GEOTECHNICAL UPDATE REVIEW
PROPOSED RESIDENTIAL DEVELOPMENT
LOCATED AT THE SOUTHEAST CORNER OF SUMNER AVENUE AND SCHLEISMAN ROAD
CITY OF EASTVALE, CALIFORNIA

Prepared For

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December 20, 2024

Project No. 038.0000030332

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Attention: Mr. Brandon Roth

Subject: Report of Geotechnical Update Review
Proposed Sumner Place Residential Development
Tentative Tract 38031
Southeast of Sumner Avenue and Schleisman Road
City of Eastvale, Riverside County, California

In response to your request, Leighton and Associates, Inc. has conducted this geotechnical review to update our previous geotechnical reports for the proposed Sumner Place residential development located southeast of Sumner Avenue and Schleisman Road in the City of Eastvale, Riverside County, California. The purpose of this review was to evaluate the data presented in previous reports for this project (Leighton, 2013, 2020a, and 2020b), document the current geotechnical conditions of the surface at the site, and provide updated geotechnical recommendations for design and construction for the development as currently proposed.

Based on our review of previous reports (Leighton, 2013, 2020a, and 2020b), construction of the proposed residential development is feasible from a geotechnical standpoint provided the recommendations presented in this report are implemented during design and construction. The most significant geotechnical issues for this project found during our review are those related to the potential for strong seismic shaking and compressible near-surface soils. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project that update those previously provided.

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,

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1.0 INTRODUCTION

1.1 Site Location and Description

Tentative Tract 38031 is bounded by Schleisman Road to the north with residences beyond. Existing residential properties are also present to the west (west of Sumner Avenue), south (south of Orange Street), and east of the site (see Figures 1 and 2).

The approximately 9½-acre property is located on relatively flat terrain that drains gently to the south and is currently occupied by two residential structures, one in the northwest corner and the other in the southwest corner, with remnants of former commercial structures in the center of the site. Some concrete pavement and walls and several fences are also present on the site.

One of the residential buildings in the northwest corner of the site has been present prior to 1938, becoming more fully developed by 1948 and apparently not changing much since that time. The residential structure in the southwest corner was constructed between 1959 and 1967. The rest of the site appears to have been used for agricultural or ranch type work at least until 1967. Sometime before 2005, the central portion of the site was constructed with commercial buildings. By 2018, these commercial structures had been demolished. Since 2018, the majority of the site has sat vacant with the exception of the residential structures in the northwest and southwest corners (NETR, 2024).

1.2 Proposed Development

Based on the *Illustrative Site Plan – SP0* prepared by Urban Arena, dated October 30, 2024, the development is proposed to consists of approximately 109 alley-loaded single-family dwellings. We presume that these dwellings will generally be lightly loaded (wood-framed) buildings that are three stories or less. We also assume that the proposed development will include drainage, utility, street, sidewalk, landscape and associated improvements. Although grading plans for the site were not available at the time of this report, we expect relatively shallow cuts and fills to achieve design grade (generally on the order of 5 feet or less).

1.3 Purpose of Study

The purpose of this study has been to evaluate the general geotechnical conditions at the site based on our previous work onsite with respect to the proposed development and provide preliminary geotechnical recommendations for design and construction.

1.4 Previous Work

Geotechnical Investigation Report (Leighton, 2013):
Leighton provided a geotechnical investigation report presenting our findings, conclusions, and preliminary recommendations for the development of Tentative Tract 38031 with the residential design concept provided at that time, which included only single-family detached homes. That investigation included drilling, logging and sampling of six (6) hollow-stem auger borings throughout the site ranging in depth from approximately 21.5 to 51.5 feet below the existing ground surface (bgs). The most significant geotechnical issues identified in that study were those related to strong seismic shaking and potentially compressible soils. Based on our 2013 field exploration, laboratory testing, and engineering analysis, we had concluded that development of the site was feasible from a geotechnical point of view.

Draft Results of Infiltration Testing (Leighton, 2020a):
Leighton provided a geotechnical report presenting the results of our field infiltration testing for the subject site. Infiltration testing was conducted within four (4) hollow-stem auger borings (LI-1 through LI-4) at depths ranging from approximately 9 to 44 feet bgs. Testing locations and depths had been selected based on a preliminary site exhibit for the development proposed at the time of our field work. Measured, unfactored infiltration rates varied from 0.5 in./hr. to 24 in./hr. Based on our site observations, laboratory testing, and infiltration test results, Leighton concluded that because soil types and infiltration rates were highly variable throughout the site, additional testing was recommended to be performed to evaluate the specific conditions at design locations and invert depths or during installation of BMP facilities. Leighton also updated the seismic design recommendations originally presented in our 2013 report to be in conformance with the 2019 California Building Code, which was the current code at the time of our 2020 infiltration report.

Geotechnical Supplement (Leighton, 2020b):
Leighton provided a geotechnical update to our 2013 investigation report to provide recommendations for a design that included multi-family apartment buildings that were three-stories or less. Based upon our review, the recommendations presented in our previous reports (Leighton, 2013 and 2020a) were deemed to remain applicable.

1.5 Scope of Work

The scope of our this geotechnical update review has included the following tasks:

- Literature Review: We reviewed available, relevant geotechnical geologic maps, reports, and aerial photographs available from our in-house library, available online, or provided by you. Our review included previous exploration reports provided by Leighton (2013, 2020a, and 2020b). Other references used in the preparation of this report are also listed at the end of this report.

- Engineering Analysis: Data obtained from our geotechnical update review, along with data collected during our previous field explorations and geotechnical laboratory testing (Leighton, 2013, 2020a, and 2020b) were analyzed to develop geotechnical conclusions and provide the geotechnical recommendations presented in this report.

- Report Preparation: Results of our geotechnical review have been summarized in this report, presenting a summary of our findings, conclusions and updated geotechnical recommendations for design and construction of the residential development as currently proposed.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located on a valley alluvial plain descending southward from the San Gabriel Mountains to the Santa Ana River. It is located within the Upper Santa Ana River Valley, which is an alluvium-filled depression formed by erosion during uplift of surrounding mountains. Several active or potentially active faults have been mapped in the region and are believed to accommodate compressional and lateral crustal displacement from tectonics associated with the San Andreas transform system, which defines the location of the interaction between the North American and Pacific tectonic plates. The San Andreas transform includes the San Andreas fault zone as well as other components of the system such as the San Jacinto, Elsinore, and Cucamonga fault zones. Active tectonics associated with the San Andreas transform system has also resulted in the uplift of the mountains surrounding the Upper Santa Ana River Valley, which includes the San Gabriel, San Bernardino, and San Jacinto Mountains as well as the Puente Hills.

The project site has been regionally mapped to be underlain by middle Holocene-age young alluvial fan deposits (see Figure 3, *Regional Geology Map*). Sediment from this unit has been described to consist of silt, sand, and coarse-grained sand to bouldery deposits (Morton and Miller, 2006).

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our previous subsurface exploration (Leighton, 2013 and 2020a), the site is underlain by alluvial soil deposits. The alluvial soil encountered within our excavations generally consisted of combinations of silty sand and sandy silt. Silty and clayey layers were encountered at depth of 10 to 20 feet in one of our borings. The soil was generally moist and stiff to very stiff or medium dense to dense. The in-situ moisture content within the upper approximately 10 feet generally ranged from 4 to 11 percent. More detailed descriptions of the subsurface soil are presented in Appendix B – Exploration Logs.

2.2.1 Expansive Soil

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 both building foundations and slabs-on-grade could result.

A representative sample of the onsite soil was tested for expansion potential (Leighton, 2013). That test result indicated an expansion index of 5. Based this test result and on our experience in the area, onsite soils are expected to have low to very low expansion potential.

2.2.2 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 the American Concrete Institute (ACI) publication 318-19, Section 19.3 (ACI, 2019), adopted by the 2022 CBC (Section 1904A.2).A near-surface soil sample was tested during our past investigation for soluble sulfate content. The results of these tests indicate a maximum sulfate content of about 0.02 percent by weight. As such, the soils exposed at grade are expected to pose negligible potential (Exposure Class S0) for sulfate reaction with concrete.

2.2.3 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content, and pH level. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive, while soil having a minimum resistivity of 1,000 to 2,000 is considered corrosive. Soil with a chloride content of 500 ppm or greater is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during our prior investigation to estimate minimum resistivity, chloride content, and pH. The test indicates minimum resistivity of 878 ohm-cm, chloride content 24 ppm, and pH of 7.5. Based on these results, the onsite soil is considered severely corrosive to ferrous metals.

2.3 Groundwater

During our previous investigation (Leighton, 2013), no groundwater was encountered within borings explored to depths of 51.5 feet. During subsequent exploration (Leighton 2020), groundwater was encountered at a depth of approximately 50 feet below ground surface (bgs) within Borings LI-2 and LI-3.

Based on our review of available data, the depth to groundwater was on the order of 75 feet below the ground surface during 2006 (Chino Basin Watermaster, 2007).

An unpublished map prepared by the County of Riverside (Riverside County, 2004) indicates that the historically shallowest groundwater level at the site is estimated to have been on the order of 8 feet beneath the ground surface. The Chino Basin Watermaster controls the elevation of groundwater in the basin through a series of desalter wells. The wells draw water to reduce the nitrate levels in the water and to monitor and control the groundwater fluctuations in the basin. With such monitoring and control, shallow groundwater levels at the site in the future are unlikely.

2.4 Faulting and Seismicity

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

Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Chino - Elsinore fault zone, located approximately 6 miles west of the site.

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include the Chino-Elsinore fault zone, San Jose, Whittier, Cucamonga, Sierra Madre, Puente Hills, San Jacinto, and San Andreas faults.

2.4.1 Faulting

The site has been mapped outside any state of California Alquist-Priolo Fault Zones or any County of Riverside fault zones. Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Chino - Elsinore fault zone, located approximately 6 miles west of the site.

2.4.2 Ground Shaking

The site will experience strong ground shaking after the proposed project is developed 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 related to seismic shaking can be reduced.

The following parameters should be considered for design under the 2022 CBC:

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude (degrees):	33.9600, -117.5747
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_S	1.656 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.593 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.707*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.656 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.012* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.104 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.675* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.682 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.750 g

* See section 11.4.8 of ASCE 7-16. A site specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. Per supplement 3 to ASCE 7-16, a site specific ground motion hazard analysis is not required where the value of the parameters S_{M1} and S_{D1} in the table are **increased by 50%.**

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

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate the predominant modal earthquake has a magnitude of approximately 6.7 (MW) at a distance of approximately 5.8 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

2.5 Secondary Seismic Hazards

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

2.5.1 Liquefaction Potential

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

The State of California has evaluated the site for liquefaction hazards. The County of Riverside has mapped the northwest portion of the site as having very high liquefaction potential and the southeast portion as having high liquefaction potential (Riverside County, 2024).

Historical groundwater levels have been estimated to have been at a depth of approximately 8 feet bgs based on available groundwater data. Although we anticipate groundwater levels at the site unlikely to reach this shallow of a depth in the future, we have analyzed the potential for liquefaction using a design groundwater level of 8 feet bgs

Our analysis was 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 et 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 historically highest level. In the SPT method, soil resistance to liquefaction is estimated based on several factors, including SPT sampling blow counts normalized and corrected for several factors including fines content, and overburden pressure. Soil plasticity and

moisture content are also considered in an evaluation of liquefaction. Parameters utilized in our analysis include Standard Penetration Test (SPT) results from the borings, visual descriptions of soil samples retrieved, and geotechnical laboratory test results.

Based on our analysis, subsurface soil layers are considered susceptible to liquefaction when considering a design historic high groundwater depth of 8 feet bgs and peak ground acceleration. Potentially liquefiable layers were encountered at a depth of 12.5 feet to 17.5 feet within borings LB-1 and LB-3 and at a depth of 32.5 feet to 37.5 feet bgs within boring LB-3. The identified liquefiable layers within LB-1 consisted of stiff clays. The identified liquefiable layers within LB-3 generally consisted of stiff silt layers and loose silty sand layers. No identified layers were encountered within our other deep borings previously advanced onsite (LI-2 and LI-3).

Even though our analysis was conducted assuming relatively shallow ground water conditions (8 feet below the ground surface), as previously noted, the Chino Basin Watermaster currently controls the elevation of groundwater in the basin through a series of desalter wells. With such monitoring and control, shallow groundwater levels such as those used in our analysis are very unlikely to return. Under current and estimated future groundwater conditions, liquefaction would not be expected onsite.

A key aspect of liquefaction is what effect it may have on the proposed improvements, in terms of surface manifestations, lateral spreading, and seismic settlement. These are addressed in the following sections.

A summary of the liquefaction analysis is included in Appendix D.

2.5.2 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. Our analysis, based on this method, indicates no potential for structural damage due to surface manifestations for the potentially liquefiable layers identified in the previous section. Results of our analysis are included in Appendix D.

2.5.3 Lateral Spreading

Based on the $(N_1)_{60}$ blow counts and depth to liquefiable soils, the site is not considered susceptible to flow slides (large transitional or rotational failures), the most catastrophic form of liquefaction-induced ground failure. Below, we evaluate the potential of lateral spreads, which are liquefaction-induced lateral ground movements limited to on the order of several feet or less, and, thus, smaller than flow failures.

A consideration in lateral spread analysis is to evaluate whether laterally continuous liquefiable layers exist. It appears that continuous layers do not span the site.

We performed lateral spreading analysis based on the Youd et al. (2002) empirical method, which considers the thickness of certain liquefiable layers based on sampling blow count data, along with grain size and fines content, earthquake characteristics, sloping ground conditions, and nearby free-face conditions. Based on Bartlett and Youd (1995), the free-face condition used in this method is apparently for channels or rivers. This method does not directly consider the depth of potentially liquefiable layers, but the authors indicated that sites that experienced lateral spread usually had susceptible layers that were within a few meters of the surface (Bartlett and Youd, 1995).

Based on our analysis, identified potentially liquefiable layers within our borings that had a value of $(N_1)_{60}$ of 15 or less were generally located at a depth of 12.5 to 17.5 feet bgs within boring LB-1 and at a depth of 12.5 feet to 17.5 feet bgs and 32.5 feet to 37.5 feet bgs within boring LB-3. However, a majority of the identified materials are high fines content soils (silts and clays) and based on Youd et al (2009), this material is likely not susceptible to lateral spreading.

Granular layers with an $(N_1)_{60}$ value less than 15 were identified within LB-3 at a depth of 12.5 to 17.5 feet bgs. The estimated lateral spreading potential for the identified layers is under 11 inches for sloping ground conditions and under approximately 2 inches for free face conditions associated with the Santa Ana River south of the project site; these values are below the threshold values for lateral spreading horizontal ground displacement for shallow foundations presented in ASCE 7-16 Table 12.13-2, where the upper limit is 18 inches for Risk Category II buildings.

2.5.4 Seismically Induced Settlement

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

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 8 feet below ground surface were used in the analysis. Based on our analysis, a potential for approximately 2.8 inches of seismic settlement is estimated at the site. Results of our seismic settlement analysis are presented in Appendix D.

The potential differential settlement is estimated as half of the total seismic settlement over a horizontal distance of 30 feet, resulting in 1.4 inch of differential settlement in 30 feet, or angular distortion of 0.0039L.

The structural engineer should determine Structure Type and Risk Category and evaluate whether the differential settlement estimates described above are tolerable.

Table 12.13-3 Differential Settlement Threshold

Structure Type	Risk Category		
	I or II	III	IV
Single-story structures with concrete or masonry wall systems	0.0075L	0.005L	0.002L
Other single-story structures	0.015L	0.010L	0.002L
Multistory structures with concrete or masonry wall systems	0.005L	0.003L	0.002L
Other multistory structures	0.010L	0.006L	0.002L

2.5.5 Seismically Induced Landslides

The site is generally level without significant slopes. This site is not considered susceptible to static slope instability or seismically induced landslides.

