DRAFT

REPORT OF GEOTECHNICAL INVESTIGATION
FORMER PIONEER ELEMENTARY SCHOOL
1651 EAST ROWLAND AVENUE
CITY OF WEST COVINA, CALIFORNIA

Prepared For:

LEWIS LAND DEVELOPERS, LLC
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Project No. 12064.004

April 17, 2020

DRAFT

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To: Lewis Land Developers, LLC
1156 North Mountain Avenue
Upland, California 91786

Attention: Mr. Adam Collier
Project Manager

Subject: Report of Geotechnical Investigation, Proposed Residential Development, Former Pioneer Elementary School, 1651 East Rowland Avenue, City of West Covina, California

In response to your request and authorization, Leighton and Associates, Inc. (Leighton) has conducted a geotechnical investigation for a proposed residential development within the former campus of Pioneer Elementary School located at 1651 East Rowland Avenue in the City of West Covina, California. The purpose of this study has been to evaluate the geotechnical conditions with respect to the proposed development and to provide geotechnical recommendations for design and construction of the improvements.

The most significant geotechnical issues at the site include the presence of compressible soils and the potential for strong seismic shaking. Good planning and design of the project can limit the impacts of these constraints. This report presents our findings, conclusions, and preliminary geotechnical recommendations for the project.

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

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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Principal Engineer

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Principal Geologist

AIK/SGO/JDH/PB/rsm

Distribution: (1) Addressee

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1.0 INTRODUCTION

1.1 Site Location and Description

The approximately 9-acre site formerly used as the campus of Pioneer Elementary School is located at 1641 East Rowland Avenue (north of East Rowland Avenue and west of North Azusa Avenue) in the City of West Covina, California. Existing retail properties are present to the northeast and east, existing residences are located to the northwest and west, and Rowland Avenue bounds the site to the south. In general, vacant school structures occupy the southern portion of the campus with asphalt parking areas adjacent to Rowland Avenue as well as in the northeastern portion of the site. Grass fields are present in the northwest and western portions of the property. The site is relatively flat and drains gently to the south.

Based on our review of historical aerial photographs, it appears that the Pioneer site was used for agricultural purposes from prior to 1948 until approximately 1964 when rough grading for school buildings appeared to begin. Development of the northeastern parking lot began in 1965. The school on the site became defunct in 1989 and appears to have been fully abandoned by 2014. Since then, the site appears to have been left dormant.

1.2 Proposed Development

The 40-scale *Conceptual Site Plan: G-1* dated January 27, 2020 that you provided shows the development of 66 homes, 158 townhomes, and a recreation area as well as parkways, parking areas, hardscape and landscape improvements. Based on the relative flatness of the site, we anticipate shallow cuts and fills less than 5 feet thick will be required to achieve design grades.

1.3 Purpose of Study

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

1.4 Scope of Study

The scope of our geotechnical study included the following tasks:

- Document Review: We reviewed pertinent, readily available geologic and geotechnical literature covering the site. Our review included regional geologic maps and reports and historical aerial photographs available in our library and online as well as the site plan you provided.

- Site Clearance: We coordinated with Underground Service Alert (USA) and private utility service (GPRS) to have existing underground utilities located and marked prior to our subsurface investigation.

  Field Exploration: A total of six (6) exploratory soil borings (LB-1 through LB-6) were excavated, logged, and sampled at selected locations throughout the site to observe and evaluate subsurface conditions. The borings were drilled to a maximum depth of 51.5 feet below the existing ground surface (bgs) by a subcontracted drill rig operator and logged at the surface by our field representative during drilling. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests were performed at selected intervals, and soil samples were collected. Representative bulk soil samples were also collected from the borings.

  Borings were backfilled with soil cuttings and patched with cold patch asphalt at the surface in parking areas. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Boring Location Map, Figure 2.

- Infiltration Testing: Well permeameter tests were conducted within two of our borings (LB-1 and LB-2) onsite to estimate infiltration characteristics of subsurface soils at the depths and locations tested. Well permeameter tests were conducted based on the USBR-7300-89 method and in general accordance with Los Angeles County guidelines. Tests were conducted at depths of approximately 15 feet bgs to estimate the infiltration rate.

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

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

In situ moisture content and dry density test results are presented on the boring and test pit logs in Appendix B. Results of the remaining laboratory tests are presented in Appendix C.

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

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

2.0 FINDINGS

2.1 Regional Geologic Conditions

The site is located in the northeastern portion of the Los Angeles Basin within the Peninsular Ranges geomorphic province of California. The Peninsular Ranges are characterized by elongate structural blocks bounded by northwest to west-northwest trending fault zones. Several of these faults terminate at or merge with the east-west trending thrust faults at the southern edge of the Traverse Ranges geomorphic province to the north of the site. Several faults that have been mapped in the region are active or potentially active and are believed to accommodate stresses associated with the interaction between the two geomorphic provinces. These faults include the Indian Hills fault (approximately 1 mile east of the site), the Walnut Creek fault (approximately 1.6 miles southeast of the site), and the Sierra Madre Fault Zone (approximately 3.4 miles north of the site). The site is underlain by alluvial soil deposits eroded from surrounding mountains and deposited in the site vicinity. Previous grading to accommodate the former school has resulted in the placement of artificial fill in portions of the site.

2.2 Subsurface Soil Conditions

Based on our review of pertinent geologic data, the site is mapped as being underlain by Holocene-age older alluvial soil deposits. The alluvial soil is generally described as alluvial gravel, sand and silt (Dibblee and Ehrenspeck, 1999).

Based upon field exploration, the onsite soil encountered consisted of alluvial deposits consisting of silt with sand, overlain in many areas by artificial fill.

Near surface alluvial soil encountered at the site generally consisted of silty sand and poorly graded sand. Below depths of about 30 feet, sandy silt, sandy clay and clayey sand was also encountered. The soils were medium dense in the upper 15 feet, becoming stiff or very dense with depth.

Artificial fill was observed in each of our borings to depths of approximately 5 feet bgs. The fill is generally composed of silty sand that is loose to medium dense.

2.2.1 Compressible and Collapsible Soil

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

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Based on this study, the onsite soils are anticipated to have a low collapse potential when inundated with water.

2.2.2 Expansive Soils

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

A representative soil sample from the site yielded an expansion index of 7 Based on this laboratory result, the onsite near-surface soil is generally expected to exhibit a very low to low expansion potential.

2.2.3 Sulfate Content

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

A near-surface soil sample was tested during this study for soluble sulfate content. The result of this test indicated a sulfate content of less than 0.1 percent by weight, indicating negligible sulfate exposure. Recommendations for concrete in contact with the soil are provided in Section 3.6.

