.26
Helendale-So Lockhart [7]		4.50	7.22	0.56	117.151°W	34.641°N	42.65	7.06
Cucamonga [0]		46.11	7.87	1.63	117.445°W	34.192°N	207.28	1.10

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

Input variables | **Errors and warnings**

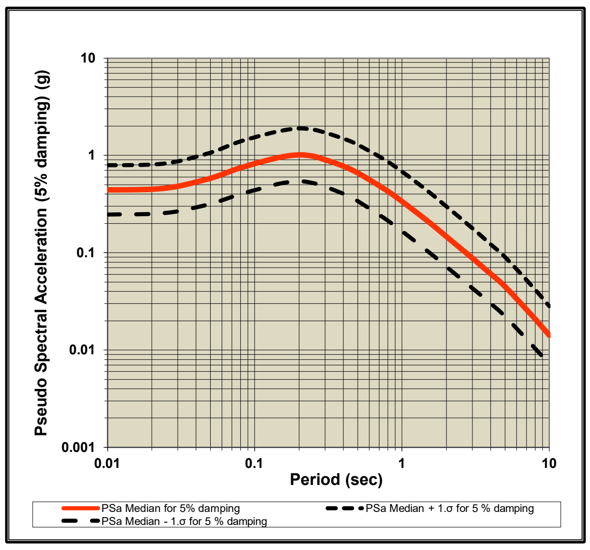
		Errors and warnings
M_w	7.39	
R_{RUP} (km)	9.3	
R_{JB} (km)	4.5	
R_X (km)	4.5	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	560	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	0	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	0	*1: hanging wall side*
Dip (deg)	90	
Z_{TOR} (km)	8.2	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.06	*If unknown use 999*
$Z_{2.5}$ (km)	0.26	*If unknown use 999*
W (km)	11.5	*If unknown use 999*
Vs30Flag	inferred	*Choose options for V_{s30} from the list*
F_{AS}	no	*Aftershock effect is not applicable.*
Region	California	*Choose region from the list*

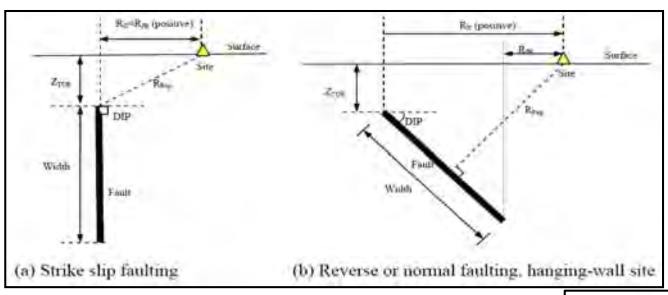
Calculated Variables/Flags

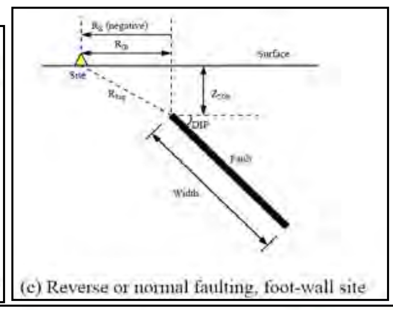
ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.389	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	1	*auto calculated*
$V_{s30Flag}$	0	*inferred*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

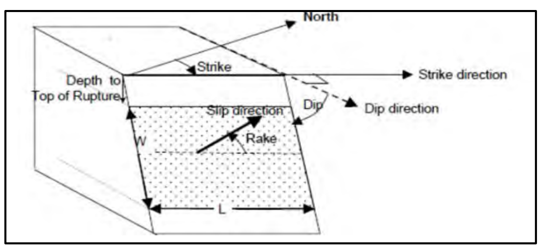
RotD50 Horizontal Component of PGA, PGV and *IMs*