2.6 Infiltration Testing

Well permeameter tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. The test consists of excavating a boring to the depth of the test (or deeper if it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean gravel or sand is placed in the boring bottom to support temporary perforated well casing pipe and a float valve. In addition, coarse 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 float valve, lowered into the boring inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a relatively constant water head in the boring. The incremental infiltration rate as measured during intervals of the test is defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface. The test was conducted based on the USBR 7300-89 test method and in general conformance with Riverside County guidelines.

Four well permeameter tests (LI-1 through LI-4) were conducted onsite during Leighton's previous exploration (2013) to estimate the infiltration rate of native soils. Soils at the site are variable and yielded various infiltration rates. A summary of the infiltration tests performed is provided in the following table.

Boring	Depth (ft)	Material	Unfactored Infiltration Rate (in/hr)
LI-1	9	Sand with Silt	24
LI-2	44	Sand with Silt	3.5
LI-3	43	Sand	7
LI-4	9	Silty Sand	0.5

3.0 CONCLUSIONS AND RECOMMENDATIONS

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

Although not identified during this investigation, abandoned septic tanks, seepage pits, or other buried structures, trash pits, or items related to past site uses may be present. If such items were encountered during grading, they would require further evaluation and special consideration.

3.1 General Earthwork and Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. Overexcavation and recompaction recommendations are presented in the following paragraphs. The General Earthwork and Grading Recommendations are included in Appendix E. In case of conflict, the following recommendations shall supersede those provided in Appendix E.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed, as should large trees and their root systems. 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 Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. For structures with shallow foundations, we recommend that onsite alluvial

soils be overexcavated and recompacted to a minimum depth of 3 feet below the bottom of the proposed footings or 5 feet below existing grade, whichever is deeper. In addition, any undocumented artificial fill should be overexcavated. Overexcavation and recompaction should extend horizontally a minimum of 5 feet beyond the outside edges of footings (including columns connected to buildings), or a distance equal to the depth of overexcavation below the footings, whichever is greater, and where feasible. Suitability of all removal bottoms should be reviewed and evaluated by an engineering geologist or representative of the geotechnical engineer.

Local conditions may require that deeper overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and site walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper. In addition, any undocumented artificial fill should be overexcavated.

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

All fills should be observed and tested in the field by a Leighton representative prior to fill placement or foundation or pavement construction to ensure adequate moisture conditioning and compaction requirements are met. Fill soil should be compacted to a minimum of 90 percent relative compaction per ASTM D1557. The upper 6 inches of subgrade soil in pavement areas and aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

These recommendations should be reviewed once a grading plan is available.

3.1.3 Fill Placement and Compaction

Onsite soil may be used for compacted structural fill provided it is 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, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. The upper 6 inches of subgradesoil in pavement areas and ggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 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.

3.1.5 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. 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 +/- 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.15 feet

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.

3.1.6 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 soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment of 18 inches, 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 1,800 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width 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 2,500 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 0.30. The passive resistance may be computed using an

allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

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

3.2.5 Settlement Estimates

The recommended overexcavation, relative compaction and allowable bearing pressure are based on a total allowable, post construction settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet between or along similarly loaded footings. 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 result in a maximum 1.4-inch differential settlement in 30 feet, or angular distortion of 0.0039L for the design earthquake.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a low expansion potential. 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 percent above optimum moisture content to a minimum depth of 18 inches prior to placing 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 (4 inches of clean ½-inch-minimum gravel) should be placed under the vapor retarder and whether or not a sand blotter layer should be placed over the vapor retarder. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a heavier vapor retarder can be used.

- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

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.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

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.

3.4 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 6 (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	240 pcf (allowable) (Maximum of 3,500 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 0.3 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

We recommend that the wall designs for walls 6 feet tall or taller be checked seismically using an *additive seismic* Equivalent Fluid Pressure (EFP) of 22 pcf, which is added to the EFP. The *additive seismic* EFP should be applied at the retained midpoint.

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

3.5 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 40, flexible pavement sections may consist of the following for the Traffic Indices 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, Type I Subgrade Soil			
Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Total Pavement Section Thickness (inches)
5 or less	3	4	7
6	3.5	5.5	9
7	4	7	11

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 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

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.

3.6 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.

3.7 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction* (SSPWC, “Greenbook”), 2018 Edition. Utility trenches may be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Granular Bedding:** a uniform sand material with a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer).

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit and vibrated.

We recommend that open-graded crushed rock or similar material not be used as bedding material, unless special provisions are implemented to limit the migration of surrounding soil into the open-graded material, including surrounding the open-

graded material with filter fabric (Mirafi 140N or equivalent), or mixing sand with the open-graded material. Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment.

Trench backfill over the pipe bedding zone may consist of native and clean fill soils. All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned to slightly above optimum, and mechanically compacted to at least 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557

3.8 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.

3.9 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. Type II cement may be used for concrete in contact with the soil. Concrete should be designed in accordance with Table 4.3.1 of the American Concrete Institute ACI 318-19 provisions (ACI, 2019).

Based on our laboratory testing, the onsite soil is considered severely corrosive to ferrous metals. Use of non-ferrous buried pipe may be prudent, or ferrous pipe can 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 corrosion protection is considered critical to the project.

3.10 Infiltration Testing

Results of Leighton's previous infiltration testing yielded highly variable infiltration rates. The locations and depths of our testing was selected based on an exhibit for a previous design concept. We understand that the newest design will incorporate dry wells, however, locations have not been provided at this time. We recommend that Leighton review preliminary BMP plans when available and conduct additional infiltration testing based on the proposed locations and depths.

3.11 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our preliminary 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:

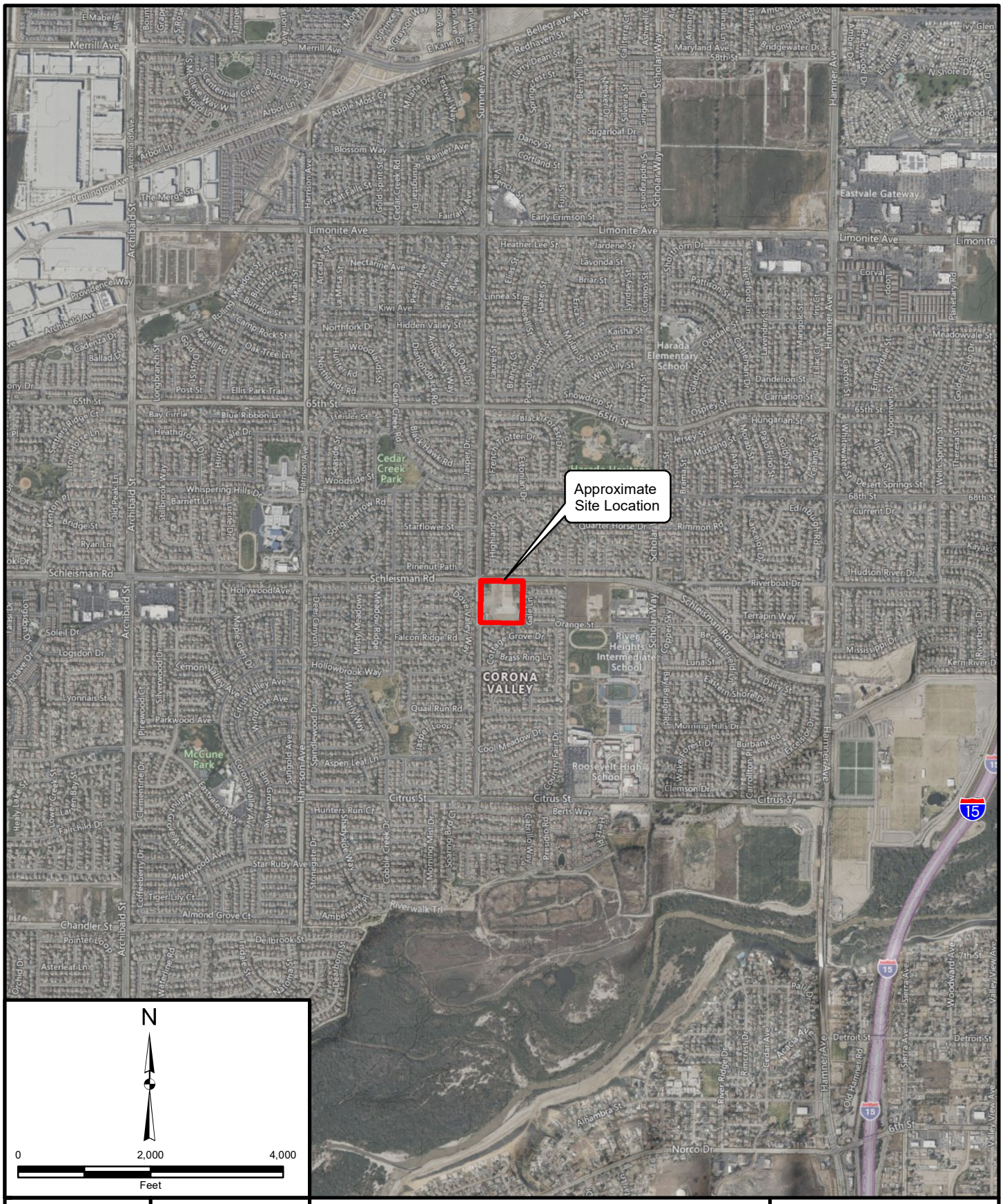
- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When 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 and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Stratham Homes, for application to the design of the proposed residential 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.



Project: 30332	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: December 2024

Reference: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus

SITE LOCATION MAP

Stratham Sumner Residential Development
Southeast Corner of Sumner Avenue
and Schleisman Road
City of Eastvale, Riverside County, California

FIGURE 1

Leighton

Map Saved as Z:\Project Files\SA-TZ\StrathamHomes\30332 - Stratham Eastvale Sumner Geo Update\GIS\Maps\30332_F01_SLM_2024-11-18.mxd on 11/18/2024 9:02:03 AM Author: KVM (btran)



Project: 30332	Eng/Geol: JDH/SGO
Scale: 1 " = 100 '	Date: December 2024

Service Layer Credits: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus DS © 2024 TomTom

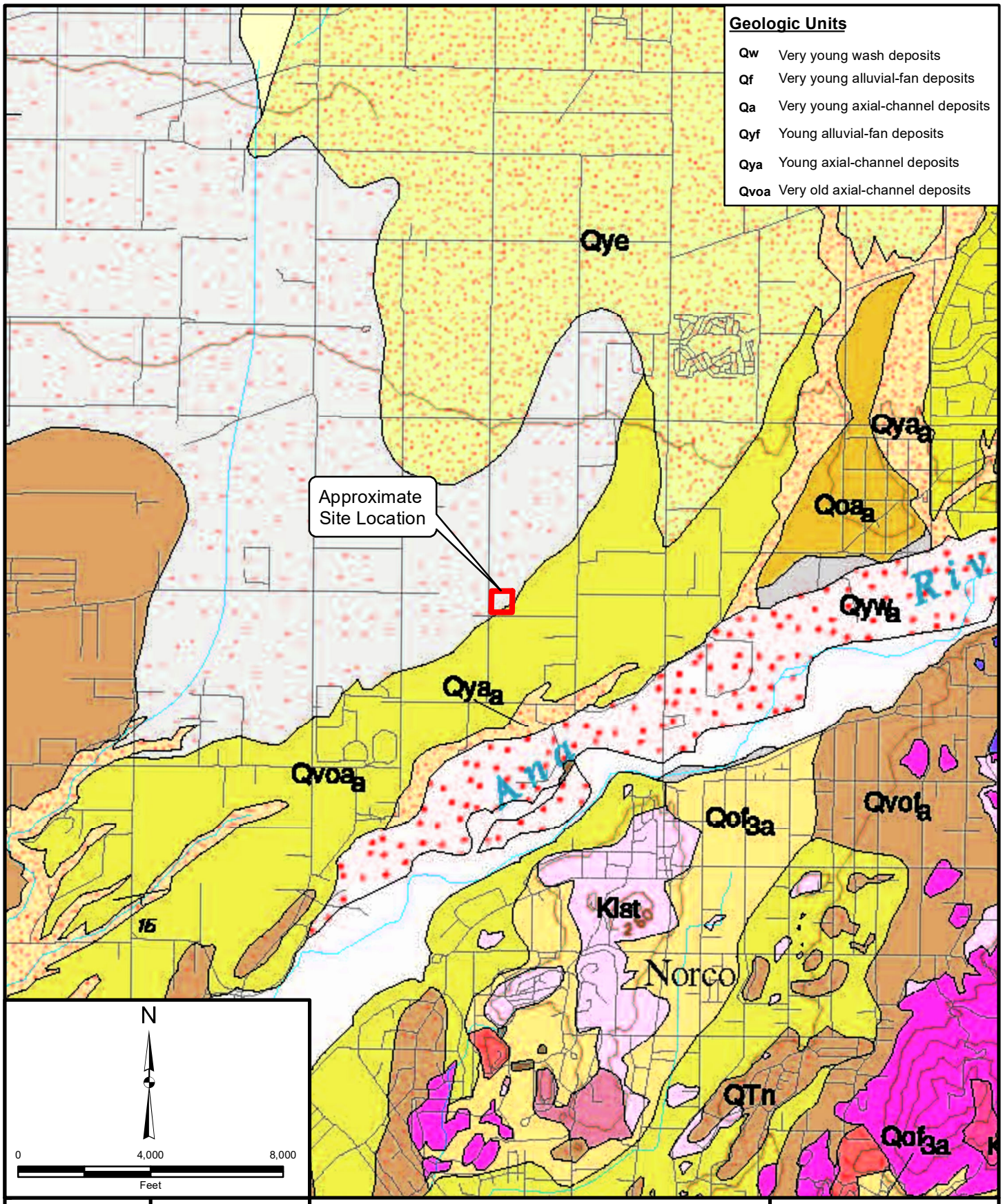
EXPLORATION LOCATION MAP

Stratham Sumner Residential Development
Southeast Corner of Sumner Avenue. & Schleisman Road
City of Eastvale, Riverside County, California

FIGURE 2

Leighton

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Project: 30332	Eng/Geol: JDH/SGO
Scale: 1 " = 4,000 '	Date: December 2024

Reference: Geologic Map of The San Bernardino and Santa Ana Quadrangles, California compiled by Doughlas Morton and Fred Miller, 2006