2.2.4 Resistivity, Chloride and pH

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

As a screening for potentially corrosive soil, representative soil samples were tested during this study to estimate minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 3,050 ohm-cm, chloride content of 70 ppm, and pH of 7.6. Based on these results, the onsite soil is considered moderately corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in any of the borings drilled to a maximum depth of 51.5 feet below the existing ground surface.

California Geological Survey has reported historically highest groundwater levels beneath the site to be in the range of 100 to 150 feet bgs (CGS, 1998). A well located approximately 2 miles west-southwest of the site maintained by the Main San Gabriel Basin Watermaster a highest historic groundwater level of approximately 144 feet bgs based on measurements taken from July 2011 through July 2019. Groundwater is not expected to be constraint to site development.

2.4 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region could include strong ground shaking and fault rupture. The potentials for fault rupture and seismic shaking are discussed below.

2.4.1 Surface Faulting

The State of California has mapped the site to be outside of an Earthquake Fault Zone. Our review of available other in-house and online literature indicated that no known active faults have been mapped across the site. Based on our understanding of the current geologic framework, the potential for future surface rupture onsite is low.

2.4.2 Seismic Design Parameters

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 relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2019 CBC:

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude: 34.0802, -117.9101	
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	1.658 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.610 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000 g
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.700* g
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.658 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.037* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.105 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.691* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.702 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	0.772 g

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

Based on the 2019 CBC Table 1613.2.3(2) footnote c., F_v should be determined in accordance with Section 11.4.8 of ASCE 7-16, since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D; in accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is required. However, the values provided in the table above may be utilized if design is performed in accordance with Exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_s), and F_v is only used for calculation of T_s. This exception does not apply (and the values in the table above would not be applicable) for proposed structures with a fundamental period of vibration greater than 0.5 s on sites with potentially liquefiable soils; it also does not apply for structures with seismic isolation or seismic damping systems. The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.7 (M_W) at a distance on the order of 11.7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

2.5 Secondary Seismic Hazards

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

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The site has been mapped outside of a Liquefaction Zone by the State of California (CGS, 1998). Additionally, with the absence of shallow groundwater, the potential for liquefaction to occur onsite is low.

2.5.2 Seismically Induced Settlement

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

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). The results of our analyses indicate that the onsite soils are susceptible to about 2½ to 3½ inches of seismic settlement based on the PGA_M of 0.77g. Differential settlement due to seismic loading considering the PGA_M is estimated to be 1½ inches over a horizontal distance of 40 feet based on the MCE. The resultant seismic settlement is primarily due to loose sands encountered within the upper 10 feet. Seismic settlement potential is anticipated to be reduced to about 1½ inches after preparing building pads in accordance with our over excavation and compaction recommendations in Section 3.1; differential settlement due to seismic loading is estimated to be less than 1 inch over a horizontal distance of 40.

Based on the seismic settlement analyses, the building would not be subject to collapse, nor would it be subject to special design considerations. A summary of seismic settlement analysis is included in Appendix D.

2.6 Infiltration Testing

Two well permeameter tests (LB-1 and LB-2) were conducted onsite and were located based on our previous discussions. LB-1 was located in the north central portion of the site, Boring LB-2 was located in the southwest. Well permeameter tests were performed within granular soils at depths of about 15 feet.

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 sand is placed in the boring bottom to support temporary perforated well casing pipe and a float valve. In addition, gravel is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float valve, lowered into the boring inside the casing, adds water 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.

Well permeameter testing indicated a raw infiltration rate of 1.0 inch per hour at location B-1 and essentially no infiltration at location B-2. See Section 3.7 for infiltration recommendations, including infiltration rates.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, the proposed residential development at this site is suitable from a geotechnical standpoint, provided the findings, conclusions and recommendations presented in this report are incorporated into the planning, design and construction of the project. No severe geologic or soils related issues were identified that would preclude the proposed development of the site. One to three-story structures may be founded on conventional spread footings bearing on a zone of compacted fill soils, derived from site soils. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking and compressible soils.

Although not identified during this study, abandoned utility lines, or other buried structures 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 grading should be performed in accordance with the *General Earthwork and Grading Specifications* presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

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

3.1.2 Overexcavation and Recompaction

Based upon this study, one- to three- story structures proposed for the development may be supported on shallow foundation systems. However, in order to reduce the potential for adverse differential settlement, the underlying subgrade soil must be prepared in such a manner that a uniform response to the applied loads is achieved.

All artificial fill should be removed to firm native soil. The onsite alluvial soil should be overexcavated a minimum of 6.5 feet below existing grade or 3 feet below the bottom of footings, whichever is deeper. If compressible, loose, or overly dry soils are found, the removal should be continued until firm native soil is encountered. All such areas should be observed in the field by a Leighton representative prior to fill placement. Where possible, overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of proposed footings (including footings for exterior columns structurally connected to the building), or a horizontal distance equal to the depth of overexcavation below footings, whichever is farther.

Areas outside the overexcavation limits of structures planned for asphalt or concrete pavement, flatwork, sidewalks, and areas to receive fill should be overexcavated a minimum depth of 12 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper.

After completion of 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.

These recommendations should be reviewed once grading plans for the development are available.

3.1.3 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should be free of debris, organic material and oversized material (greater than 8 inches in largest dimension). Significant oversized material was not observed during our work on the site. 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 at or slightly above optimum moisture content. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 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 natural ground is moisture-conditioned and densified to receive fill. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site and the measured in-place densities of soils encountered. We anticipate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15 percent (alluvium) ±3%. Approximately 5 percent (existing compacted fill) ±3%
Subsidence	Approximately 0.10 foot (alluvium)

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.

3.2 Recommendations for Foundations

Based on our study, conventional shallow foundations or post-tensioned foundations may be used to support the loads of 1- to 3-story wood-frame structures. Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. If taller structures are planned, additional evaluation should be provided based on the proposed design. The following design parameters are based on soils with a low expansion potential. Additional testing of the soils expansion should be conducted at the conclusion of site grading.

3.2.1 Minimum Embedment and Width

Footings for one to three-story structures should have a minimum embedment depth in accordance with California Building Code (CBC) requirements, with a minimum width of 24 and 15 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on 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 4,000 psf. If additional allowable bearing pressure is needed, this should be evaluated on a case-by-case basis. 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 an allowable coefficient of friction of 0.35. The passive resistance may be computed using an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. Friction and passive pressure may be combined without reduction, provided the footings can move laterally sufficiently to develop passive pressure (approximately ¼ inch); otherwise, friction alone should be assumed.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

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

3.2.5 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 40 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

As discussed in Section 2.5.2, the potential total seismic settlement is estimated to be about 2½ to 3½ inches for the design earthquake in the sites current state. This is primarily due to loose sands encountered within the upper 10 feet. Seismic settlement is reduced to about 1½ inches after preparing building pads in accordance with our over excavation and compaction recommendations in Section 3.1. Differential settlement due to seismic loading is estimated to be less than 1 inch over a horizontal distance of 40.