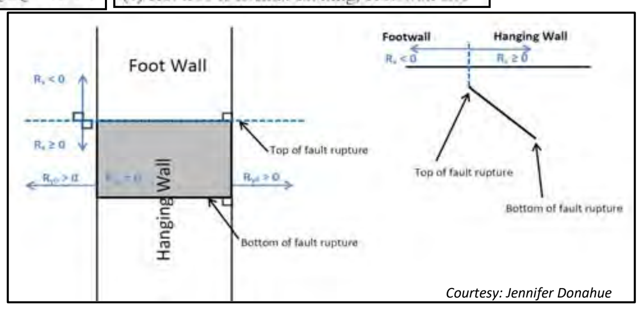
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.44259	0.79263	0.24713	0.00110	0.44259	0.79263	0.24713	0.00110
	0.02	0.45005	0.80698	0.25100	0.00447	0.45005	0.80698	0.25100	0.00447
	0.03	0.48374	0.87315	0.26800	0.01081	0.48325	0.87228	0.26773	0.01080
	0.05	0.58458	1.07168	0.31888	0.03628	0.58458	1.07168	0.31888	0.03628
	0.075	0.72210	1.34475	0.38776	0.10083	0.72355	1.34744	0.38853	0.10103
	0.1	0.82159	1.53681	0.43923	0.20395	0.82405	1.54142	0.44054	0.20456
	0.15	0.96186	1.79169	0.51637	0.53723	0.96379	1.79527	0.51740	0.53831
	0.2	1.01283	1.89526	0.54126	1.00569	1.01486	1.89905	0.54234	1.00770
	0.25	0.97640	1.83431	0.51974	1.51486	0.97933	1.83981	0.52129	1.51941
	0.3	0.90447	1.71746	0.47632	2.02070	0.90537	1.71918	0.47680	2.02272
	0.4	0.77477	1.48833	0.40332	3.07723	0.77555	1.48982	0.40372	3.08031
	0.5	0.66126	1.28861	0.33934	4.10375	0.66193	1.28990	0.33967	4.10786
	0.75	0.45833	0.91856	0.22869	6.39985	0.45833	0.91856	0.22869	6.39985
	1	0.33842	0.68578	0.16701	8.40086	0.33808	0.68509	0.16684	8.39246
	1.5	0.21067	0.42915	0.10342	11.76658	0.21088	0.42958	0.10352	11.77834
	2	0.14719	0.29995	0.07223	14.61553	0.14690	0.29935	0.07209	14.58630
	3	0.08804	0.17926	0.04324	19.66897	0.08795	0.17908	0.04319	19.64930
	4	0.06077	0.12250	0.03015	24.13850	0.06071	0.12238	0.03012	24.11437
	5	0.04503	0.09088	0.02231	27.94257	0.04489	0.09061	0.02224	27.85874
	7.5	0.02335	0.04694	0.01162	32.61110	0.02328	0.04680	0.01158	32.51326
	10	0.01431	0.02843	0.00720	35.52353	0.01425	0.02832	0.00717	35.38143
PGA (g)	0	0.44050	0.78827	0.24616	0.00109	0.44050	0.78827	0.24616	0.00109
PGV (cm/s)	-1	42.83167	78.14243	23.47703	0.10632	NA	NA	NA	NA











Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} ***(km)*** = The depth to the bottom of the seismogenic crust
Z_{BOR} ***(km)*** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	11.50			15.000		
$Z_{1.0}$ (km)	0.060	0.060			0.162	
$\delta Z_{1.0}$ (km)	-0.102		-0.102			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.260			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.260			0.861		
Z_{hyp} (km)	999.00			10.227		
Z_{tor} (km)	8.20			0.000	0.000	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

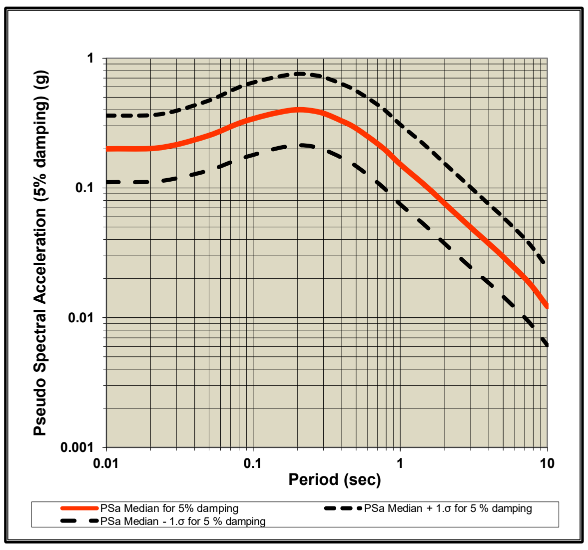
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