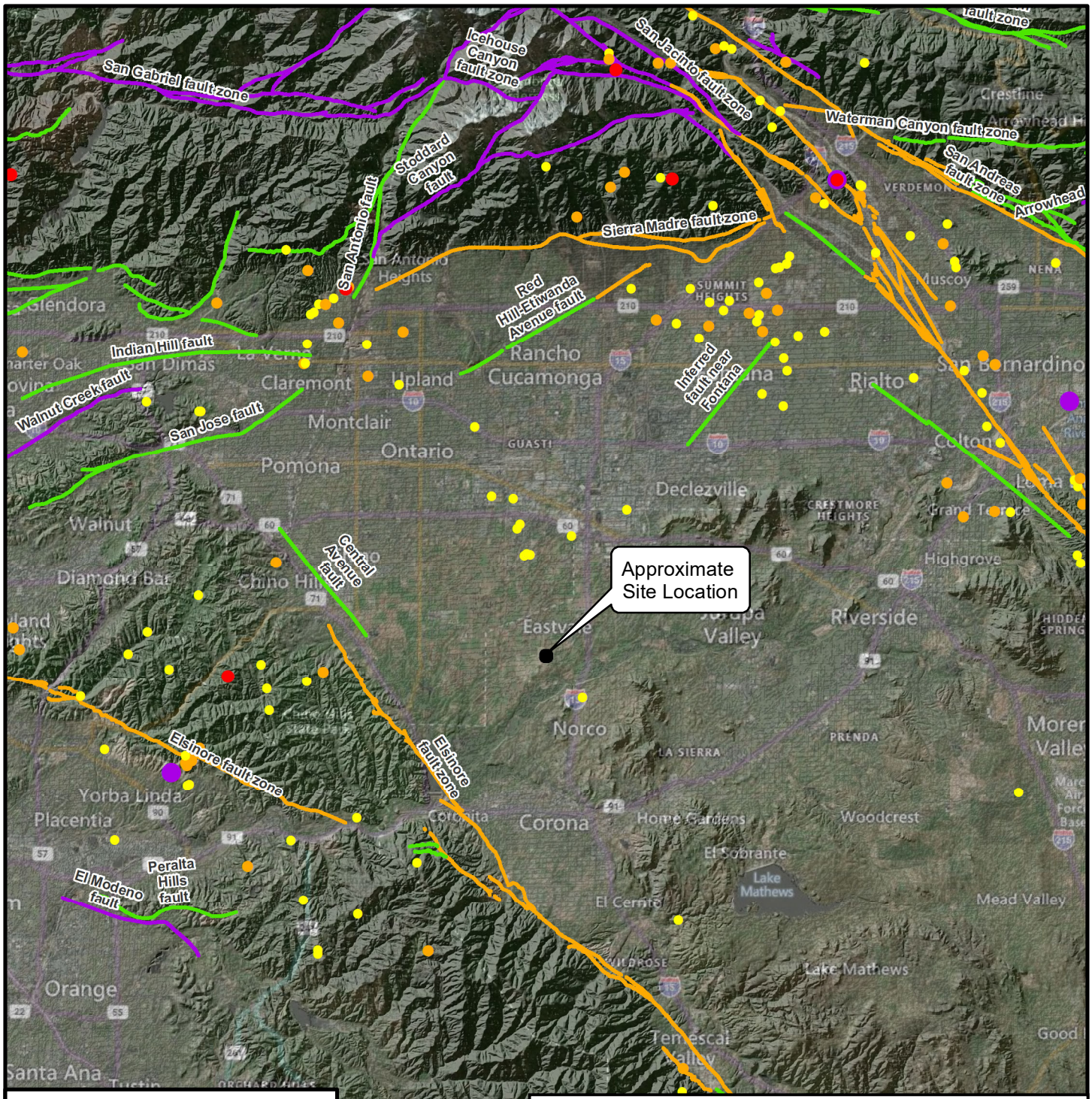
REGIONAL GEOLOGY MAP

Stratham Sumner Residential Development
Southeast Corner of Sumner Avenue
and Schleisman Road
City of Eastvale, Riverside County, California

FIGURE 3

Leighton

Map Saved as Z:\Project Files\SA-TZ\StrathamHomes\30332 - Stratham Eastvale Sumner Geo Update\GIS\Maps\30332_F03_RGM_2024-11-18.mxd on 11/18/2024 9:05:53 AM Author: KVM (btran)



LEGEND

Fault activity

Recency of Movement

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

Historical Earthquakes (≥M3.5)

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

Project: 30332	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: December 2024

Basemap Reference: © 2024 Microsoft Corporation
Earthstar Geographics SIO © 2024 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

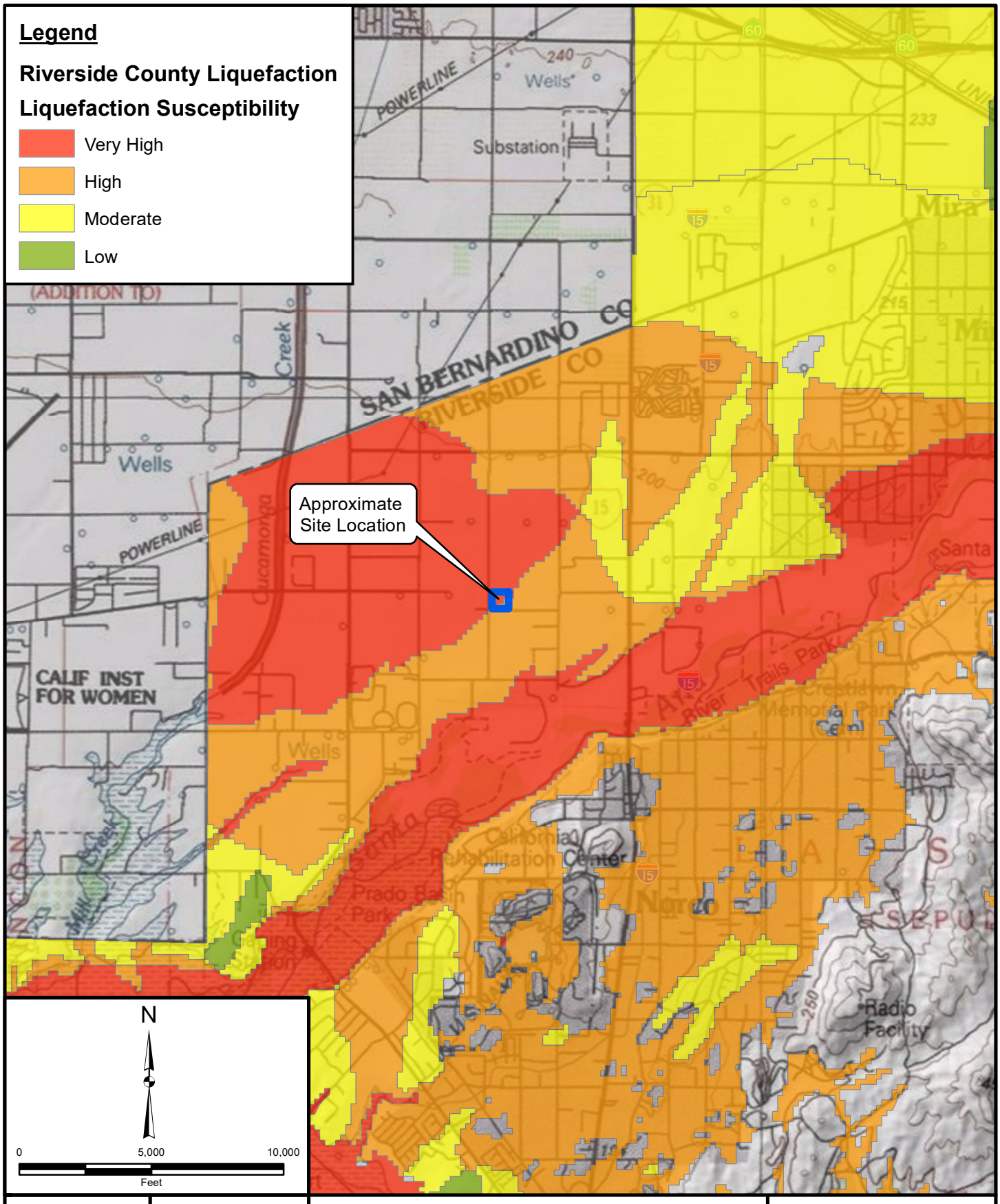
REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Stratham Sumner Residential Development
Southeast Corner of Sumner Avenue
and Schleisman Road
City of Eastvale, Riverside County, California

FIGURE 4

Leighton

Map Saved as Z:\Project Files\SA-TZ\StrathamHomes\30332 - Stratham Eastvale Sumner Geo Update\GIS\Maps\30332_F04_RFHSM_2024-11-18.mxd on 11/18/2024 9:10:08 AM Author: KVM (btran)



Project: 30332	Eng/Geol: JDH/SGO
Scale: 1 " = 5,000 '	Date: December 2024

Base Map: ESRI ArcGIS Online 2024
Reference: Riverside County Mapping Portal

LIQUEFACTION MAP

Stratham Sumner Residential Development
Southeast Corner of Sumner Avenue
and Schleisman Road
City of Eastvale, Riverside County, California

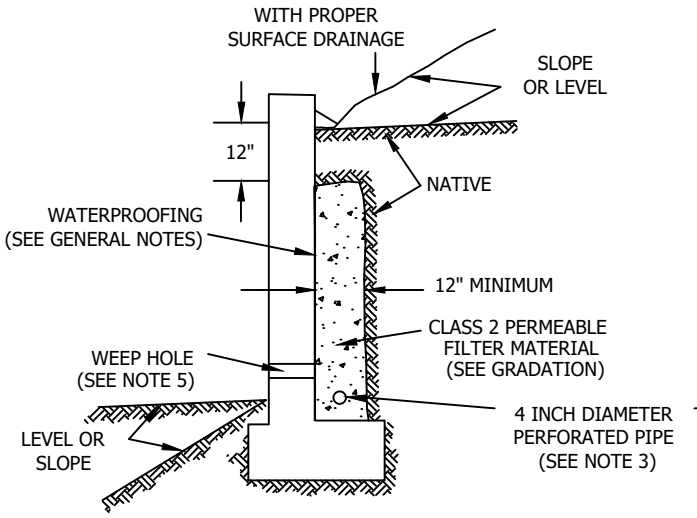
FIGURE 5

Leighton

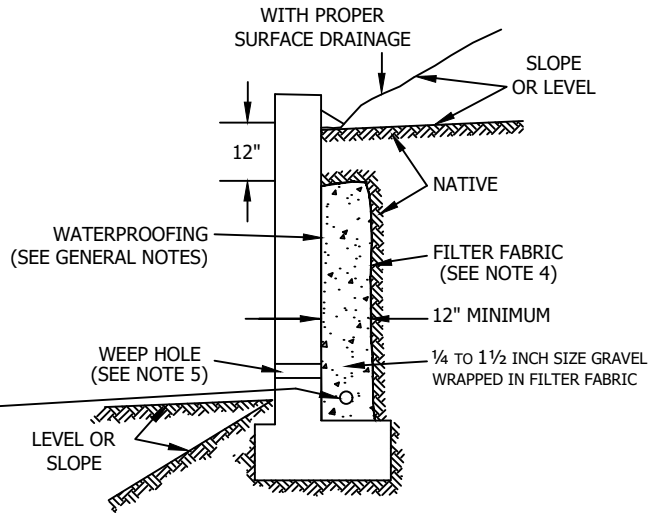
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SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 6

V:\DRAFTING\TEMPLATES\STANDARD-FIGURES\WALL-STANDARD-FIGURES.DWG (09-02-21 9:07:35AM) Plotted by bbain

APPENDIX A

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APPENDIX A

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APPENDIX B

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

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			BULK				SM	@Surface: dirt with coarse gravel **Alluvium (Qal)**	
				R-1	4 5 5	111	7	SM	@2.5' SILTY SAND, loose, olive brown, moist, 30% fines (field estimate), trace gravel, 1.25" max gravel size	
	5			R-2	5 10 10	84	6	SM	@5' SILTY SAND, medium dense, olive brown, moist, 40% fines (field estimate), trace gravel, 1.75" max gravel size	
	10			R-3	12 34 44	130	4	SP	@10' SILTY SAND, dense, moist, 20% fines (field estimate), with some gravel, 1.5" max gravel size	
	15			S-4	4 5 5			CL	@15' LEAN CLAY, stiff, yellowish brown, moist, with some silt	
	20			S-5	5 15 22			SC	@20' CLAYEY SAND, dense, yellowish brown, moist, coarse sand, 40% fines (field estimate), trace fine gravel	
	25			S-6	12 21 30			SM	@25' SILTY SAND, very dense, orange-brown, dry to moist, medium to coarse sand, 30% fines (field estimate), homogenous	
	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-1

Project No.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							*This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	
	30			S-7	10 14 15			SM	@30' SILTY SAND, medium dense, olive, moist, 40% fines, with some clay, specks of FeO2 staining	-200
	35			S-8	6 8 11			ML	@35' SILT, very stiff, olive, moist, 75% fines, some FeO2 staining, homogenous, PP=2.25	-200
	40			S-9	25 40 50/5			SP	@40' SAND, very dense, light gray, moist, trace fines, trace fine gravel	
	45			S-10	50/6			SP	@45' SAND, very dense, white brown, dry to moist, trace fine gravel	
	50			S-11	30 50/5			SP	@50' SAND, very dense, dark gray, moist, trace fines	
									Total depth of 50.9' No groundwater encountered Backfilled with spoils	
	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

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GEOTECHNICAL BORING LOG LB-2

Project No.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							*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			BULK				SM	@Surface: dirt with coarse gravel **Alluvium (Qal)**	MD
				R-1	3 3 5	118	10	SM	@2.5' SILTY SAND, loose, dark olive brown, moist, fine sand, 20% fines (field estimate), trace fine gravel	
	5			R-2	9 16 19	125	11	SP	@5' SAND, medium dense, light brown to dark gray, moist, trace fines, trace fine gravel	
	10			R-3	27 39 28	121	11	SM	@10' SILTY SAND, dense, light brown, moist, 30% fines (field estimate), with some FeO2 staining, homogenous	
	15			R-4	5 12 13			ML	@15' SANDY SILT, stiff, olive, moist, homogenous	CO
	20			R-5	23 50/6			SP	@20' SAND, very dense, light gray brown, moist, coarse sand, 10% fines (field estimate), trace FeO2 staining	
									Total depth of 21' No groundwater encountered Backfilled with spoils	
	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.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

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			BULK					@Surface: dirt with coarse gravel	EI, CR
									Alluvium (Qal)	
				R-1	7 7 7	109	7	ML	@2.5' SANDY SILT, stiff, brown, dry to moist, homogenous	
	5			R-2	5 6 7	105	4	SP	@5' SAND, loose, reddish light brown, dry to moist, fine to medium sand, 10% fines (field estimate)	
	10			R-3	17 32 44	108	18	ML	@10' SILT, hard, light olive to light reddish brown, dry, homogenous	
	15			S-4	5 5 3			SM	@15' SILTY SAND, loose, gray, moist, 45% fines, some FeO2 staining	-200
	20			S-5	10 22 28			SM	@20' SILTY SAND, dense, olive, moist, 40% fines (field estimate), with faint FeO2 staining, homogenous	
	25			S-6	10 17 23			SM	@25' SILTY SAND, dense, light brown, moist, fine sand, 40% fines (field estimate), homogenous	
	30									

The 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
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GEOTECHNICAL BORING LOG LB-3

Project No.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-7	17 24 25			ML	@30' SILT, very stiff, light olive, moist, homogenous	
	35			S-8	5 6 7			ML	@35' SILT, stiff, olive, moist, 71% fines, homogenous	-200
	40			S-9	3 11 15			SM	@40' SILTY SAND, medium dense, gray, dry to moist, with FeO2 staining, homogenous	
	45			S-10	41 50/5			SP	@45' SAND, very dense, gray, dry to moist, coarse sand, with some gravel, 2.5" max gravel size	
	50			S-11	17 23 24			SP	@50' SAND, very dense, gray, dry to moist, coarse sand, with trace fine gravel	
	55								Total depth of 51.5' No groundwater enountered Backfilled with spoils	
	60									

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

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

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

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GEOTECHNICAL BORING LOG LB-4

Project No.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

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			BULK				SP	@Surface: dirt with coarse gravel **Alluvium (Qal)**	
				R-1	16 27 28	136	2	SP	@2.5' SAND, dense, dark gray, dry to moist, fine to medium sand, 10% fines (field estimate), with some gravel, 1.25" max gravel size	
	5			R-2	20 28 25	117	3	SM	@5' SILTY SAND, dense, light brown, moist, 30% fines (field estimate), with some gravel, 3" max gravel size	
	10			R-3	15 18 30	111	7	SM	@10' SILTY SAND, medium dense, light brown, moist, 30% fines (field estimate), trace fine gravel, trace FeO2 staining	
	15			R-4	17 23 26			SM	@15' SILTY SAND, medium dense, light gray, moist, medium sand, 20% fines (field estimate), large portions of FeO2 staining	
	20			R-5	7 12 10			SM	@20' SILTY SAND, medium dense, dark gray to black, moist, very fine sand, 40% fines (field estimate), with some clay, trace organics, PP=1.5	
	25								Total depth of 21.5' No groundwater encountered Backfilled with spoils	
	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.	10290.001	**Date Drilled**	5-6-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

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			BULK				SM	@Surface: dirt with coarse gravel **Alluvium (Qal)**	
				R-1	3 5 13	114	7	SM	@2.5' SILTY SAND, medium dense, brown, dry to moist, 40% fines (field estimate), trace fine gravel	
	5			R-2	20 24 33	144	1	SP	@5' SAND with silt and gravel, dense, gray, dry to moist, 10% fines (field estimate), 2" max gravel size	
	10			R-3	10 12 15	110	17	SM	@10' SILTY SAND, medium dense, olive brown, moist, 40% fines (field estimate), pinhole porosity, homogenous, PP=4.5	CO
	15			R-4	12 15 25			ML	@15' SANDY SILT, very stiff, light gray, moist, homogenous, PP=4.5+	
	20			R-5	9 12 30			ML	@20' SILT, very stiff, olive to reddish brown, moist, some FeO2 staining, PP=3.0	
	25								Total depth of 21.5' No groundwater encountered Backfilled with spoils	
	30									