3.3 Recommendations for Slabs-On-Grade

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 the end of rough grading to evaluate the expansion index of near-surface subgrade soils. Slabs-on-grade should have the following minimum recommended components:

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

- Concrete and Structural Design Thickness: Slabs-on-grade should be designed by the structural engineer, but should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced slabs) should be No. 3 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, aggregate that is not sufficiently clean, 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, reinforcement in slabs and foundations can generally reduce the potential for shrinkage cracking. The structural engineer should consider these and other pertinent concrete design and construction considerations in slab design and specifications.

3.3.1 Slab Underlayment for Moisture Vapor Retarding

Because moisture vapor from the underlying soils will be transmitted through slabs-on-grade without preventive measures, slab underlayment for moisture vapor retarding should be designed by qualified professionals (such as the structural engineer and/or architect) where control of moisture vapor transmission through slabs is considered important to this project (such as where moisture-sensitive floor coverings or equipment are planned). Slab underlayment typically includes a moisture vapor retarder membrane (such as 10-mil thick or greater), underlain by a capillary break and provisions for protection of the vapor retarder during construction. The structural engineer and/or architect should specify pertinent slab and concrete design parameters, such as whether a sand blotter layer should be placed over the vapor retarder (ACI does not recommend placing sand under the slab and above the vapor barrier, but rather recommends specific concrete properties and curing procedures to mitigate cracking/curling during curing, such as wet curing of the slab to reduce the potential of rapid top hydration).

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute (ACI), Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

Leighton does not practice in the field of moisture vapor transmission evaluation/mitigation, since this does not fall under the geotechnical discipline. Therefore, we recommend that a qualified person, such as the flooring subcontractor, structural engineer, and/or architect, be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person (or persons) should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate. In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the current California Building Code. The CBC seismic design parameters listed in of Section 2.4.2 of this report should be considered for the seismic analysis of the subject site.

3.5 Lateral Earth Pressures

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 3, *Retaining Wall Backfill and Subdrain Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall and are, therefore, not recommended. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Table 1 - Lateral Earth Pressures

Equivalent Fluid Pressure (pcf)	
Condition	**Level Backfill**
Active	40
At-Rest	60
Passive	350 (Maximum of 5,000 psi)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

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

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that soil providing passive resistance, embedded against the foundation elements, will remain intact with time. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

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 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 28 pcf, which is added to the EFP. The *additive seismic* EFP should be applied at the retained midpoint.

Conventional retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 12 inches below the lowest adjacent grade. An allowable bearing pressure of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 300 psf per foot increase in width or depth to a maximum allowable bearing pressure of 4,500 psf.

3.6 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. Concrete should be designed in accordance with ACI 318-14, Section 19.3 (ACI, 2014), adopted by the 2019 CBC (Section 1904.2).

Based on our laboratory testing, the onsite soil is considered moderately corrosive to ferrous metals. Non-metallic underground utilities should be used. As an alternative, corrosion protection of underground metallic utilities should be based on recommendations of a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility contractors and consultation with a Corrosion Engineer should be considered.

3.7 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and an assumed design R-value of 45, preliminary flexible pavement sections may consist of the following for the Traffic Indices (TI) 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.

Table 2 - Asphalt Pavement Section Thicknesses

Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
5 or less	3	4
6	3.5	4.5
7	4	6

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.

PCC sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing will help reduce severity of cracking.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. 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.

3.8 Infiltration Recommendations

Infiltration tests performed at depths of about 15 feet yielded a raw infiltration rate of 1.0 inch per hour at location B-1 (central portion of the site) and essntiall no infiltration at location B-2 (southern portion of the site). Considering these results, infiltration into the onsite soils in the south will be marginal at best. Infiltration systems may not be suitable in portions of the site. If infiltration systems are to be considered, additional testing at the location and depth may be warranted. It appears that deep chambers reaching at least 18 to 20 feet bgs or dry wells may be feasible options.

These measured rates are applicable only at the specific locations and depths tested. 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.

We recommend that a correction factor/safety factor be applied to this infiltration rate in conformance with the Los Angeles County Administrative Manual (2014), since monitoring of actual facility performance has shown that actual infiltration rates are lower than for small-scale tests. The small-scale infiltration rate should be divided by a correction factor of at least 3, but the correction/safety factor may be higher based on project specific aspects.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

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

Additional Review and Evaluation

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

General Design Consideration

The periodic flow of water carrying sediments into the basin or chamber, plus the introduction of wind-blown sediments and sediments from erosion of the basin side walls, can eventually cause the bottom of the basin or chamber to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape on site. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these. Such features should be brought to Leighton's attention as they are identified.

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

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overfilling to not be a concern to the facility or nearby improvements.

For buried chambers, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the cambers.

<u>Additional Design Considerations (Particularly to Open Basins)</u>

If open basins are planned, additional evaluation may be needed, as the soils that will be exposed at the bottom of the basin are critical to the basin's success. Soils at the bottom of buried chambers are also important, but not as critical to their success, provided the infiltration chamber cuts through sufficiently granular soils.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber, these conditions would be of less concern.

Parks or play/recreation areas should not be constructed within basin bottoms or below the spillway level.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install infiltration trenches or borings in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/boring depth and media backfill material, would be developed based on conditions observed. Such a contingency should be anticipated for open basins.

Construction Considerations

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

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations

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

Additional Review and Evaluation:

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

3.9 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, and the current edition of the California Construction Safety Orders, latest edition. OSHA Type C soils should be assumed for planning purposes.

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 the active fluid pressure presented in the retaining wall section. 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 22H, where H (feet) 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 Leighton should be maintained to facilitate construction while providing safe excavations.

3.10 Surface Drainage

Positive surface drainage should be provided to direct surface water away from structures and towards suitable collective drainage facilities. 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 buildings. Care should be taken to avoid heavy irrigation, and under-irrigation should also be avoided.

3.11 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our 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 analysis may be required based on final development 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 recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our 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 Lewis Land Developers and their design team for application to the design of the proposed development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the Geoprofessional Business Association (GBA) insert on the following page for important information about this geotechnical engineering report.