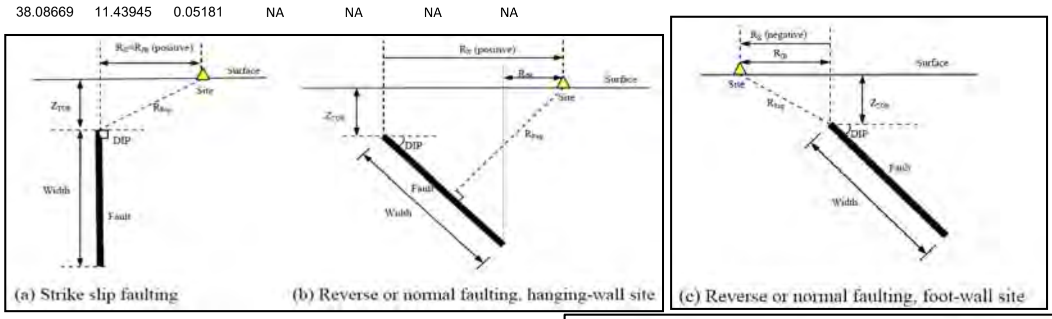
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

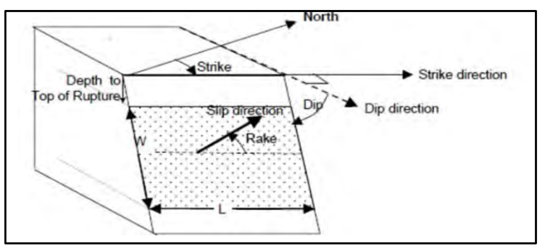
RotD50 Horizontal Component of PGA, PGV and *IMs*

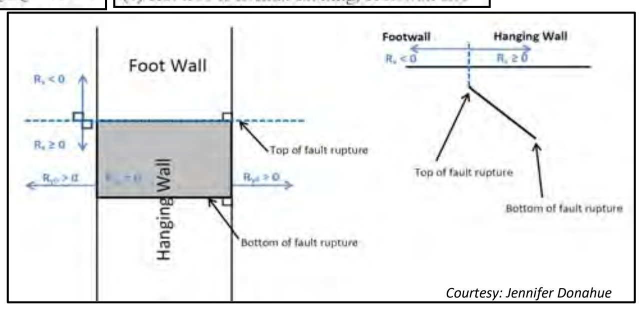
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.20031	0.36123	0.11108	0.00050	0.20031	0.36123	0.11108	0.00050
	0.02	0.20154	0.36403	0.11158	0.00200	0.20154	0.36403	0.11158	0.00200
	0.03	0.21606	0.39371	0.11857	0.00483	0.21584	0.39332	0.11845	0.00482
	0.05	0.25383	0.47166	0.13660	0.01575	0.25408	0.47214	0.13674	0.01577
	0.075	0.30568	0.57802	0.16166	0.04268	0.30660	0.57975	0.16215	0.04281
	0.1	0.34033	0.64661	0.17913	0.08448	0.34169	0.64920	0.17984	0.08482
	0.15	0.38234	0.72179	0.20253	0.21355	0.38349	0.72395	0.20314	0.21419
	0.2	0.39992	0.75407	0.21209	0.39710	0.40112	0.75633	0.21273	0.39829
	0.25	0.39415	0.74353	0.20894	0.61152	0.39415	0.74353	0.20894	0.61152
	0.3	0.37445	0.71248	0.19680	0.83657	0.37557	0.71462	0.19739	0.83908
	0.4	0.32832	0.63128	0.17075	1.30401	0.32897	0.63254	0.17109	1.30661
	0.5	0.28739	0.56045	0.14737	1.78355	0.28739	0.56045	0.14737	1.78355
	0.75	0.20587	0.41275	0.10268	2.87464	0.20608	0.41317	0.10279	2.87751
	1	0.15174	0.30755	0.07486	3.76662	0.15189	0.30786	0.07494	3.77039
	1.5	0.10288	0.20961	0.05050	5.74625	0.10298	0.20982	0.05055	5.75200
	2	0.07586	0.15462	0.03722	7.53256	0.07578	0.15446	0.03718	7.52502
	3	0.04967	0.10115	0.02439	11.09739	0.04972	0.10125	0.02442	11.10848
	4	0.03730	0.07519	0.01851	14.81614	0.03723	0.07504	0.01847	14.78650
	5	0.02947	0.05949	0.01460	18.29153	0.02942	0.05937	0.01457	18.25495
	7.5	0.01869	0.03756	0.00930	26.09496	0.01856	0.03730	0.00923	25.91230
	10	0.01225	0.02435	0.00617	30.41544	0.01218	0.02420	0.00613	30.23294
PGA (g)	0	0.19945	0.35937	0.11069	0.00050	0.19945	0.35937	0.11069	0.00050
PGV (cm/s)	-1	20.87321	38.08669	11.43945	0.05181	NA	NA	NA	NA