The 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
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GEOTECHNICAL BORING LOG LB-6

Project No.	10290.001	**Date Drilled**	5-3-13
Project	Stratham Eastvale Sumner and Schleisman	**Logged By**	J. Dupre
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2	**Sampled By**	J. Dupre

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							*This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	
	0								@Surface: dirt with coarse gravel	
									Alluvium (Qal)	
				R-1	4 10 17			SM	@2.5' SILTY SAND, medium dense, light olive gray, moist, very fine to fine sand, 40% fines (field estimate), trace fine gravel, a large amount of FeO2 staining	
	5								Total depth of 5' No groundwater encountered Backfilled with spoils	
	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 L1-1

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	599'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N–S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B1					medium dense, gray-brown, slightly moist, nonplastic, @surface: gravel 1-2 inch, weak cementation	
				R-1	11 25 18	111	2	SP-SM	@2.5' SAND with silt, medium dense, gray-brown, slightly moist, nonplastic	
595	5			S-2	6 7 12			SP-SM	@5' SAND with silt, medium dense, gray-brown, slightly moist, fine to coarse sand, nonplastic, weak cementation, poorly graded, gravel, trace gravel, 8% fines (lab)	
				S-3	20 50/3"				No Recovery	
590	10			S-4	10 11 8			SM	@10' SILTY SAND, medium dense, olive brown, moist, fine to coarse sand, nonplastic, moderate cementation, 42% fines (lab)	
				R-5	13 23 27	118	14	SM	@12.5' SILTY SAND, medium dense, dark brown, moist, 20-30% fines (field estimate), moderate to strong cementation	
585	15			S-6	6 6 8			ML	@15' SILT with sand, stiff, olive-green, moist, moderate to strong cementation	
580	20								**Total Depth: 16.5 feet** **Groundwater not encountered.** **Well permeameter test performed at 9 feet.** **Backfilled with soil cuttings**	
575	25									
570	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
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- 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 L1-2

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	598'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1					@surface: minor vegetation, gravel, some debris	
595				R-1	5 10 17	116	11	SM	@2.5' SILTY SAND, medium dense, brown, moist, 20% fines (field estimate), moderate cement, some gravel	
	5			S-2	4 2 3			SM	@5' SILTY SAND, very loose, tan-brown, slightly moist, nonplastic, 20-30% fines (field estimate), weak cementation	
590				R-3	50/6"	100	9	SM	@7.5' SILTY SAND, very dense, brown, moist, nonplastic, 17% fines (lab), some gravel, weak to moderate cementation	
	10			S-4	9 20 30			ML	@10' SILT with sand, medium dense, brown, moist, moderate cementation, some gravel	
585				R-5	22 28 45	111	9	ML	@12.5' SANDY SILT, dense, brown, moist, fine to medium sand, moderate cementation	
	15			S-6	6 7 14			SM	@15' SILTY SAND, medium dense, gray brown, moist, 30% fines (field estimate), weak to moderate cementation	
580										
	20			R-7	13 38 39	111	7	ML	@20' SILT with sand, dense, gray, moist, fine to coarse sand, weak to moderate cementation	
575										
	25									
570										
	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 L1-2

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	598'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30							ML		
565										
	35			S-8	10 15 12			ML	@35' SANDY SILT, medium dense, gray brown, moist, fine sand, 58% fines (lab), weak to moderate cementation	
560										
	40			R-9	31 50/6"	110	7	SP-SM	@40' SAND with Silt, very dense, gray brown, moist, medium to coarse sand, 7% fines (lab), poorly graded, gravel 2 in. (field estimate) weak to moderate cementation	
555										
	45			S-10	50/6"			SP-SM	@45' SAND with Silt, very dense, gray brown, moist, medium sand, some gravel	
550										
	50			R-11	50/6"	106	9	SP	@50' SAND, very dense, gray-brown, very moist, medium to coarse sand, 1-2 in. gravel (field estimate), weak cementation	
545										
	55			S-12	10 18 23			SP-SM	@55' SAND with gravel, medium dense, gray brown, wet, fine to coarse sand, 10% fines (field estimate), moderate cementation	
540									**Total Depth: 56.5 feet** **Groundwater encountered at 49' 6"** **Well permeameter test performed at 43 feet** **Backfilled with soil cuttings**	
	60									

The 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 L1-3

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	596'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
595	0			B-1				SM	@surface: minor vegetation, silty sand with gravel	
				R-1	11 7 4	117	8	SM	@2.5' SILTY SAND, loose, brown, moist, fine to coarse sand, 20% fines (field estimate)	
590	5			S-2	2 2 3			SM	@5' SILTY SAND, very loose, light brown, moist, fine to medium sand, 20-30% fines (field estimate)	
				R-3	19 33 54	116	5	SM	@7.5' SILTY SAND, very dense, brown, moist, 18% fines (lab)	
585	10			S-4	19 34 44			SM	@10' SILTY SAND, dense, tan, moist, fine to medium sand, 20% fines (field estimate)	
				R-5	13 19 25	106	21	ML	@12.5' SANDY SILT, medium dense, gray brown, moist, fine to coarse sand, 58% fines (lab)	
580	15			S-6	4 6 14			ML	@15' SANDY SILT, medium dense, gray brown, very moist, some gravel	
575	20			R-7	50/6"	114	9	ML	@20' SANDY SILT with clay, very dense, brown, moist, 50-60% fines (field estimate), top of sample sandy gravel	
570	25			S-8	15 18 38			SM	@25' SILTY SAND, dense, grayish brown, moist, medium to coarse sand, nonplastic, 22% fines (lab)	
	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
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GEOTECHNICAL BORING LOG L1-3

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	596'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
565	30			R-9	50/6"	105	6		@30' SILTY SAND, very dense, brown, moist, 40% fines (field estimate), weak cementation, some gravel	
560	35			S-10	20 50/6"			SP-SM	@35' SAND with silt, very dense, gray to light brown, moist, 10-15% fines (field estimate), some gravel	
555	40			R-11	50/4"	123	3	SP	@40' SAND, very dense, gray, moist, medium to coarse sand, moderate cementation, 3% fines (lab)	
550	45			S-12	23 50/5"				@45' SAND, very dense, gray, moist, medium to coarse sand, 10-15% fines (field estimate), weak cementation, gravel towards top	
545	50			R-13	37 50/6"	133	10		@50' SAND, very dense, gray brown, wet, 2 in. gravel (field estimate), moderate cementation	
540	55			S-14	16 50/5"				@55' SAND, very dense, gray brown, wet, medium to coarse sand, 2 in. gravel (field estimate), strong cementation	
	60								**Total Depth: 56 feet** **Groundwater encountered at 50 feet** **Well permeameter test conducted at 42 feet** **Backfilled with soil cuttings**	

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 L1-4

Project No.	10290.002	**Date Drilled**	2-20-20
Project	Stratham Sumner Residential	**Logged By**	AIK
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	597'
Location	See Figure 2	**Sampled By**	AIK

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				ML	@surface: vegetation	
595				R-1	10 17 30	123	3	ML	@2.5' SANDY SILT, medium dense, light brown to gray, moist, fine to medium sand, moderate cementation	
	5			S-2	8 18 30			SM	@5' SILTY SAND, medium dense, brown gray, slightly moist, weak cementation, gravel	
590				R-3	6 9 10			SM	@7.5' SILTY SAND, medium dense, brown, moist, fine to coarse sand, 20-30% fines (field estiamte), gravel	
	10			S-4	2 4 7			ML	@10' SILT with sand, stiff, greenish brown to olive, moist, moderate to strong cementation	
585				R-5	7 9 15			SM	@12.5' SILTY SAND, medium dense, olive brown, moist, medium to coarse sand, moderate cementation	
	15			S-6	4 10 14			ML	@15' SILT with sand, medium dense, olive brown, moist, moderate cementation	
580									**Total Depth: 10 feet** **Groundwater not encountered.** **well permeameter test conducted at 9 feet** **Backfilled with soil cuttings**	
	20									
575										
	25									
570										
	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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Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: 10290.002

Exploration #/Location: LI-1

Depth Boring drilled to (ft): 15

Tested by: AK

USCS Soil Type in test zone: SP-SM

Weather (start to finish): Sunny

Liquid Used/pH: H2O

Measured boring diameter: 8 in. 4 in. Well Radius

Approx Depth to GW below GS: 50 ft

Well Prep: Drilled to 15', Caved to 9', Set 2" of Gravel at Bottom, Set Pipe, Backfill to 7' gravel outside pipe.

Initial estimated Depth to Water Surface (in.): 94

Average depth of water in well, "h" (in.): 14

approx. h/r: 3.5

Tu (Fig. 8) (ft): 42.2

Tu>3h?: yes, OK

Cross-sectional area for vol calcs (in.^2): 20.1

	ft	in.	Total (in.)
Depth to Bot of well (or top of soil over Bentonite)	9. ft		108
Pilot Tube stickup (+ is above ground)	1. ft	6. in.	18
Depth to top of sand outside of casing from top of pilot tube			
Depth to top of float assembly from top of pilot tube			0
Float Assembly ID		DHVA	
Float assembly Extension length (in.)			

-18 Depth below GS (in.)

Flow Meter:

Meter ID:

Meter Color: Black

Meter Units: Gallons

DL ID:

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of pilot tube)		Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3)			Flow (in^3/ min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
		Reading (cu-ft or gal)	Interval Pulse Count											from supply	from Δh	Total					
Start Date	Start time:	Gallons		ft	in.																
2/21/20	9:10	3114.1		9.7					0	98.4	9.6										
2/21/20	9:20	3125.8		9.5				10	10	96.0	12.0	2.4	11	2703	-48	2655	265	15927	0.9	16.60	**45.64**
2/21/20	9:31	3137.9		9.3				11	21	93.6	14.4	2.4	13	2795	-48	2747	250	14983	0.9	12.38	**36.16**
2/21/20	9:41	3149.1		9.2				10	31	92.4	15.6	1.2	15	2587	-24	2563	256	15379	0.921	11.72	**33.18**
2/21/20	9:51	3162.1		9.2				10	41	92.4	15.6	0	16	3003	0	3003	300	18018	0.9	14.06	**37.55**
2/21/20	10:01	3176.1		9.1				10	51	91.2	16.8	1.2	16	3234	-24	3210	321	19259	0.9	13.25	**38.82**
2/21/20	10:11	3185.8		9.1				10	61	91.2	16.8	0	17	2241	0	2241	224	13444	0.9	9.45	**26.23**
2/21/20	10:22	3199.5		9.1				11	72	91.2	16.8	0	17	3165	0	3165	288	17262	0.9	12.13	**33.68**
2/21/20	10:32	3213.8		9.1				10	82	91.2	16.8	0	17	3303	0	3303	330	19820	0.9	13.93	**38.67**
2/21/20	10:42	3225.9		8.9				10	92	88.8	19.2	2.4	18	2795	-48	2747	275	16482	0.9	9.23	**30.23**
2/21/20	10:52	3237.1		9.1				10	102	90.6	17.4	-1.8	18	2587	36	2623	262	15740	0.9	10.83	**28.44**
2/21/20	11:02	3247.4		9.0				10	112	90.0	18.0	0.6	18	2379	-12	2367	237	14204	0.9	8.96	**26.45**
2/21/20	11:12	3263.0		8.9				10	122	88.8	19.2	1.2	19	3604	-24	3580	358	21477	0.9	12.25	**38.24**
2/21/20	11:22	3285.1		8.9				10	132	88.8	19.2	0	19	5105	0	5105	511	30631	0.9	17.79	**53.00**
2/21/20	11:54	3313.7		9.3				32	164	93.6	14.4	-4.8	17	6607	96	6703	209	12568	0.9	12.03	**24.52**
2/21/20	12:11	3326.5		9.3				17	181	93.6	14.4	0	14	2957	0	2957	174	10436	0.9	9.10	**23.34**
2/21/20	12:31	3340.1		9.6				20	201	97.2	10.8	-3.6	13	3142	72	3214	161	9642	0.9	13.74	**24.22**
2/21/20	12:54	3355.9		9.5				23	224	95.4	12.6	1.8	12	3650	-36	3614	157	9427	0.9	9.40	**25.24**

template updated: 8/14/19

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

Leighton

Project:	10290.002
Exploration #/Location:	LI-2
Depth Boring drilled to (ft):	55
Tested by:	AK
USCS Soil Type in test zone:	SP-SM
Weather (start to finish):	Sunny
Liquid Used/pH:	H2O
Measured boring diameter:	8 in.
Approx Depth to GW below GS:	50 ft
Well Prep:	Drilled to 55', Pulled Augers, Caved to 44.5', Placed Gravel

4 in. Well Radius

Initial estimated Depth to Water Surface (in.):	497
Average depth of water in well, "h" (in.):	31
approx. h/r:	7.6
Tu (Fig. 8) (ft):	8.5
Tu>3h?:	yes, OK
Cross-sectional area for vol calcs (in.^2):	49.3

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	44. ft		528	
Pilot Tube stickup (+ is above ground)	2. ft		24	
Depth to top of sand outside of casing from top of pilot tube	37. ft			
Depth to top of float assembly from top of pilot tube			0	-24 Depth below GS (in.)
Float Assembly ID		DHVA		
Float assembly Extension length (in.)				

Flow Meter:

Meter ID	
Meter Color	Black
Meter Units	Gallons
DL ID	

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of pilot tube)		Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3)			Flow (in^3/ min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
		Reading (cu-ft or gal)	Interval Pulse Count											from supply	from Δh	Total					
Start Date	Start time:	Gallons		ft	in.																
2/20/20	12:40	3015.3		44.8					0	513.8	14.2										
2/20/20	12:50	3036.5		40.8				10	10	465.6	62.4	48.24	38	4900	-2380	2520	252	15119	0.9	1.17	**13.77**
2/20/20	13:00	3056.0		40.3				10	20	459.7	68.3	5.88	65	4507	-290	4217	422	25301	0.9	2.03	**13.78**
2/20/20	13:20	3069.3		44.8				20	40	513.7	14.3	-54	41	3063	2664	5727	286	17181	0.921	26.23	**14.56**
2/20/20	13:30	3071.9		44.9				10	50	514.8	13.2	-1.08	14	601	53	654	65	3923	0.9	3.96	**9.14**
2/20/20	13:40	3076.2		43.3				10	60	495.1	32.9	19.68	23	1005	-971	34	3	204	0.9	0.04	**0.30**
2/20/20	13:50	3079.1		43.6				10	70	498.6	29.4	-3.48	31	675	172	846	85	5077	0.9	1.61	**5.62**
2/20/20	14:20	3086.5		43.3				30	100	495.8	32.2	2.76	31	1691	-136	1555	52	3110	0.9	0.82	**3.48**
2/20/20	14:30	3088.8		43.3				10	110	495.2	32.8	0.6	32	531	-30	502	50	3010	0.9	0.78	**3.20**
2/20/20	15:00	3096.8		43.1				30	140	493.4	34.6	1.8	34	1864	-89	1775	59	3551	0.9	0.85	**3.65**
2/20/20	15:30	3104.8		43.0				30	170	492.1	35.9	1.32	35	1836	-65	1771	59	3543	0.9	0.80	**3.49**
2/20/20	16:00	3110.6		43.7				30	200	499.9	28.1	-7.8	32	1356	385	1741	58	3481	0.9	1.23	**3.76**
2/20/20	16:30	3117.2		43.6				30	230	499.0	29.0	0.96	29	1522	-47	1475	49	2950	0.9	0.92	**3.54**

template updated: 8/14/19

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

Leighton

Project:	10290.002
Exploration #/Location:	LI-3
Depth Boring drilled to (ft):	55
Tested by:	AK
USCS Soil Type in test zone:	SP-SM
Weather (start to finish):	Sunny
Liquid Used/pH:	H2O
Measured boring diameter:	8 in.
Approx Depth to GW below GS:	50 ft
Well Prep:	Drilled to 55', Pulled Augers, Caved to 44.5', Placed Gravel

4 in. Well Radius

Initial estimated Depth to Water Surface (in.): 497
Average depth of water in well, "h" (in.): 19
approx. h/r: 4.7
Tu (Fig. 8) (ft): 8.6
Tu>3h?: yes, OK

Cross-sectional area for vol calcs (in.^2): 49.3

	ft	in.	Total (in.)
Depth to Bot of well (or top of soil over Bentonite)	43. ft		516
Pilot Tube stickup (+ is above ground)	2. ft	9.25 in.	33.3
Depth to top of sand outside of casing from top of pilot tube	36. ft		
Depth to top of float assembly from top of pilot tube			0 -33.25 Depth below GS (in.)
Float Assembly ID		DHVA	
Float assembly Extension length (in.)			