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Important Information about This

Geotechnical-Engineering Report

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

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

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

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

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

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

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This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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

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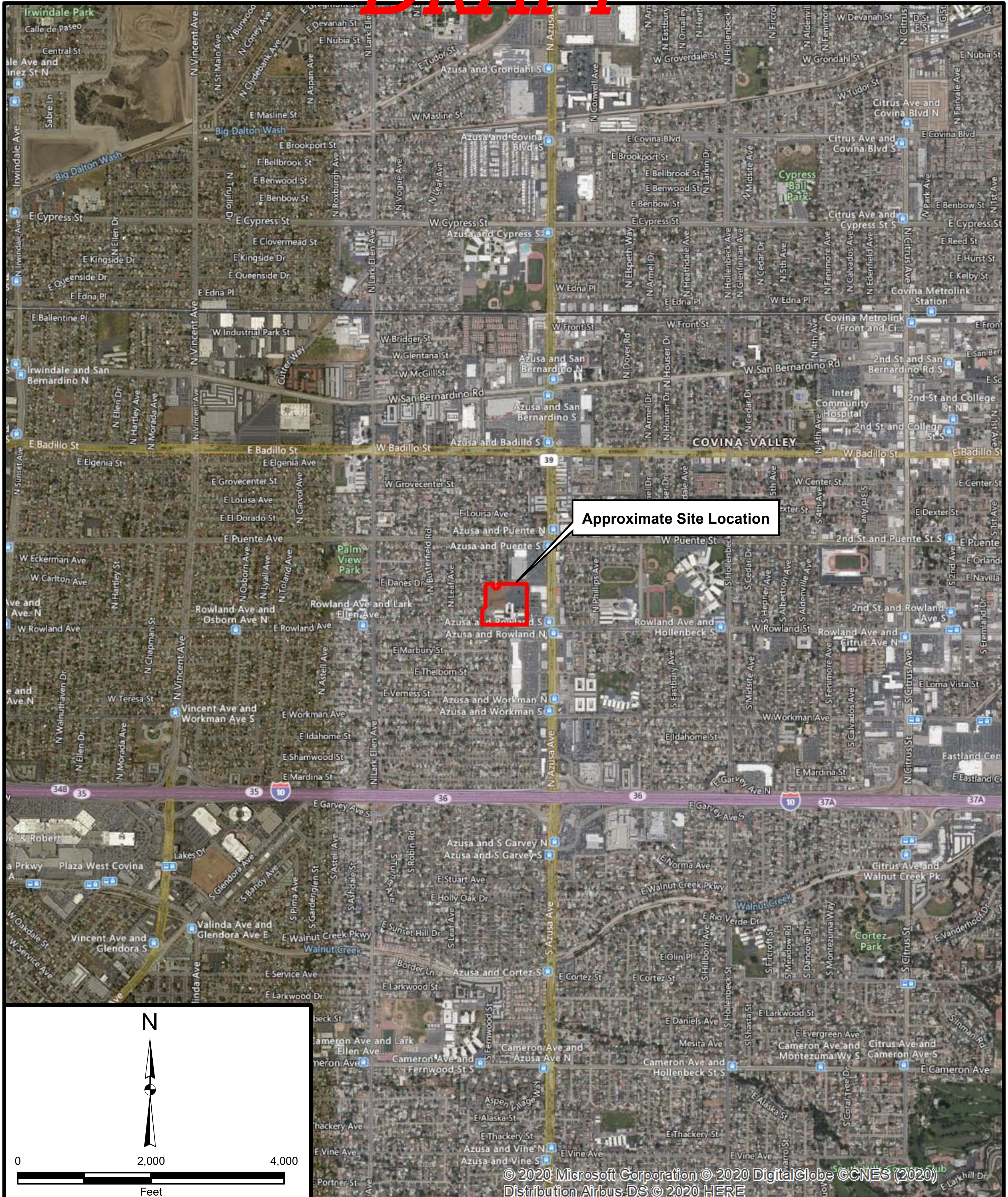

Project: 12064.004	Eng/Geol: JDH/PB
Scale:1 " = 2,000 '	Date: April 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton
Author: Leighton Geomatics (kmanchikanti)

SITE LOCATION MAP

Proposed Residential Development
Former Pioneer School Site
E. Rowland Avenue, City of West Covina, California

Figure 1

Leighton

Map Saved as V:\Drafting\12064\004\Maps\12064.004_F01_SLM_2020-04-14.mxd on 4/16/2020 1:10:12 PM

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Project: 12064.004	Eng/Geol: JDH/PB
Scale:1 " = 100 '	Date: April 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton
Author: Leighton Geomatics (kmanchikanti)

BORING LOCATION MAP

Proposed Residential Development, Former Pioneer School Site
E. Rowland Avenue
City of West Covina, California

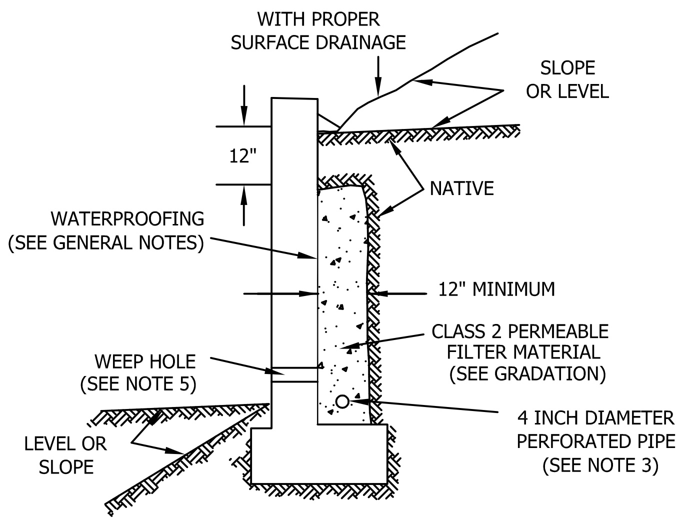
Figure 2

Leighton

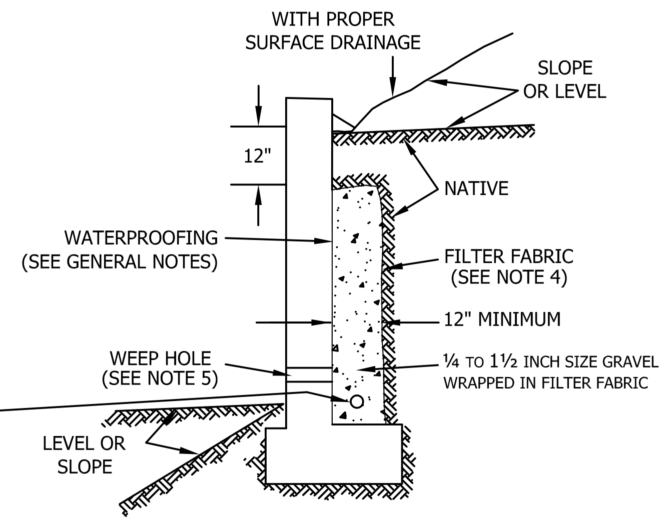
Map Saved as V:\Drafting\12064\004\Maps\12064.004_F02_BLM_2020-04-14.mxd on 4/16/2020 2:12:38 PM