Input variables	Errors and warnings
M $_w$	
8.2	
R $_{RUP}$ **(km)**	
44.7	
R $_{JB}$ **(km)**	
44	
R $_X$ **(km)**	
44	
***Ry0* (km)**	*If unknown use 999*
999	
V $_{S30}$ ***(m/sec)***	
560	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F $_{RV}$	*1: reverse fault*
0	
F $_{NM}$	*1: normal fault*
0	
F $_{HW}$	*1: hanging wall side*
0	
Dip (deg)	
90	
Z $_{TOR}$ ***(km)***	*If unknown use 999*
7.68	
Z $_{HYP}$ ***(km)***	*If unknown use 999*
999	
Z $_{1.0}$ ***(km)***	*If unknown use 999*
0.06	
Z $_{2.5}$ ***(km)***	*If unknown use 999*
0.26	
W (km)	*If unknown use 999*
12.8	
Vs30Flag	
inferred	*Choose options for V* $_{s30}$ *from the list*
F $_{AS}$	
no	*Aftershock effect is not applicable.*
Region	
California	*Choose region from the list*







Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP	*Always 0 for median calcs.*
0	
PGA $_r$ ***(g)***	
0.146	
Z $_{BOT}$ ***(km) (CB14)***	*Enter for default W calcs*
15	
SS	
1	*auto calculated*
V $_{s30Flag}$	
0	*inferred*
F $_{AS}$	
0	*Aftershock effect is not applicable.*
Region	
0	*California*
Option for Sa value	
1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S $_d$ = Relative displacement response spectrum (cm)
M $_W$ = Moment magnitude
R $_{RUP}$ = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R $_{JB}$ = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R $_X$ = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R $_{y0}$ = The horizontal distance off the end of the rupture measured parallel to strike (km)
V $_{S30}$ = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F $_{RV}$ = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F $_{NM}$ = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F $_{HW}$ = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z $_{TOR}$ = Depth to top of coseismic rupture (km)
Z $_{HYP}$ = Hypocentral depth from the earthquake
Z $_{1.0}$ = Depth to Vs=1 km/sec
Z $_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
V $_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F $_{AS}$ = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA $_r$ ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z $_{BOT}$ ***(km)*** = The depth to the bottom of the seismogenic crust
Z $_{BOR}$ ***(km)*** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	12.80			15.000		
$Z_{1.0}$ (km)	0.060	0.060			0.162	
$\delta Z_{1.0}$ (km)	-0.102		**-0.102**			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.260			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.260			0.861		
Z_{hyp} (km)	999.00			**10.227**		
Z_{tor} (km)	7.68			0.000	0.000	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