Flow Meter:

Meter ID	3236
Meter Color	Black
Meter Units	Gallons
DL ID	

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of pilot tube)		Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3)			Flow (in^3/ min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
		Reading (cu-ft or gal)	Interval Pulse Count											from supply	from Δh	Total					
Start Date	Start time:	Gallons		ft	in.																
2/20/20	10:20	1867.4		42.9					0	481.0	35.1										
2/20/20	10:40	1883.2		43.3				20	20	486.5	29.5	-5.52	32	3643	272	3915	196	11746	0.9	3.76	**12.56**
2/20/20	10:50	1888.7		43.7				10	30	491.5	24.5	-5.04	27	1271	249	1519	152	9115	0.9	3.89	**11.52**
2/20/20	11:00	1895.2		43.1				10	40	484.2	31.8	7.32	28	1511	-361	1150	115	6898	0.921	1.78	**8.39**
2/20/20	11:10	1900.9		42.9				10	50	481.4	34.6	2.76	33	1321	-136	1185	119	7111	0.9	1.68	**7.41**
2/20/20	11:20	1905.4		43.1				10	60	484.1	31.9	-2.64	33	1021	130	1151	115	6907	0.9	1.92	**7.19**
2/20/20	11:30	1910.6		43.4				10	70	487.0	29.1	-2.88	30	1208	142	1350	135	8101	0.9	2.61	**9.15**
2/20/20	12:00	1923.4		43.4				30	100	487.2	28.8	-0.24	29	2961	12	2973	99	5947	0.9	1.90	**7.05**
2/20/20	12:30	1937.1		43.4				30	130	487.3	28.7	-0.12	29	3153	6	3159	105	6318	0.9	2.03	**7.54**
2/20/20	13:10	1954.8		43.7				40	170	491.4	24.6	-4.08	27	4091	201	4292	107	6438	0.9	2.70	**8.24**
2/20/20	14:10	1973.2		44.5				60	230	500.8	15.3	-9.36	20	4248	462	4710	78	4710	0.9	4.43	**7.88**
2/20/20	14:40	1982.0		44.5				30	260	500.8	15.3	0	15	2054	0	2054	68	4107	0.9	3.31	**8.73**

template updated: 8/14/19

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

Leighton

Project: 10290.002

Exploration #/Location: LI-4

Depth Boring drilled to (ft): 15

Tested by: AK

USCS Soil Type in test zone: SM

Weather (start to finish): Sunny

Liquid Used/pH: H2O

Measured boring diameter: 8 in. 4 in. Well Radius

Approx Depth to GW below GS: 50 ft

Well Prep: Caved to 13', Backfilled to 10', Gravel placed at bottom to 9'4"

Initial estimated Depth to Water Surface (in.): 71

Average depth of water in well, "h" (in.): 41

approx. h/r: 10.2

Tu (Fig. 8) (ft): 44.1

Tu>3h?: yes, OK

Cross-sectional area for vol calcs (in.^2): 20.1

	ft	in.	Total (in.)
Depth to Bot of well (or top of soil over Bentonite)	9. ft	4. in.	112
Pilot Tube stickup (+ is above ground)		11.5 in.	11.5
Depth to top of sand outside of casing from top of pilot tube			
Depth to top of float assembly from top of pilot tube			0 -11.5 Depth below GS (in.)
Float Assembly ID		DHVA	
Float assembly Extension length (in.)			

Flow Meter:

Meter ID:

Meter Color: Black

Meter Units: Gallons

DL ID:

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube) ft	Depth to WL in Boring (measured from top of pilot tube) in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3): from supply	Vol Change (in.^3): from Δh	Vol Change (in.^3): Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	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	Start time:	Gallons																			
2/21/20	9:03																				
2/21/20	9:03	1986.7		9.05					0	97.1	14.9										
2/21/20	9:14	1998.8		6.45				11	11	65.9	46.1	31.2	31	2795	-626	2169	197	11829	0.9	1.51	**13.35**
2/21/20	9:24	2009.9		5.8				10	21	58.1	53.9	7.8	50	2564	-157	2408	241	14445	0.9	1.68	**10.19**
2/21/20	9:35	2021.1		5.35				11	32	52.7	59.3	5.4	57	2587	-108	2479	225	13521	0.921	1.36	**8.46**
2/21/20	9:45	2031		5.1				10	42	49.7	62.3	3	61	2287	-60	2227	223	13360	0.9	1.25	**7.80**
2/21/20	9:55	2041.7		4.75				10	52	45.5	66.5	4.2	64	2472	-84	2387	239	14324	0.9	1.20	**7.91**
2/21/20	10:06	2054.8		4.3				11	63	40.1	71.9	5.4	69	3026	-108	2918	265	15915	0.9	1.17	**8.20**
2/21/20	10:16	2065.7		3.83				10	73	34.5	77.5	5.64	75	2518	-113	2405	240	14428	0.9	0.94	**6.90**
2/21/20	10:26	2074.3		3.6				10	83	31.7	80.3	2.76	79	1987	-55	1931	193	11587	0.9	0.72	**5.25**
2/21/20	10:36	2083.8		3.8				10	93	34.1	77.9	-2.4	79	2195	48	2243	224	13456	0.9	0.89	**6.09**
2/21/20	10:46	2091.5		3.6				10	103	31.7	80.3	2.4	79	1779	-48	1731	173	10383	0.9	0.64	**4.70**
2/21/20	10:56	2099.9		3.3				10	113	28.1	83.9	3.6	82	1940	-72	1868	187	11209	0.9	0.65	**4.89**
2/21/20	11:06	2107.8		3				10	123	24.5	87.5	3.6	86	1825	-72	1753	175	10516	0.9	0.57	**4.40**
2/21/20	11:16	2116.9		2.65				10	133	20.3	91.7	4.2	90	2102	-84	2018	202	12107	0.9	0.60	**4.85**
***Falling Head**																					
2/21/20	12:17			6.4					194	64.7	47.3										
2/21/20	12:27			7.3				10	204	76.1	35.9	-11.4	42	0	229	229	23	1373	0.9	0.33	**1.16**
2/21/20	12:38			7.7				11	215	80.9	31.1	-4.8	34	0	96	96	9	526	0.9	0.15	**0.54**
2/21/20	12:48			7.7				10	225	80.9	31.1	0	31	0	0	0	0	0	0.9	0.00	**0.00**
2/21/20	13:26			8.7				38	263	92.3	19.7	-11.4	25	0	229	229	6	361	0.9	0.23	**0.48**
2/21/20	13:36			8.7				10	273	93.4	18.6	-1.08	19	0	22	22	2	130	0.9	0.08	**0.23**
2/21/20	13:51			9.0				15	288	96.5	15.5	-3.12	17	0	63	63	4	251	0.9	0.21	**0.48**
2/21/20	14:01			9.0				10	298	96.5	15.5	0	16	0	0	0	0	0	0.9	0.00	**0.00**
2/21/20	14:11			9				10	308	96.5	15.5	0	16	0	0	0	0	0	0.9	0.00	**0.00**

template updated: 8/14/19

APPENDIX C

LABORATORY TEST RESULTS

Boring No.	LB-1	LB-1	LB-3	LB-3				
Sample No.	S-7	S-8	S-4	S-8				
Depth (ft.)	30.0	35.0	15.0	35.0				
Sample Type	SPT	SPT	SPT	SPT				
Visual Soil Classification	SM	(ML)s	SM	(ML)s				
Moisture Correction								
Wet Weight of Soil + Container (gm.)	937.5	946.3	946.6	948.5				
Dry Weight of Soil + Container (gm.)	886.8	821.7	849.9	832.2				
Weight of Container (gm)	420.8	419.7	420.5	418.5				
Moisture Content (%)	10.9	31.0	22.5	28.1				
Container No.:	SN	NK	ML	35				
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	937.5	946.3	946.6	948.5				
Weight of Container (gm.)	420.8	419.7	420.5	418.5				
Weight of Dry Sample (gm.)	466.0	402.0	429.4	413.7				
Container No.:	SN	NK	ML	35				
After Wash								
Dry Weight of Sample + Container (gm)	700.2	518.8	655.7	538.2				
Weight of Container (gm)	420.8	419.7	420.5	418.5				
Dry Weight of Sample (gm)	279.4	99.1	235.2	119.7				
% Passing No. 200 Sieve	**40**	**75**	**45**	**71**				
% Retained No. 200 Sieve	60	25	55	29				

PERCENT PASSING No. 200 SIEVE
ASTM D 1140

Leighton

Project Name: STRATHAM EASTVALE SUMNER & SCHLEISMAN

Project No.: 10290.001

Client Name: **

Tested By: MRV / JAP Date: 5/10/13

Rev. 08-04

Leighton

One-Dimensional Swell or Settlement Potential of Cohesive Soils

(ASTM D 4546) -- Method 'B'

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN	Tested By:	JAP	Date:	5/14/13
Project No.:	10290.001	Checked By:	JMB	Date:	5/16/13
Boring No.:	LB-2	Sample Type:	IN SITU		
Sample No.:	R-4	Depth (ft.)	15.0		

Sample Description: SANDY SILT s(ML), olive brown.

Source and Type of Water Used for Inundation: Arrowhead (Distilled)

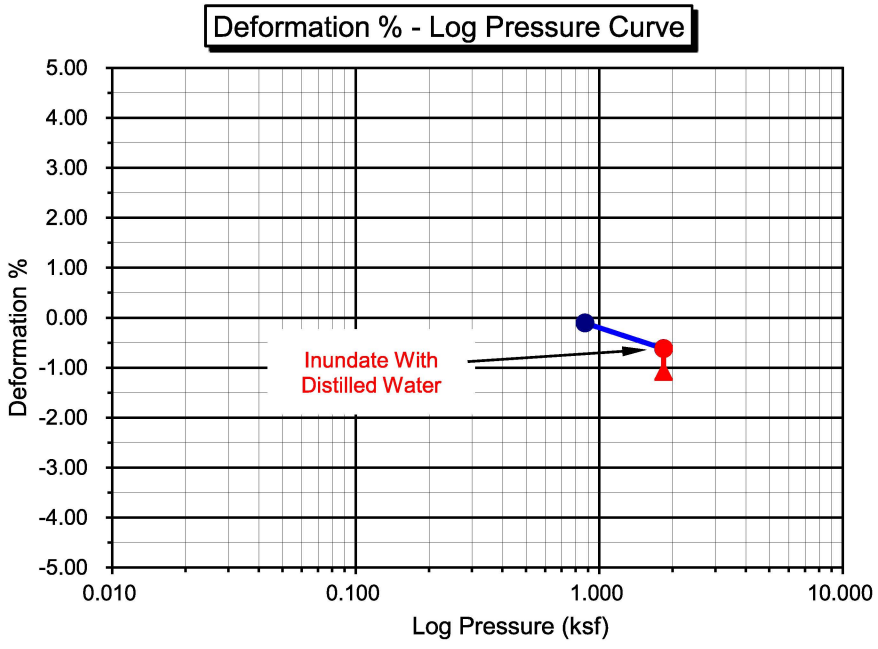
** Note: Loading After Wetting (Inundation) not Performed Using this Test Method.

Initial Dry Density (pcf):	107.8
Initial Moisture (%):	14.0
Initial Height (in.):	0.9980
Initial Dial Reading (in):	0.0500
Inside Diameter of Ring (in):	2.422

Final Dry Density (pcf):	108.9
Final Moisture (%) :	18.3
Initial Void ratio:	0.5630
Specific Gravity (assumed):	2.70
Initial Degree of Saturation (%):	67.0

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.875	0.0531	0.9969	0.00	-0.11	0.5612	-0.11
1.838	0.0582	0.9918	0.00	-0.62	0.5533	-0.62
H2O	0.0628	0.9872	0.00	-1.08	0.5460	-1.08

Percent Swell / Settlement After Inundation = -0.46



Calculation

Sample Diameter(in):	2.422

Before Test

Sample Thickness(in.):	0.998
Weight of Sample + ring(gm):	191.40
Weight of Ring(gm):	43.48
Height after consolidation(in.):	0.9852

After Test

Weight of Wet Sample+Ring (gm):	197.04
Weight of Dry Sample+Ring (gm):	173.26
Weight of Ring (gm):	43.5
Vertical Rdg. (in): Initial	0.0500
Vertical Rdg. (in): Final	0.0628
Specific Gravity (Assumed):	2.70
Water Density (pcf) -- target:	62.43
Wet density (pcf)	122.6

Final Wet Density (pcf)	128.9
Final Dry Density (pcf)	108.9

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN
Project No.:	10290.001
Boring No.:	LB-2
Sample No.:	R-4
Depth (ft.):	15.0
Date:	5/14/13
Test By:	JAP
Sample Desc.:	SANDY SILT s(ML), olive brown.
Date Sampled:	5/6/2013

Rev. 01-10

SIEVE ANALYSIS (POST TEST) -- OVERSIZE MATERIAL

DRY MASS OF SOIL: 129.78g

SIZE	MASS RET.	% RET.
1 1/2"	0.0	0.0
1"	0.0	0.0
3/4"	0.0	0.0
1/2"	0.0	0.0
3/8"	0.0	0.0
#4	0.0	0.0

SPECIMEN CRITICAL DIMENSIONS (PRE TEST)

HEIGHT MEASUREMENTS	INSIDE DIAMETER OF RING
1) 1.000"	1) 2.414"
2) 0.999"	2) 2.431"
3) 0.998"	3) 2.408"
4) 0.993"	4) 2.435"
AVG: 0.998"	AVG: 2.422"

** NOTES: A) SAMPLE PRESSURES ASSIGNED BY PROJECT ENGINEER (TYPICALLY AT OVERBURDEN STRESS).
B) DEVIATIONS FROM TEST PROCEDURE (Y / N) ? IF YES, EXPLANATION LISTED BELOW.

DEVIATION SPECIFICS: **

Leighton

One-Dimensional Swell or Settlement Potential of Cohesive Soils

(ASTM D 4546) -- Method 'B'

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN	Tested By:	JAP	Date:	5/14/13
Project No.:	10290.001	Checked By:	JMB	Date:	5/16/13
Boring No.:	LB-5	Sample Type:	IN SITU		
Sample No.:	R-3	Depth (ft.)	10.0		

Sample Description: SANDY SILT s(ML), olive.