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 3

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APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING LOGS AND
INFILTRATION TEST RESULTS

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GEOTECHNICAL BORING LOG B-1

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~469'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Artificial Fill (Af):** @0': Grass at the surface. SILTY SAND (SM): orangish brown; moist, fine grained.	
				R-1	5 7 16	104	4		@2.5':SILTY SAND (SM): orangish brown, medium dense, moist, fine grained, pinhole pores, trace rootlets.	
	5			R-2	8 16 24	111	4		**Quaternary Alluvium (Qal):** @5': SILTY SAND (SM): orangish brown, medium dense, moist, fine grained, pinhole pores, trace rootlets.	
	10			R-3	22 28 35	110	4		@10': SILTY SAND (SM): orangish brown, dense, moist, fine grained, trace fine subangular gravel, pinhole pores.	
	15			R-4	11 20 21			SP	@15': SAND (SP): light yellowish brown, medium dense, fine to coarse grained, some fine subangular gravel.	
	20			R-5	22 35 35				@20': SAND (SP): light yellowish brown, dense; light yellow brown, moist, fine to coarse grained, some fine subangular to subrounded gravel.	
									Total Depth: 21.5 feet **No groundwater observed** **Backfilled with soil cuttings**	
	25									
	30									

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

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

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

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GEOTECHNICAL BORING LOG B-2

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~466'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**Artificial Fill (Af):** @0': Grass at the surface. SILTY SAND (SM): dark brown, moist, fine grained.	
				R-1	2 2 3	104	12		@2.5': SILTY SAND (SM): dark brown, very loose, moist, fine grained, pinhole pores, trace roots.	
	5			R-2	2 4 7	104	9		**Quaternary Alluvium (Qal):** @5': SILTY SAND (SM): orangish brown, loose, moist fine grained, trace rootlets.	
	10			R-3	4 6 7	114	11		@10': SILTY SAND (SM): orangish brown, loose, moist, fine grained, trace fine subangular gravel.	
	15			R-4	5 17 24			SP	@15': SAND (SP): light yellowish brown, medium dense, moist, fine to coarse grained, few fine gravel, trace medium gravel, subangular to subround, some mechanical fracturing; weak cementation.	
	20			R-5	7 40 42				@20': SAND (SP): light yellowish brown, dense, moist, fine to coarse grained, poorly graded, few fine to medium gravel.	
									Total Depth: 21.5 feet **No groundwater observed** **Backfilled with soil cuttings**	
	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 B-3

DRAFT

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~469'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								@0': 4 inches of asphalt concrete over 6 inches of base.	
				B-1				SM	**Artificial Fill (Af):** @0.8': SILTY SAND (SM): dark brown, moist, fine grained.	
				R-1	2 3 3	109	11		@2.5': SILTY SAND (SM): dark brown, very loose, moist, fine grained, few micacous grains.	
	5			R-2	3 4 7	113	9		**Quaternary Alluvium (Qal):** @5': SILTY SAND (SM): dark brown, loose, moist, fine to coarse grained, trace quartzite fragments.	
	10			R-3	3 5 6	111	15		@10': Silty SAND; loose; dark brown; moist; some fine sand; few medium to coarse sand; some silt; trace quartzite fragments, fine mechanical fracturing.	
	15			R-4	16 36 42	119	2	SM-SP	@15': SAND to SILTY SAND (SM-SP): yellowish brown, dense, moist, fine to coarse grained, poorly graded, few finesubrounded to angular gravel.	
	20			R-5	17 50/6"			SP	@20': SAND (SP):yellowish brown, very dense, moist; some fine sand, fine to coarse grained, poorly graded, few fine angular and subangular gravel, trace carbonates.	
	25			S-6	35 50/5"				@25': SAND (SP): yellowish brown, very dense, moist, fine to coarse grained, poorly graded, few fine angular and subround gravel.	
	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 B-3

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~469'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	30			R-7	12 12 20			ML-CL	@30': SANDY SILT to SANDY CLAY (ML-CL): orangish brown, very stiff, moist, some fine sand, trace coarse sand.	
	35			S-8	5 8 8				@35': SILT to CLAY with SAND (ML-CL): orangish brown, stiff to very stiff, moist, with fine sand.	
	40			R-9	10 14 17			SC	@40': CLAYEY SAND (SC): orangish brown, medium dense, moist, fine to coarse grained.	
	45			S-10	30 50/3.5"			SM	@45': SILTY SAND (SM): yellowish brown, very dense, moist, fine to coarse grained, trace subangular to angular gravel, fractured quartzite pieces.	
	50			R-11	21 23 34				@50': SILTY SAND (SM): orangish brown, dense, moist, fine grained.	
									Total Depth: 51.5 feet **No groundwater observed** **Backfilled with soil cuttings, tamped, and patched with asphalt**	
	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

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

GEOTECHNICAL BORING LOG B-4

DRAFT

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~467'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	**Artificial Fill (Af):** @0': Grass at the surface. SILTY SAND (SM): dark brown, moist, fine grained.	
				R-1	3 3 3	104	11		@2.5': SILTY SAND (SM): dark brown, very loose, moist, fine grained, few roots.	
	5			R-2	3 2 3	107	9		**Quaternary Alluvium (Qal):** @5': SILTY SAND (SM): orangish brown, very loose, moist, fine grained, pinhole pores, trace roots.	
	10			R-3	13 12 13	113	6		@10': SILTY SAND (SM): orangish brown, medium dense, moist, fine grained, trace fine subngular to subround gravel.	
	15			R-4	3 5 5	111	14		@15': SILTY SAND (SM): orangish brown, loose, moist, fine grained, few fine subangular to subround gravel.	
	20			R-5	30 50/5"			SP	@20': SAND (SP): light yellowish brown, very dense, moist,fine to coarse grained, poorly graded, few to little subangular gravel.	
									Total Depth: 21.5 feet **No groundwater observed** **Backfilled with soil cuttings**	
	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
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GEOTECHNICAL BORING LOG B-5