CEA CALIFORNIA EARTHQUAKE AUTHORITY
Caltrans
PG&E

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 13673.003 Cordova Rd

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)						Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2	LB-4	LB-5	LB-8	LB-10		1.3	
5	7.5	60	60	60	47	60	60	58	75	0.10
10	5	72	60	60	60	60	100	69	89	0.06
15	5	100	100	100	100	100	100	100	100	0.05
20	5	100	100	100	60	60		84	100	0.05
25	5				100	100		100	100	0.05
30	5					100		100	100	0.05
35	5					100		100	100	0.05
40	5					100		100	100	0.05
45	5					100		100	100	0.05
50	7.5					100		100	100	0.08
60	10					100		100	100	0.10
70	10					100		100	100	0.10
80	10					100		100	100	0.10
90	10					100		100	100	0.10
100	5					100		100	100	0.05
Summatioı	100									1.03
								Navg = Sum(di) / Sum(di / Ni) =		**97**

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2019 CBC 1613A.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	**50.001**	**100**	1200	2500	366	762	97	2423
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

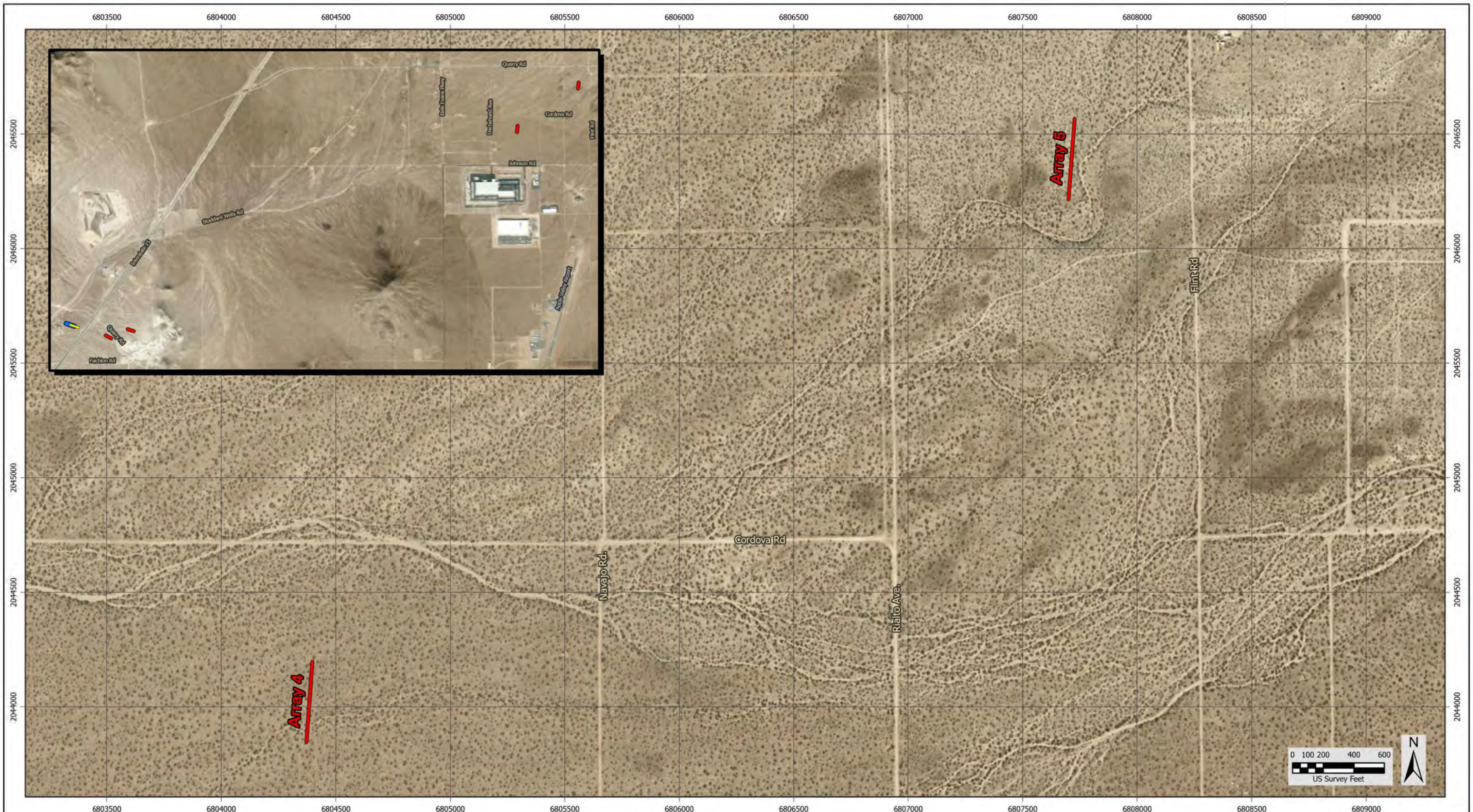
SITE CLASS, Table 20.3-1: $\boxed{\textbf{C}}$