Source and Type of Water Used for Inundation: Arrowhead (Distilled)

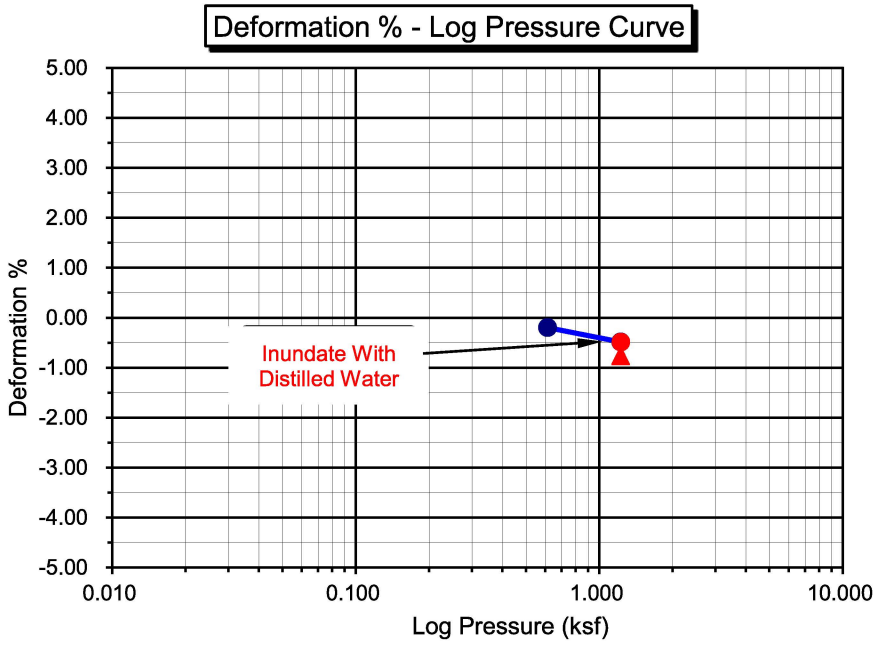
** Note: Loading After Wetting (Inundation) not Performed Using this Test Method.

Initial Dry Density (pcf):	108.3
Initial Moisture (%):	15.3
Initial Height (in.):	0.9990
Initial Dial Reading (in):	0.0500
Inside Diameter of Ring (in):	2.423

Final Dry Density (pcf):	108.7
Final Moisture (%) :	18.5
Initial Void ratio:	0.5562
Specific Gravity (assumed):	2.70
Initial Degree of Saturation (%):	74.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.613	0.0530	0.9970	0.00	-0.20	0.5531	-0.20
1.226	0.0559	0.9941	0.00	-0.49	0.5486	-0.49
H2O	0.0587	0.9913	0.00	-0.77	0.5443	-0.77

Percent Swell / Settlement After Inundation = -0.28



Calculation

Sample Diameter(in):	2.423

Before Test

Sample Thickness(in.):	0.999
Weight of Sample + ring(gm):	194.23
Weight of Ring(gm):	43.96
Height after consolidation(in.):	0.9903

After Test

Weight of Wet Sample+Ring (gm):	198.36
Weight of Dry Sample+Ring (gm):	174.30
Weight of Ring (gm):	44.0
Vertical Rdg. (in): Initial	0.0500
Vertical Rdg. (in): Final	0.0587
Specific Gravity (Assumed):	2.70
Water Density (pcf) -- target:	62.43
Wet density (pcf)	124.3

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN
Project No.:	10290.001
Boring No.:	LB-5
Sample No.:	R-3
Depth (ft.):	10.0
Date:	5/14/13
Test By:	JAP
Sample Desc.:	SANDY SILT s(ML), olive.
Date Sampled:	5/6/2013

Final Wet Density (pcf)	128.8
Final Dry Density (pcf)	108.7

Rev. 01-10

SIEVE ANALYSIS (POST TEST) -- OVERSIZE MATERIAL

DRY MASS OF SOIL: 130.34g

SIZE	MASS RET.	% RET.
1 1/2"	0.0	0.0
1"	0.0	0.0
3/4"	0.0	0.0
1/2"	0.0	0.0
3/8"	0.0	0.0
#4	0.0	0.0

SPECIMEN CRITICAL DIMENSIONS (PRE TEST)

HEIGHT MEASUREMENTS	INSIDE DIAMETER OF RING
1) 1.000"	1) 2.427"
2) 0.998"	2) 2.430"
3) 0.999"	3) 2.424"
4) 0.997"	4) 2.411"
AVG: 0.999"	AVG: 2.423"

** NOTES: A) SAMPLE PRESSURES ASSIGNED BY PROJECT ENGINEER (TYPICALLY AT OVERBURDEN STRESS).
B) DEVIATIONS FROM TEST PROCEDURE (Y / N) ? IF YES, EXPLANATION LISTED BELOW.

DEVIATION SPECIFICS: **

Leighton

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN	Tested By :	JMB / RTS	Date:	5-10-13
Project No.:	10290.001	Input By :	JMB	Date:	5-13-13
Exploration No.:	**LB-2**	Depth (ft.)	0-5.0		
Sample No :	**B-2**				
Soil Identification:	SILTY SAND WITH LITTLE GRAVEL (SM)g, olive brown.				

Preparation Method:	X	Moist	Scalp Fraction (%)		Rammer Weight (lb.) = 10.0
		Dry	#3/4	6.6	Height of Drop (in.) = 18.0
Compaction Method		Mechanical Ram	#3/8		
	X	Manual Ram	#4		Mold Volume (ft³) 0.07480
			Spec. Gravity	2.62	

Moisture Added (ml)	0	110	-110	-220		
TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	10582	10428	10354	10075		
Weight of Mold (g)	5679	5679	5679	5679		AS-REC
Net Weight of Soil (g)	4903	4749	4675	4396		M/C
Wet Weight of Soil + Cont. (g)	2312.5	1493.7	1999.9	1689.7		1130.8
Dry Weight of Soil + Cont. (g)	2144.2	1362.3	1900.2	1624.0		1059.4
Weight of Container (g)	230.9	136.4	321.6	200.2		217.6
Moisture Content (%)	8.8	10.7	6.3	4.6		8.5
Wet Density (pcf)	144.5	140.0	137.8	129.6		
Dry Density (pcf)	132.8	126.4	129.6	123.8		

Maximum Dry Density (pcf)	**133.0**	**Optimum Moisture Content (%)**	**8.5**
Corrected Dry Density (pcf)	**134.5**	**Corrected Moisture Content (%)**	**8.0**

☐ **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

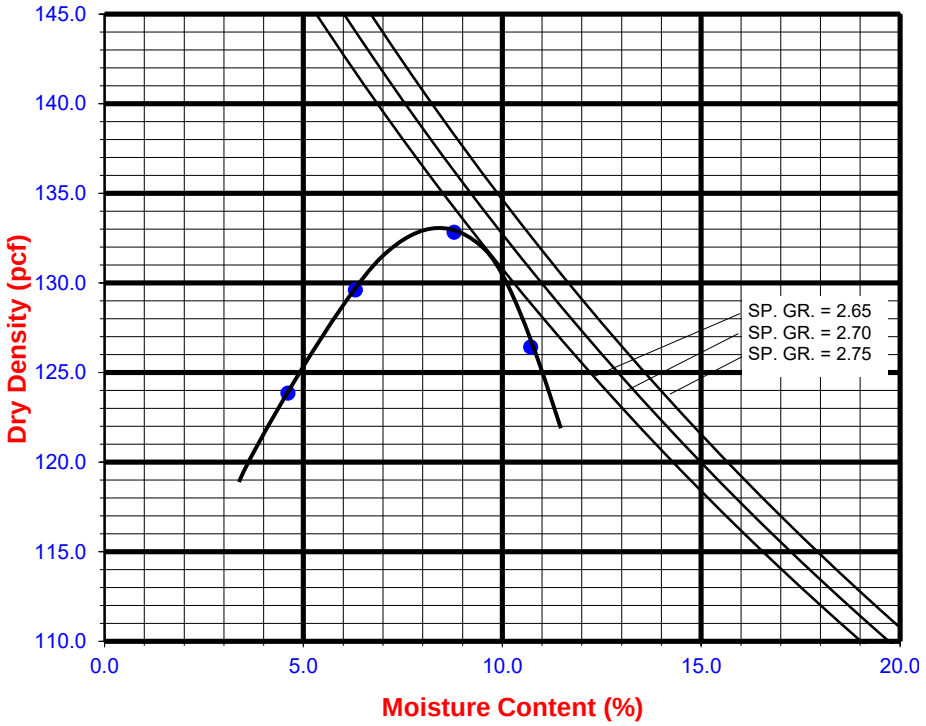
☒ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Leighton

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: STRATHAM EASTVALE SUMNER & SCHLEISMAN Tested By : MRV Date: 5/17/13

Project No. : 10290.001 Data Input By: JMB Date: 5/17/13

Boring No.	**LB-3**				
Sample No.	**B-1**				
Sample Depth (ft)	0-5.0				
Visual Soil Classification	SM				
Wet Weight of Soil + Container (g)	86.89				
Dry Weight of Soil + Container (g)	86.60				
Weight of Container (g)	37.15				
Moisture Content (%)	0.6				
Weight of Soaked Soil (g)	100.0				

SULFATE CONTENT, Hach Kit Method

Dillution : 1	3				
Water Fraction (ml)	25				
Tube Reading	70				
PPM Sulfate	210				
% Sulfate	0.0210				

CHLORIDE CONTENT, AASHTO T-291

ml of Chloride Soln. For Titration (B)	25				
ml of AgNO3 Soln. Used in Titration (C)	1				
PPM of Chloride (C -0.2) * Titre (1) * 1000 / 10g	24				
PPM of Chloride, Dry Wt. Basis	**24**				

pH TEST, ASTM D-4972

Container No.	A				
Temperature (C°)	23.3				
pH Value (METHOD A)	**7.49**				

Leighton

SOIL RESISTIVITY TEST
ASTM G-187

Project Name: STRATHAM EASTVALE SUMNER & SCHLEISMAN Tested By : MRV Date: 5/17/13

Project No. : 10290.001 Data Input By: JMB Date: 5/17/13

Boring No.: **LB-3** Checked By: JMB Date: 5/17/13

Sample No. : **B-1** Depth (ft.) : 0-5.0

Visual Soil Identification: SM

** NOTE: ASTM G-187 REQUIRES SOIL SPECIMENS TO PASS THROUGH NO.8 SIEVE PRIOR TO TESTING. THEREFORE, THIS TEST METHOD MAY NOT BE REPRESENTATIVE FOR COARSER MATERIALS.

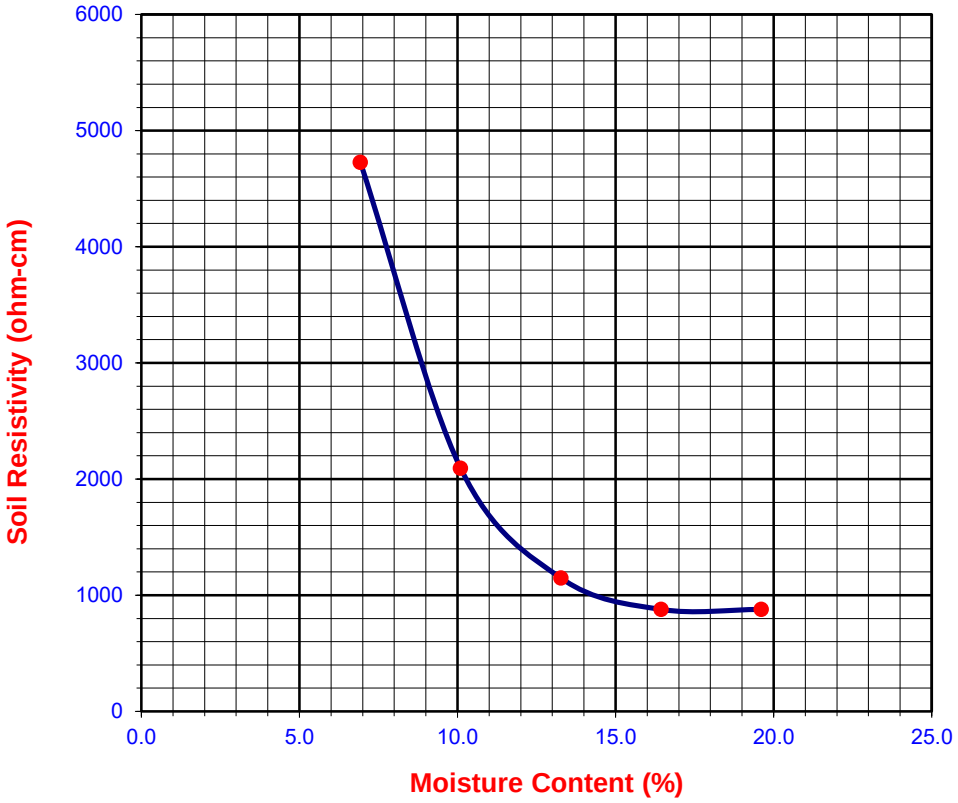
Initial Moisture Content (%)

Wet Wt. of Soil + Cont. (g)	86.9
Dry Wt. of Soil + Cont. (g)	86.6
Wt. of Container (g)	37.2
Moisture Content (%) (MCi)	0.59

Initial Soil Weight (gm)(Wt)	1586.0
Box Constant:	6.7522

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Remolded Specimen	Moisture Adjustments				
Water Added (ml) (Wa)	100	150	200	250	300
Adj. Moisture Content (%) (MC)	6.93	10.10	13.27	16.44	19.61
Resistance Rdg. (ohm)	700	310	170	130	130
Soil Resistivity (ohm-cm)	4727	2093	1148	878	878



Minimum Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content ppm / %	Chloride Content (ppm)	Soil pH
ASTM G-187, D-2216		HACH KIT METHOD	AASHTO T-291	ASTM D-4972
878	16.44	**210** 0.0210	**24**	7.49

Rev. 11-04

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	STRATHAM EASTVALE SUMNER & SCHLEISMAN	Tested By:	MRV	Date:	5/16/13
Project No. :	10290.001	Checked By:	JMB	Date:	5/17/13
Boring No.:	**LB-3**	Depth (ft.)	0-5.0		
Sample No. :	**B-1**	Location:	**		
Sample Description:	SILTY SAND WITH TRACE GRAVEL (SM), brown.				

Dry Wt. of Soil + Cont. (gm.)	3209.5
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	3209.5
Weight Soil Retained on #4 Sieve	66.4
Percent Passing # 4	97.9

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0045
Wt. Comp. Soil + Mold (gm.)	610.5	629.7
Wt. of Mold (gm.)	190.7	190.7
Specific Gravity (Assumed)	2.70	2.70
Container No.	4	4
Wet Wt. of Soil + Cont. (gm.)	337.3	629.7
Dry Wt. of Soil + Cont. (gm.)	313.8	386.9
Wt. of Container (gm.)	37.3	190.7
Moisture Content (%)	8.5	13.5
Wet Density (pcf)	126.6	132.3
Dry Density (pcf)	116.7	116.6
Void Ratio	0.444	0.451
Total Porosity	0.308	0.311
Pore Volume (cc)	63.7	64.6
Degree of Saturation (%) [S meas]	**51.6**	**80.6**

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
5/16/13	10:45	1.0	0	0.5000
5/16/13	10:55	1.0	10	0.4997
	Add Distilled Water to the Specimen			
5/17/13	5:45	1.0	1130	0.5045
5/17/13	6:45	1.0	1190	0.5045

Expansion Index (EI meas) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	4.8
Expansion Index (Report) = Nearest Whole Number or Zero (0) if Initial Height is > than Final Height	**5**

APPENDIX D

SUMMARY OF SECONDARY SEISMIC HAZARD ANALYSIS

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.96

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

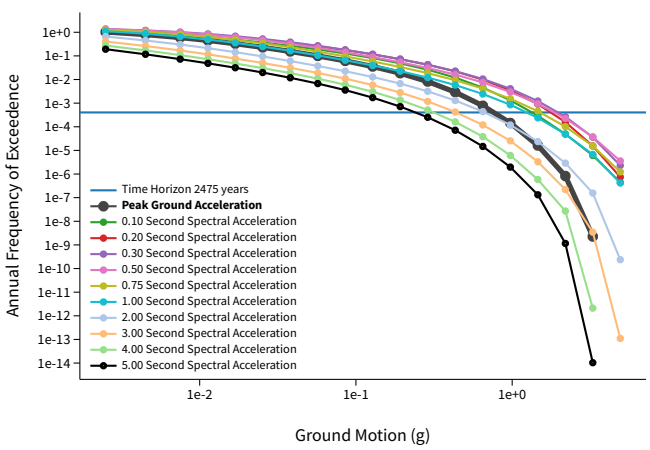
-117.5747

Site Class

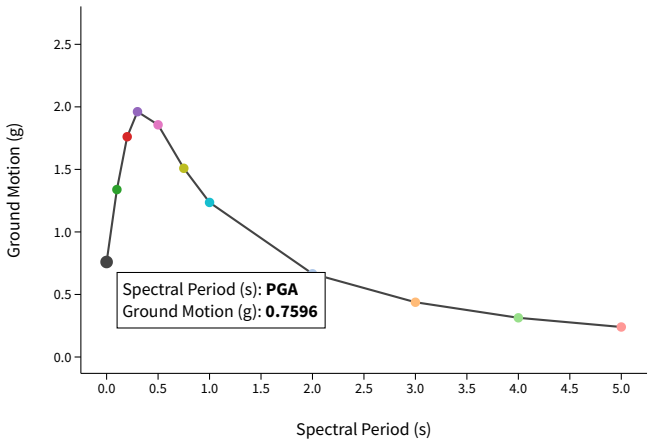
259 m/s (Site class D)