DRAFT

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~473'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	@0': 3 inches of asphalt concrete over 4 inches of base. **Artificial Fill (Af):** @0.6': SILTY SAND (SM): dark brown, moist, fine grained.	
				R-1	2 3 4	107	18		@2.5': SILTY SAND (SM): dark brown, very loose, moist, fine grained, trace fine gravel in cuttings, few roots.	
	5			R-2	4 7 10	116	11		**Quaternary Alluvium (Qal):** @5': SILTY SAND (SM): dark brown, medium dense, moist, fine grained, trace roots.	
	10			R-3	4 6 6	115	11		@10': SILTY SAND (SM): orangish brown, loose, moist, fine grained.	
	15			R-4	3 3 4	112	11		@15': SILTY SAND (SM): orangish brown, very loose, moist, fine grained.	CO
	20			R-5	5 11 22	117	8		@20': SILTY SAND (SM): orangish brown, medium dense, moist, fine grained, few fine angular to subangular gravel at sampled interval at depths of 21' - 21.5', few micacous grains, medium gravel in sampler shoe.	
									Total Depth: 21.5 feet **No groundwater observed** **Backfilled with soil cuttings and patched with asphalt**	
	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
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GEOTECHNICAL BORING LOG B-6

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~469'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SC	**Artificial Fill (Af):** @0': Grass field at the surface. CLAYEY SAND (SC): dark brown, moist, fine grained, with minor amounts of silt, fine subround gravel in cuttings.	-200, MD, EI, CR
				R-1	5 5 4	99	13		@2.5': CLAYEY SAND (SC): dark brown, loose, moist, fine grained, with minor amounts of silt, few roots throughout sample.	
	5			R-2	3 4 7	112	7	ML-SM	**Quaternary Alluvium (Qal):** @5': SANDY SILT to SILTY SAND (ML-SM): orangish brown, loose, moist, fine grained.	
	10			R-3	7 23 28	112	5		@10': SANDY SILT to SILTY SAND (ML-SM): orangish brown, dense, moist, fine grained.	
	15			R-4	7 16 20			SM	@15': SILTY SAND (SM): orangish brown, medium dense, moist, fine to coarse grained, trace fine subangular gravel, pinhole pores.	
	20			R-5	8 21 30			SP	@20': SAND (SP): yellowish brown, dense, moist, fine to coarse grained, poorly graded, little fine angular to subangular gravel.	
	25			S-6	13 25 50/5"				@25': SAND (SP): yellowish brown, very dense, moist, fine to coarse grained, poorly graded, little fine angular to subangular gravel.	
	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 B-6

Project No.	12064.004	**Date Drilled**	3-24-20
Project	Pioneer Geo Investigation	**Logged By**	MM
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~469'
Location	See Figure 2- Boring Location Map	**Sampled By**	MM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-7	50/5"				@30': SAND (SP): yellowish brown, very dense, moist, fine to coarse grained, poorly graded, minor amounts of silt, few fine subangular to angular gravel.	
	35			S-8	13 40 44				@35': SAND (SP): yellowish brown, very dense, moist, fine to coarse grained, poorly graded, minor amounts of silt, few fine subangular to angular gravel.	
	40			R-9	16 14 16			ML-CL	@40': SILT to CLAY (ML-CL): orangish brown, stiff, moist, fine sand lenses, low plasticity.	
	45			S-10	4 13 25			SM-SC	@45': SILTY SAND to CLAYEY SAND (SM-SC): orangish brown, dense, moist, fine grained, minor amounts of clay.	
	50			R-11	12 20 24			SM	@50': SILTY SAND (SM): light orangish to yellowish brown, medium dense, moist, fine grained.	
									Total Depth: 51.5 feet **No groundwater observed** **Backfilled with soil cuttings**	
	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
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Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: Pioneer Geo Inv 12064.004

Exploration #/Location:	*B1*	Initial estimated Depth to Water Surface (in.):	132
Depth Boring drilled to (ft):	15	Average depth of water in well, "h" (in.):	46
Tested by:	JDO	approx. h/r:	8.8
USCS Soil Type in test zone:	SM	Tu (Fig. 8) (ft):	114.0
Weather (start to finish):	Sunny	Tu>3h?:	yes, OK
Liquid Used/pH:	Water		
Measured boring diameter:	10.5 in.	5.25 in. Well Radius	Cross-sectional area for vol calcs (in.^2): 34.6
Approx Depth to GW below GS:	125 ft		
Well Prep:			

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	14. ft	10. in.	178	
Pilot Tube stickup (+ is above ground)		0. in.	0	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	9. ft	6. in.	114	114 Depth below GS (in.)
Float Assembly ID		E		
Float assembly Extension length (in.)		34		

Diameter of barrels (in.): 22.5

No. of Supply barrels: 1

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

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of pilot tube) ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	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 3/30/2020	Start time: 9:27																			
-																				
3/30/20	9:27	33						0	0.0	178.0										
3/30/20	9:37	25	11.22				10	10	134.6	43.4	-134.6	111	3179	4658	7838	784	47026	0.9	10.23	**11.60**
3/30/20	9:49	23	11.23				12	22	134.8	43.2	-0.12	43	795	4	799	67	3995	0.9	0.60	**2.43**
3/30/20	9:59	21.5	11.23				10	32	134.8	43.2	0	43	596	0	596	60	3577	0.921	0.54	**2.18**
3/30/20	10:09	20	11.23				10	42	134.8	43.2	0	43	596	0	596	60	3577	0.9	0.54	**2.18**
3/30/20	10:19	18.75	11.22				10	52	134.6	43.4	0.12	43	497	-4	493	49	2956	0.9	0.44	**1.80**
3/30/20	10:29	17.5	11.22				10	62	134.6	43.4	0	43	497	0	497	50	2981	0.9	0.45	**1.81**
3/30/20	10:39	16.25	11.23				10	72	134.8	43.2	-0.12	43	497	4	501	50	3005	0.9	0.45	**1.83**
3/30/20	10:49	15	11.22				10	82	134.6	43.4	0.12	43	497	-4	493	49	2956	0.9	0.44	**1.80**
3/30/20	11:03	13.25	11.23				14	96	134.8	43.2	-0.12	43	695	4	700	50	2998	0.9	0.45	**1.82**
3/30/20	11:14	12	11.23				11	107	134.8	43.2	0	43	497	0	497	45	2710	0.9	0.41	**1.65**
3/30/20						Refill														
3/30/20	11:20	31	11.2					113	134.4	43.6										
3/30/20	11:30	30	11.22				10	123	134.6	43.4	-0.24	43	397	8	406	41	2434	0.9	0.37	**1.48**
3/30/20	11:40	29	11.23				10	133	134.8	43.2	-0.12	43	397	4	402	40	2409	0.9	0.36	**1.47**
3/30/20	11:50	27.75	11.23				10	143	134.8	43.2	0	43	497	0	497	50	2981	0.9	0.45	**1.82**
3/30/20	12:00	26.5	11.22				10	153	134.6	43.4	0.12	43	497	-4	493	49	2956	0.9	0.44	**1.80**
3/30/20	12:10	25.5	11.22				10	163	134.6	43.4	0	43	397	0	397	40	2384	0.9	0.36	**1.45**
3/30/20	12:20	24.5	11.23				10	173	134.8	43.2	-0.12	43	397	4	402	40	2409	0.9	0.36	**1.47**
3/30/20	12:30	23.5	11.23				10	183	134.8	43.2	0	43	397	0	397	40	2384	0.9	0.36	**1.45**
3/30/20	12:40	22.25	11.24				10	193	134.9	43.1	-0.12	43	497	4	501	50	3005	0.9	0.45	**1.83**
3/30/20	12:50	21.75	11.23				10	203	134.8	43.2	0.12	43	199	-4	195	19	1167	0.9	0.18	**0.71**
3/30/20	1:00	21	11.22				-710	0	134.6	43.4	0.12	43	298	-4	294	0	-25	0.9	0.00	**-0.02**
3/30/20	1:10	20	11.23				10	0	134.8	43.2	-0.12	43	397	4	402	40	2409	0.9	0.36	**1.47**
3/30/20	1:20	19.25	11.23				10	0	134.8	43.2	0	43	298	0	298	30	1788	0.9	0.27	**1.09**
3/30/20	1:30	18.5	11.23				10	0	134.8	43.2	0	43	298	0	298	30	1788	0.9	0.27	**1.09**