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	2423	738
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1472	449
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1204	367
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	1196	365

APPENDIX E

GEOPHYSICAL DATA



MASW Array

Notes:
Coordinate System:NAD 1983 StatePlane California V FIPS 0405 Feet
Base map source: San Bernardino County, Earthstar Geographics, San Bernardino County, Maxar

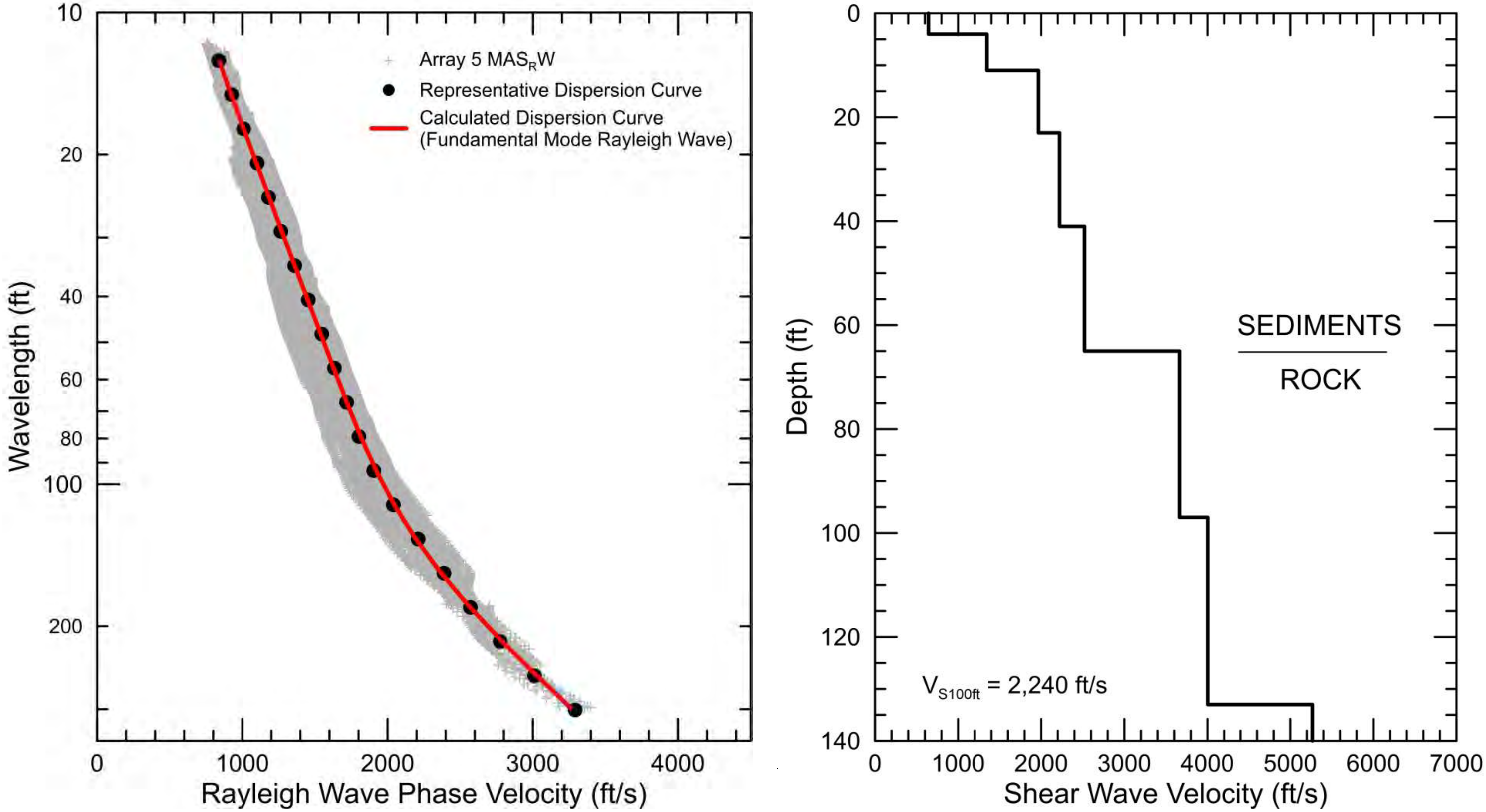
GEOVision geophysical services

Date:	11/8/2022
GV Project:	22441
Developed by:	A Martin
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	GV-22441

FIGURE 2
SITE MAP

VVLIG CORDOVA ROAD AND QUARRY ROAD WAREHOUSES APPLE VALLEY, CALIFORNIA

PREPARED FOR LEIGHTON CONSULTING, INC.



GEOVision geophysical services

FIGURE 8
SURFACE WAVE MODEL - ARRAY 5

Project No: 22441
Date: NOV 4, 2022
Drawn By: A MARTIN
Approved By: Antony J Martin

R:\GVProjects\2022\22441 - Leighton\Report\Figure 8.cdr

VVLIG QUARRY ROAD WAREHOUSE
APPLE VALLEY, CALIFORNIA

PREPARED FOR
LEIGHTON CONSULTING, INC.

Table 6 Array 5 S-wave Velocity Model (VVLIG Quarry Road Warehouse)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Inferred Unit Weight (lb/ft^3)
0.0	4.0	642	1201	0.300	114.0
4.0	7.0	1347	2519	0.300	123.0
11.0	12.0	1967	3680	0.300	129.0
23.0	18.0	2221	4156	0.300	131.0
41.0	24.0	2522	4716	0.300	132.0
65.0	32.0	3663	6855	0.300	137.0
97.0	36.0	4002	7489	0.300	140.0
133.0	Half Space	5267	9853	0.300	145.0

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

APPENDIX F

LEIGHTON CONSULTING, INC.

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

D-1.0 GENERAL

D-1.1 <u>Intent</u>

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

D-1.2 <u>Role of Leighton Consulting, Inc.</u>

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

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

D-1.3 <u>The Earthwork Contractor</u>

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

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

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

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

D-2.0 PREPARATION OF AREAS TO BE FILLED

D-2.1 Clearing and Grubbing

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

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D2974). Nesting of the organic materials shall not be allowed.

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

D-2.2 Processing

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

D-2.3 Overexcavation

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

D-2.4 Benching

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

D-2.5 Evaluation/Acceptance of Fill Areas

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

D-3.0 FILL MATERIAL

D-3.1 Fill Quality

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

D-3.2 Oversize

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

D-3.3 Import

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

D-4.0 FILL PLACEMENT AND COMPACTION

D-4.1 Fill Layers

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

D-4.2 Fill Moisture Conditioning

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

D-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D1557 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

D-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D1557 laboratory maximum density.

D-4.5 Compaction Testing

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

D-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document approximate elevation and horizontal coordinates of each density test location, relying on site survey control provided by others. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton Consulting, Inc. can determine test locations with sufficient accuracy. Adequate grade stakes shall be provided.

D-5.0 EXCAVATION

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

D-6.0 TRENCH BACKFILLS

D-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2015 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

D-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least (≥) one-sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

D-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the *Standard Specifications for Public Works Construction* unless the Contractor can demonstrate to Leighton Consulting, Inc. and the Owner that their proposed fill lift can be compacted to the specified relative compaction using the proposed alternative equipment and method; and only if the building official, with the appropriate jurisdiction, approves this proposed lift thickness.