Hazard Curve

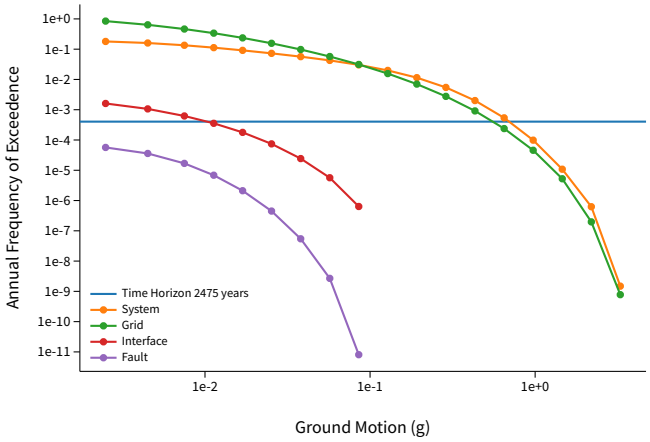
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

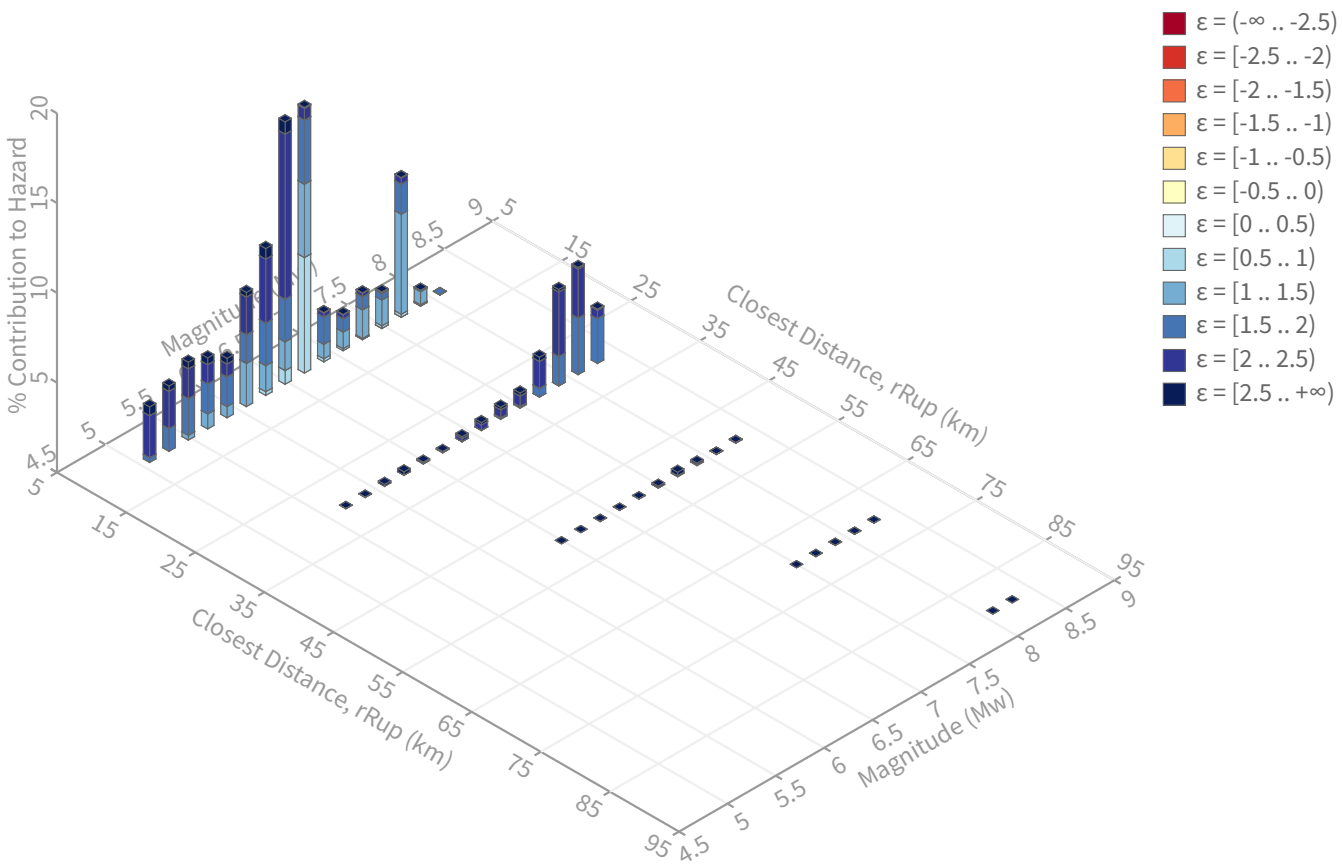


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.75964258 g

Recovered targets

Return period: 3037.0727 yrs
Exceedance rate: 0.00032926443 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.74
r: 14.06 km
ε_0: 1.79 σ

Mode (largest m-r bin)

m: 6.66
r: 5.79 km
ε_0: 1.27 σ
Contribution: 14.79 %

Mode (largest m-r-ε_0 bin)

m: 6.47
r: 14.63 km
ε_0: 2.23 σ
Contribution: 9.2 %

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 .. +∞]

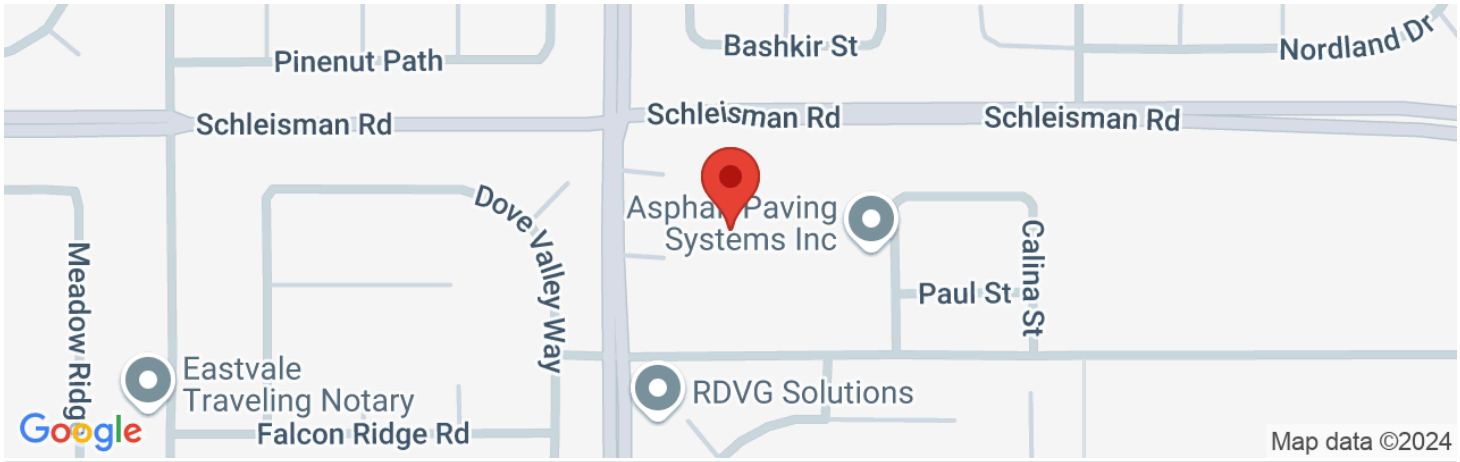
Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							36.19
Fontana (Seismicity) [2]		3.71	6.61	1.07	117.587°W	33.988°N	340.36	6.24
Whittier alt 1 [0]		12.57	7.43	1.39	117.633°W	33.854°N	204.69	6.13
Elsinore (Glen Ivy) rev [0]		14.71	6.51	2.23	117.590°W	33.829°N	185.54	6.11
Chino alt 1 [3]		9.74	6.80	1.68	117.655°W	33.904°N	229.85	4.83
San Jacinto (San Bernardino) [3]		27.92	8.09	1.94	117.328°W	34.106°N	54.47	3.19
San Andreas (San Bernardino N) [3]		34.80	8.00	2.19	117.378°W	34.227°N	31.38	2.98
UC33brAvg_FM32	System							33.08
Elsinore (Glen Ivy) rev [0]		14.71	6.50	2.24	117.590°W	33.829°N	185.54	6.23
Whittier alt 2 [0]		12.93	7.55	1.39	117.633°W	33.853°N	204.25	5.82
Fontana (Seismicity) [2]		3.71	6.61	1.07	117.587°W	33.988°N	340.36	5.11
Chino alt 2 [2]		9.72	6.93	1.62	117.658°W	33.908°N	233.02	3.99
San Jacinto (San Bernardino) [3]		27.92	8.08	1.95	117.328°W	34.106°N	54.47	3.13
San Andreas (San Bernardino N) [3]		34.80	8.00	2.19	117.378°W	34.227°N	31.38	3.04
Chino alt 2 [1]		9.92	6.35	1.91	117.670°W	33.921°N	243.54	1.13
UC33brAvg_FM31 (opt)	Grid							15.58
PointSourceFinite: -117.575, 34.009		7.24	5.76	1.62	117.575°W	34.009°N	0.00	2.95
PointSourceFinite: -117.575, 34.009		7.24	5.76	1.62	117.575°W	34.009°N	0.00	2.95
PointSourceFinite: -117.575, 34.027		8.44	5.85	1.76	117.575°W	34.027°N	0.00	1.44
PointSourceFinite: -117.575, 34.027		8.44	5.85	1.76	117.575°W	34.027°N	0.00	1.44
PointSourceFinite: -117.575, 34.036		9.43	5.75	1.92	117.575°W	34.036°N	0.00	1.20
PointSourceFinite: -117.575, 34.036		9.43	5.75	1.92	117.575°W	34.036°N	0.00	1.20
UC33brAvg_FM32 (opt)	Grid							15.15
PointSourceFinite: -117.575, 34.009		7.25	5.75	1.62	117.575°W	34.009°N	0.00	2.89
PointSourceFinite: -117.575, 34.009		7.25	5.75	1.62	117.575°W	34.009°N	0.00	2.89
PointSourceFinite: -117.575, 34.027		8.64	5.76	1.82	117.575°W	34.027°N	0.00	1.32
PointSourceFinite: -117.575, 34.027		8.64	5.76	1.82	117.575°W	34.027°N	0.00	1.32
PointSourceFinite: -117.575, 34.036		9.43	5.75	1.92	117.575°W	34.036°N	0.00	1.20
PointSourceFinite: -117.575, 34.036		9.43	5.75	1.92	117.575°W	34.036°N	0.00	1.20

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 33.96, -117.5747



Date	11/15/2024, 10:37:53 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.656	MCE_R ground motion. (for 0.2 second period)
S_1	0.593	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.656	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.104	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.682	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.75	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.656	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.763	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.87	Factored deterministic acceleration value. (0.2 second)
S1RT	0.593	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.646	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.617	Factored deterministic acceleration value. (1.0 second)
PGAd	0.763	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.682	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.939	Mapped value of the risk coefficient at short periods
C_{R1}	0.918	Mapped value of the risk coefficient at a period of 1 s
C_V	1.431	Vertical coefficient

DISCLAIMER

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Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: Stratham Eastvale ; Case 1; PGAm 0.75; design GW 8; No overex 0

Project No.: 30332

Nov 2024

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	50	8		0		-8
LB-3	50	8		0		-8
LI-2	50	8		0		-8
LI-3	50	8		0		-8
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates	
X (ft)	Y (ft)

General Parameters:

a_{max} = 0.75g

M_W = 6.7

MSF eq: 1

MSF = 1.33

Hammer Efficiency = 84

C_E = 1.40

C_B = 1

C_S for SPT? TRUE

Unlined, but room for liner

Rod Stickup (feet) = 3

Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Project: Stratham Eastvale ; Case 1; PGAm 0.75; design GW 8; No overex 0