template updated: 8/14/19

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

Leighton

Project: LePioneer Geo Inv 12064.004

Exploration #/Location: *B2*
Depth Boring drilled to (ft): 15
Tested by: JDO
USCS Soil Type in test zone: SM
Weather (start to finish): Sunny
Liquid Used/pH: Water
Measured boring diameter: 10.5 in. 5.25 in. Well Radius
Approx Depth to GW below GS: 125 ft
Well Prep:

Initial estimated Depth to Water Surface (in.): 136
Average depth of water in well, "h" (in.): 41
approx. h/r: 7.9
Tu (Fig. 8) (ft): 113.7
Tu>3h?: yes, OK
Cross-sectional area for vol calcs (in.^2): 34.6

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	14. ft	9. in.	177	
Pilot Tube stickup (+ is above ground)		0. in.	0	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	10. ft	1.5 in.	122	121.5 Depth below GS (in.)
Float Assembly ID		F		
Float assembly Extension length (in.)		30		

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

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of pilot tube)		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)
Start Date	Start time:		ft	in.									from supply	from Δh	Total					
3/30/2020	9:35																			
-																				
3/30/20	9:35	31.5						0	0.0	177.0										
3/30/20	9:42	28	11.61				7	7	139.3	37.7	-139.3	107	1391	4820	6211	887	53239	0.9	15.18	**13.53**
3/30/20	9:52	27.75	11.54				10	17	138.5	38.5	0.84	38	99	-29	70	7	422	0.9	0.08	**0.29**
3/30/20	10:02	27.5	11.56				10	27	138.7	38.3	-0.24	38	99	8	108	11	646	0.921	0.12	**0.44**
3/30/20	10:12	27.5	11.56				10	37	138.7	38.3	0	38	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	10:22	27.25	11.55				10	47	138.6	38.4	0.12	38	99	-4	95	10	571	0.9	0.10	**0.39**
3/30/20	10:32	27	11.56				10	57	138.7	38.3	-0.12	38	99	4	104	10	621	0.9	0.11	**0.42**
3/30/20	10:42	27	11.57				10	67	138.8	38.2	-0.12	38	0	4	4	0	25	0.9	0.00	**0.02**
3/30/20	10:52	26.75	11.55				10	77	138.6	38.4	0.24	38	99	-8	91	9	546	0.9	0.10	**0.37**
3/30/20	11:05	26.5	11.55				13	90	138.6	38.4	0	38	99	0	99	8	459	0.9	0.08	**0.31**
3/30/20	11:22	26.25	11.56				17	107	138.7	38.3	-0.12	38	99	4	104	6	365	0.9	0.07	**0.25**
3/30/20	11:32	26.25	11.55				10	117	138.6	38.4	0.12	38	0	-4	-4	0	-25	0.9	0.00	**-0.02**
3/30/20	11:42	26	11.54				10	127	138.5	38.5	0.12	38	99	-4	95	10	571	0.9	0.10	**0.39**
3/30/20	11:52	25.75	11.54				10	137	138.5	38.5	0	39	99	0	99	10	596	0.9	0.11	**0.40**
3/30/20	12:02	25.75	11.54				10	147	138.5	38.5	0	39	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	12:12	25.5	11.53				10	157	138.4	38.6	0.12	39	99	-4	95	10	571	0.9	0.10	**0.39**
3/30/20	12:22	25.5	11.53				10	167	138.4	38.6	0	39	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	12:32	25.5	11.53				10	177	138.4	38.6	0	39	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	12:42	25.25	11.54				10	187	138.5	38.5	-0.12	39	99	4	104	10	621	0.9	0.11	**0.42**
3/30/20	12:52	25.25	11.54				10	197	138.5	38.5	0	39	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	1:02	25.25	11.54				-710	0	138.5	38.5	0	39	0	0	0	0	0	0.9	0.00	**0.00**
3/30/20	1:12	25	11.53				10	0	138.4	38.6	0.12	39	99	-4	95	10	571	0.9	0.10	**0.39**
3/30/20	1:22	25	11.54				10	0	138.5	38.5	-0.12	39	0	4	4	0	25	0.9	0.00	**0.02**
3/30/20	1:32	24.75	11.53				10	0	138.4	38.6	0.12	39	99	-4	95	10	571	0.9	0.10	**0.39**

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APPENDIX C

LABORATORY TEST RESULTS

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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name: Lewis Pioneer | Tested By : O. Figueroa | Date: 04/07/20

Project No. : 12604.004 | Input By: A. Santos | Date: 04/15/20

Boring No.	B-6			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Dark brown (SC-SM)g			
Wet Weight of Soil + Container (g)	116.37			
Dry Weight of Soil + Container (g)	116.04			
Weight of Container (g)	67.73			
Moisture Content (%)	0.68			
Weight of Soaked Soil (g)	100.11			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	307			
Crucible No.	14			
Furnace Temperature (°C)	860			
Time In / Time Out	9:00 / 9:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	19.6893			
Wt. of Crucible (g)	19.6883			
Wt. of Residue (g) (A)	0.0010			
PPM of Sulfate (A) x 41150	41.15			
PPM of Sulfate, Dry Weight Basis	**41**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.9			
PPM of Chloride (C -0.2) * 100 * 30 / B	70			
PPM of Chloride, Dry Wt. Basis	**70**			

pH TEST, DOT California Test 643

pH Value	**7.64**			
Temperature °C	20.5			

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SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Lewis Pioneer	Tested By :	S. Seiler Date: 04/07/20
Project No. :	12604.004	Input By:	A. Santos Date: 04/15/20
Boring No.:	B-6	Depth (ft.) :	0-5
Sample No. :	B-1		