Project No.: 30332

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8		30	120	10	2	1	6.5	300	11.6	18.1	0.193	300	0.48	0.36	**NonLiq**	18.1	0.29		0.13	1.2
LB-1	3.8 to 7.5	5	3.8		40	120	20	2	1	13.0	600	23.2	32.8	>Range	600	0.48	0.36	**NonLiq**	32.8	0.23		0.10	1.1
LB-1	7.5 to 8.0	10	0.5		20	120	78	2	1	50.7	1200	79.6	89.5	>Range	1075.2	0.53	0.40	**NonLiq**	89.5	0.01		0.00	1.0
LB-1	8.0 to 12.5	10	4.5		20	120	78	2	1	50.7	1200	79.6	89.5	>Range	1075.2	0.53	0.40	**NonLiq**	89.5			0.00	1.0
LB-1	12.5 to 17.5	15	5.0		65	120	10	1	1.15	11.5	1800	14.7	22.6	0.251	1363.2	0.62	0.47	**0.54**	18.7		1.65	0.99	1.0
LB-1	17.5 to 22.5	20	5.0		40	120	37	1	1.3	48.1	2400	59.7	76.6	>Range	1651.2	0.68	0.51	**NonLiq**	76.6			0.00	0.0
LB-1	22.5 to 27.5	25	5.0		30	120	51	1	1.3	66.3	3000	73.6	89.6	>Range	1939.2	0.71	0.53	**NonLiq**	89.6			0.00	0.0
LB-1	27.5 to 32.5	30	5.0		**40**	120	29	1	1.3	37.7	3600	40.2	53.2	>Range	2227.2	0.73	0.55	**NonLiq**	53.2			0.00	0.0
LB-1	32.5 to 37.5	35	5.0		**75**	120	19	1	1.23	23.4	4200	23.1	32.7	>Range	2515.2	0.72	0.54	**NonLiq**	32.7			0.00	0.0
LB-1	37.5 to 42.5	40	5.0		5	120	100	1	1.3	130.0	4800	120.0	120.0	>Range	2803.2	0.71	0.53	**NonLiq**	120.0			0.00	0.0
LB-1	42.5 to 47.5	45	5.0		5	120	100	1	1.3	130.0	5400	113.2	113.2	>Range	3091.2	0.69	0.52	**NonLiq**	113.2			0.00	0.0
LB-1	47.5 to 52.0	50	4.5		5	120	100	1	1.3	130.0	6000	107.4	107.4	>Range	3379.2	0.66	0.50	**NonLiq**	107.4			0.00	0.0
LB-3	0 to 3.8	2.5	3.8		65	120	14	2	1	9.1	300	16.2	24.5	0.282	300	0.48	0.36	**NonLiq**	24.5	0.12		0.05	2.8
LB-3	3.8 to 7.5	5	3.8		10	120	13	2	1	8.5	600	15.1	16.3	0.173	600	0.48	0.36	**NonLiq**	16.3	1.11		0.50	2.7
LB-3	7.5 to 8.0	10	0.5		65	120	76	2	1	49.4	1200	77.5	98.1	>Range	1075.2	0.53	0.40	**NonLiq**	98.1	0.01		0.00	2.2
LB-3	8.0 to 12.5	10	4.5		65	120	76	2	1	49.4	1200	77.5	98.1	>Range	1075.2	0.53	0.40	**NonLiq**	98.1			0.00	2.2
LB-3	12.5 to 17.5	15	5.0		**45**	120	8	1	1.11	8.9	1800	11.4	18.7	0.200	1363.2	0.62	0.47	**0.43**	13.4		2.09	1.25	2.2
LB-3	17.5 to 22.5	20	5.0		40	120	50	1	1.3	65.0	2400	80.6	101.8	>Range	1651.2	0.68	0.51	**NonLiq**	101.8			0.00	1.0
LB-3	22.5 to 27.5	25	5.0		40	120	40	1	1.3	52.0	3000	57.7	74.2	>Range	1939.2	0.71	0.53	**NonLiq**	74.2			0.00	1.0
LB-3	27.5 to 32.5	30	5.0		70	120	49	1	1.3	63.7	3600	67.9	86.5	>Range	2227.2	0.73	0.55	**NonLiq**	86.5			0.00	1.0
LB-3	32.5 to 37.5	35	5.0		**71**	120	13	1	1.15	14.9	4200	14.7	22.7	0.252	2515.2	0.72	0.54	**0.46**	18.7		1.65	0.99	1.0
LB-3	37.5 to 42.5	40	5.0		25	120	26	1	1.3	33.8	4800	31.2	39.1	>Range	2803.2	0.71	0.53	**NonLiq**	39.1			0.00	0.0
LB-3	42.5 to 47.5	45	5.0		5	120	100	1	1.3	130.0	5400	113.2	113.2	>Range	3091.2	0.69	0.52	**NonLiq**	113.2			0.00	0.0
LB-3	47.5 to 52.0	50	4.5		5	120	47	1	1.3	61.1	6000	50.5	50.5	>Range	3379.2	0.66	0.50	**NonLiq**	50.5			0.00	0.0
LI-2	0 to 3.8	2.5	3.8		20	120	27	2	1	17.6	300	31.3	37.4	>Range	300	0.48	0.36	**NonLiq**	37.4	0.04		0.02	0.4
LI-2	3.8 to 6.3	5	2.5		25	120	5	1	1.1	5.5	600	9.8	15.2	0.162	600	0.48	0.36	**NonLiq**	15.2	1.15		0.35	0.3
LI-2	6.3 to 8.0	7.5	1.8		**17**	120	100	2	1	65.0	900	110.9	120.6	>Range	900	0.48	0.36	**NonLiq**	120.6	0.01		0.00	0.0
LI-2	8.0 to 8.8	7.5	0.8		**17**	120	100	2	1	65.0	900	110.9	120.6	>Range	900	0.48	0.36	**NonLiq**	120.6			0.00	0.0
LI-2	8.8 to 11.3	10	2.5		65	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1075.2	0.53	0.40	**NonLiq**	127.4			0.00	0.0
LI-2	11.3 to 13.8	12.5	2.5		65	120	73	2	1	47.5	1500	66.6	84.9	>Range	1219.2	0.58	0.44	**NonLiq**	84.9			0.00	0.0
LI-2	13.8 to 17.5	15	3.8		30	120	21	1	1.3	27.3	1800	35.0	45.1	>Range	1363.2	0.62	0.47	**NonLiq**	45.1			0.00	0.0
LI-2	17.5 to 27.5	20	10.0		65	120	77	2	1	50.1	2400	62.1	79.5	>Range	1651.2	0.68	0.51	**NonLiq**	79.5			0.00	0.0
LI-2	27.5 to 37.5	35	10.0		**58**	120	27	1	1.3	35.1	4200	34.6	46.6	>Range	2515.2	0.72	0.54	**NonLiq**	46.6			0.00	0.0
LI-2	37.5 to 42.5	40	5.0		**7**	120	100	2	1	65.0	4800	60.0	60.7	>Range	2803.2	0.71	0.53	**NonLiq**	60.7			0.00	0.0
LI-2	42.5 to 47.5	45	5.0		10	120	100	1	1.3	130.0	5400	113.2	116.5	>Range	3091.2	0.69	0.52	**NonLiq**	116.5			0.00	0.0

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LI-2	47.5 to 52.5	50	5.0		5	120	100	2	1	65.0	6000	53.7	53.7	>Range	3379.2	0.66	0.50	**NonLiq**	53.7			0.00	0.0
LI-2	52.5 to 57.0	55	4.5		10	120	41	1	1.3	53.3	6288	43.0	44.8	>Range	3667.2	0.64	0.48	**NonLiq**	44.8			0.00	0.0
LI-3	0 to 3.8	2.5	3.8		20	120	11	2	1	7.2	300	12.8	17.4	0.185	300	0.48	0.36	**NonLiq**	17.4	0.31		0.14	0.5
LI-3	3.8 to 6.3	5	2.5		25	120	5	1	1.1	5.5	600	9.8	15.2	0.162	600	0.48	0.36	**NonLiq**	15.2	1.15		0.35	0.3
LI-3	6.3 to 8.0	7.5	1.8		**18**	120	87	2	1	56.6	900	96.5	106.1	>Range	900	0.48	0.36	**NonLiq**	106.1	0.01		0.00	0.0
LI-3	8.0 to 8.8	7.5	0.8		**18**	120	87	2	1	56.6	900	96.5	106.1	>Range	900	0.48	0.36	**NonLiq**	106.1			0.00	0.0
LI-3	8.8 to 11.3	10	2.5		20	120	78	1	1.3	101.4	1200	159.2	175.4	>Range	1075.2	0.53	0.40	**NonLiq**	175.4			0.00	0.0
LI-3	11.3 to 13.8	12.5	2.5		**58**	120	44	2	1	28.6	1500	40.2	53.2	>Range	1219.2	0.58	0.44	**NonLiq**	53.2			0.00	0.0
LI-3	13.8 to 17.5	15	3.8		60	120	20	1	1.3	26.0	1800	33.3	45.0	>Range	1363.2	0.62	0.47	**NonLiq**	45.0			0.00	0.0
LI-3	17.5 to 22.5	20	5.0		55	120	100	2	1	65.0	2400	80.6	101.8	>Range	1651.2	0.68	0.51	**NonLiq**	101.8			0.00	0.0
LI-3	22.5 to 27.5	25	5.0		**22**	120	56	1	1.3	72.8	3000	80.8	92.2	>Range	1939.2	0.71	0.53	**NonLiq**	92.2			0.00	0.0
LI-3	27.5 to 32.5	30	5.0		40	120	100	2	1	65.0	3600	69.3	88.2	>Range	2227.2	0.73	0.55	**NonLiq**	88.2			0.00	0.0
LI-3	32.5 to 37.5	35	5.0		10	120	100	1	1.3	130.0	4200	128.3	132.0	>Range	2515.2	0.72	0.54	**NonLiq**	132.0			0.00	0.0
LI-3	37.5 to 42.5	40	5.0		**3**	120	100	2	1	65.0	4800	60.0	60.0	>Range	2803.2	0.71	0.53	**NonLiq**	60.0			0.00	0.0
LI-3	42.5 to 47.5	45	5.0		5	120	100	1	1.3	130.0	5400	113.2	113.2	>Range	3091.2	0.69	0.52	**NonLiq**	113.2			0.00	0.0
LI-3	47.5 to 52.5	50	5.0		5	120	100	2	1	65.0	6000	53.7	53.7	>Range	3379.2	0.66	0.50	**NonLiq**	53.7			0.00	0.0
LI-3	52.5 to 57.0	55	4.5		5	120	100	1	1.3	130.0	6288	104.9	104.9	>Range	3667.2	0.64	0.48	**NonLiq**	104.9			0.00	0.0

Surface Manifestations of Liquefaction and Liquefaction Bearing Capacity Analysis

Stratham Eastvale ; Case 1; PGAm 0.75; design GW 8; No overex 0

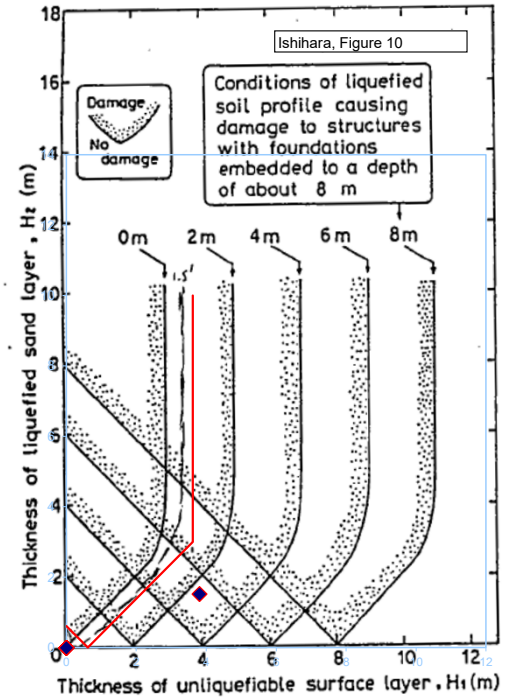
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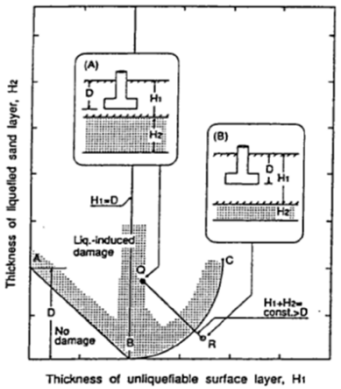
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Boring No.	New Fill (raise grade) (ft)	Footing Depth (ft)	Ishihara, 1995, Surface Manifestisations of Liquefaction Analysis: Bot. Depth of Nonliq and Liq Layers Z1 (non) (ft)	Za (liq) (ft)	Zb (non) (ft)	Zc (liq) (ft)	Thickness H1 (ft)	H2 (ft)	Thickness H1 (m)	H2 (m)	Struct Damage/ Surface Manifestations? (Ishihara, 1995)	Amount of New Fill needed to mitigate (ft)	Or, Amount of Overex. needed to mitigate (ft)
LB-1	0	2	12.5	17.5	52.0		12.5	5.0	3.8	1.5	**no**	0	0
LB-3	0	2	12.5	17.5	32.5	37.5	12.5	5.0	3.8	1.5	**no**	0	0
LI-2	0	2	57.0				57.0		17.4		**no**	0	0
LI-3	0	2	57.0				57.0		17.4		**no**	0	0

Karamitros et al., 2013, Liqufaction Bearing Capacity: Assumed maximum Footing Width Square ftg (ft)	Strip ftg (ft)	In order to achieve critical thickness of Non-liquefiable upper clay crust (where additional thickness does not further increase FSliq of bearing capacity): Amount of New Fill Needed (ft) square ftg	Strip ftg	Or, Amount of Overex. Needed (ft) square ftg	Strip ftg	Juang (2005) based on Iwasaki (1982), as presented in Tonkin & Taylor (2013), Liquefaction Potential Index (LPI): LPI = Sum[F1*W(z)*dz]	Risk of Liquefaction Damage Based on LPI
10	4	4.9	0.0	17.4	0.0	6.9	High
10	4	4.9	0.0	17.4	0.0	12.4	High
10	4	0.0	0.0	0.0	0.0	0.0	Very Low
10	4	0.0	0.0	0.0	0.0	0.0	Very Low



Footing Depth= 2 ft
0.61 m



LPI range:	Liquefaction Risk:
LPI=0	Very low
0<LPI<=5	Low
5<LPI<=15	High
LPI>15	Very High

References:

Ishihara, K., 1995, Effects of At-Depth Liquefaction on Embedded Foundations During Earthquakes, Proceedings of 11th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1995.

Iwasaki, T., Arakawa, T., and Tokida, K., 1982, Simplifed Procedures for Assessing Soil Liquefaction During Earthquakes Proc. Conference on Soil Dynamics and Earthquake Engineering. Southampton, 925-939

Juang, C.H, Yang, S.H, Yuan, H., and Fang, S.Y., 2005, Liquefaction in the Chi-Chi earthquake – effect of fines and capping non-liquefiable layers Journal of the Japanese Geotechnical Society of Soils and Foundations, Vol. 45 No. 6 pp 89-101

Karamitros, Bouckovalas, Chaloulos, and Andrianopoulos, 2013, Numerical analysis of liquefaction-induced bearing capacity degradation of shallow foundations on a two-layered soil profile, Soil Dynamics and Earthquake Engineering, Vol 44.

Tonkin & Taylor Ltd, 2013, Liquefaction Vulnerability Study, Earthquake Commission, T&T Ref

Lateral Spread Evaluation [Youd, Hansen, and Bartlett (2002)]

Proj: Stratham Eastvale
Proj No.: 38.0000030332 Date: 11/15/2024

General Parameters:

Parameter		Units	Limits	Definitions; (units)
M	6.7	Mw	6<M<8	EQ Magnitude; (Mw)
R	5.8	km	R>=0.5km	Site to Source Distance; (km)
Ro	2.1			= 10^(0.89M-5.64) [Eq. 5]
R*	7.9			= R + Ro [Eq.4]

Boring Specific Parameters:

Boring Number:	LB-3			
Comment:	Only Sandy Soils			
Case:				
Top of Layer	12.5	ft		
Bottom of Layer	17.5	ft		
T_{15}	5.	ft		
T_{15}	1.52	m	1<T15<15	Thickness of saturated layers with (N1)60<=15 (and with M<8)
F_{15}	45.	%	see Fig 5	Average fines content (particles<0.075mm)
$(D_{50})_{15}$	**0.1**	mm	see Fig 5	Average D50 in T15
comment on Fig 5:	OK			see Youd et al 2009 for higher fines cont and clay-like layers vs sand-like soil ("Zero Disp Lat Sprds, Turkey")
Z_T	12.5	ft		
Z_T	3.81	m	1<Zt<10	Depth to top of liquefied layer **[not considered in analysis]. Lateral spread usually with Zt<3m, approx.**

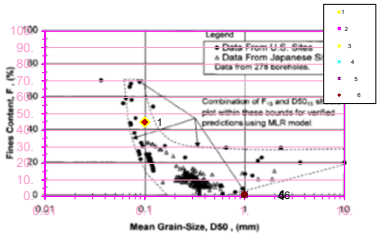


Fig. 5. Compiled grain-size data with ranges of F_{15} and $D50_{15}$ [for which Eq. (6) is applicable]

Free-Face Condition (rivers or channels):

Description:	Santa Ana River S of Site			
H(ft)	20.	ft		Free Face Height; (use consistent units w & L)
L(ft)	5500.	ft		Distance from free face
W	0.36	%	1<W<20	100 H/L
Log Dh=	-1.45			Eqn. 6a
Dh (m)=	0.04	m	Dh<6m	Displacement; flow failure may be possible if Dh>6m
Dh (ft)=	**0.12**	ft		Displacement
Dh (in.)=	**1.41**	in.		**Displacement**

Sloping Ground Condition:

Δ elevation (ft)	2.			
horiz. distance (ft)	100.			
Slope, S	2.	%	0.1<S<6	Sloping Ground
Log Dh=	-0.58			Eqn. 6b
Dh (m)=	0.26	m	Dh<6m	Displacement; flow failure may be possible if Dh>6m
Dh (ft)=	0.85	ft		Displacement
Dh (in.)=	**10.25**	in.		**Displacement**

References:

Bartlett, S.F., Youd, T.L, 1995, "Emperical Prediction of Liquefaction-Induced Lateral Spread," ASCE Journal of Geotechnical Engineering, Vol 121, No. 4, April, 1995.

Youd, T.L., DeDen, D.W., Bray, J.D., Sancio, R., Cetin, K.O., Gerber, T.M., 2009, "Zero-Displacement Lateral Spreads, 1999 Kocaeli, Turkey, Earthquake", ASCE Journal of Geotechnical and Geoenvironmental Engineering, January 2009.

Youd, T.L., Hansen, C.M., Bartlett, S.F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, ASCE Journal of Geotechnical and Geoenvironmental Engineering, December 2002.

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

VERDANTAS INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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1.0 General

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

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

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

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

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

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

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

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

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

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

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

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

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

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

4.0 Fill Placement and Compaction

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

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

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

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

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

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

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

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

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

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

APPENDIX F

GBA's INFORMATION ABOUT THIS 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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL BUSINESS ASSOCIATION

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