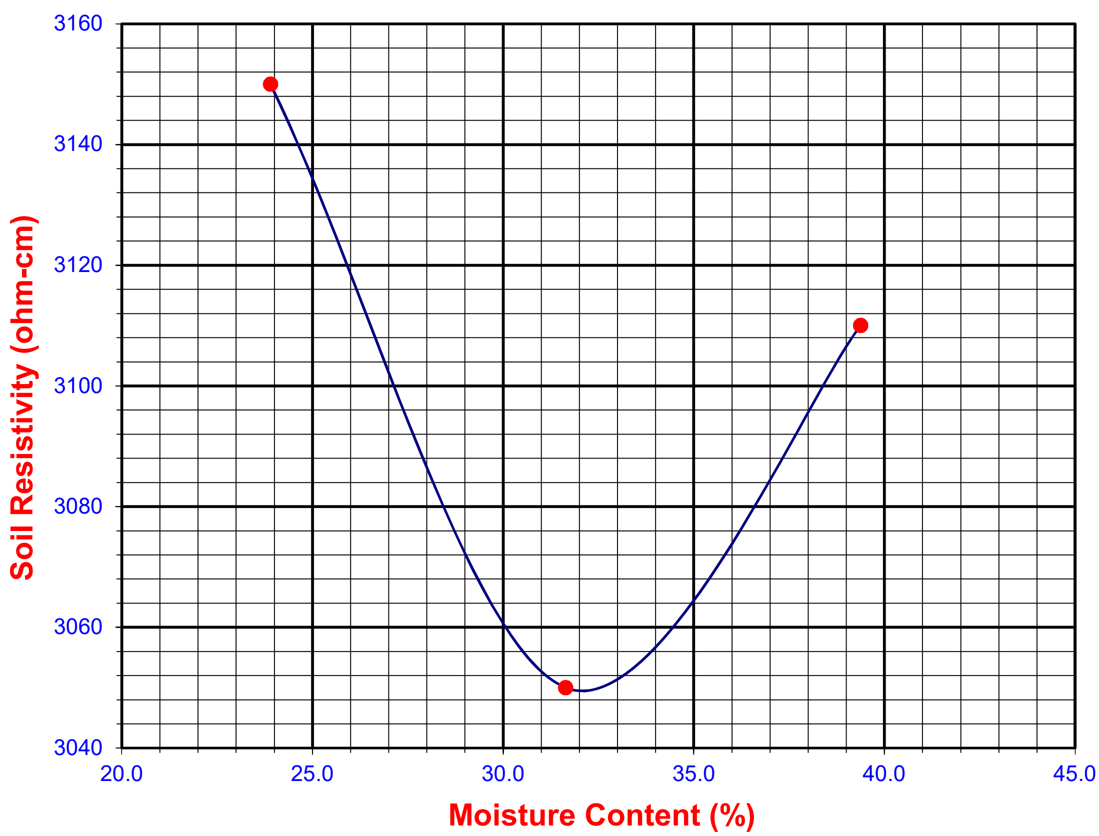
Soil Identification:* Dark brown (SC-SM)g

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.90	3150	3150
2	40	31.64	3050	3050
3	50	39.38	3110	3110
4				
5				

Moisture Content (%) (MCi)	0.68
Wet Wt. of Soil + Cont. (g)	116.37
Dry Wt. of Soil + Cont. (g)	116.04
Wt. of Container (g)	67.73
Container No.	
Initial Soil Wt. (g) (Wt)	130.10
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
3050	**32.20**	**41**	**70**	**7.64**	**20.5**



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EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	Lewis Pioneer	Tested By:	J. Gonzales	Date:	04/07/20
Project No.:	12604.004	Checked By:	A. Santos	Date:	04/18/20
Boring No.:	B-6	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark brown silty clayey sand with gravel (SC-SM)g				

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

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0065
Wt. Comp. Soil + Mold (g)	618.30	430.00
Wt. of Mold (g)	208.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	819.60	638.10
Dry Wt. of Soil + Cont. (g)	751.90	584.43
Wt. of Container (g)	0.00	208.10
Moisture Content (%)	9.00	14.26
Wet Density (pcf)	123.7	128.9
Dry Density (pcf)	113.5	112.8
Void Ratio	0.485	0.495
Total Porosity	0.327	0.331
Pore Volume (cc)	67.6	69.0
Degree of Saturation (%) [S_{meas}]	**50.1**	77.8

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.)
04/07/20	14:50	1.0	0	0.5800
04/07/20	15:00	1.0	10	0.5795
Add Distilled Water to the Specimen				
04/07/20	16:01	1.0	61	0.5855
04/08/20	7:45	1.0	1005	0.5865
04/08/20	8:45	1.0	1065	0.5865

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**7**

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MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	Lewis Pioneer	Tested By:	A. Lopez	Date:	04/02/20
Project No.:	12604.004	Input By:	A. Santos	Date:	04/16/20
Boring No.:	B-6	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark brown silty clayey sand with gravel (SC-SM)g				

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

Preparation Method:	X	Moist	Scalp Fraction (%)		Rammer Weight (lb.) = 10.0
		Dry	#3/4		Height of Drop (in.) = 18.0
Compaction Method	X	Mechanical Ram	#3/8		
		Manual Ram	#4	16.5	Mold Volume (ft³) 0.0333

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3891	3966	3922			
Weight of Mold (g)	1868	1868	1868			
Net Weight of Soil (g)	2023	2098	2054			
Wet Weight of Soil + Cont. (g)	448.3	478.1	558.9			
Dry Weight of Soil + Cont. (g)	406.8	424.8	486.0			
Weight of Container (g)	40.1	39.0	39.4			
Moisture Content (%)	11.32	13.82	16.32			
Wet Density (pcf)	133.9	138.9	136.0			
Dry Density (pcf)	120.3	122.0	116.9			

Maximum Dry Density (pcf)	**122.4**	**Optimum Moisture Content (%)**	**13.2**
Corrected Dry Density (pcf)	**128.2**	**Corrected Moisture Content (%)**	**11.2**

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

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

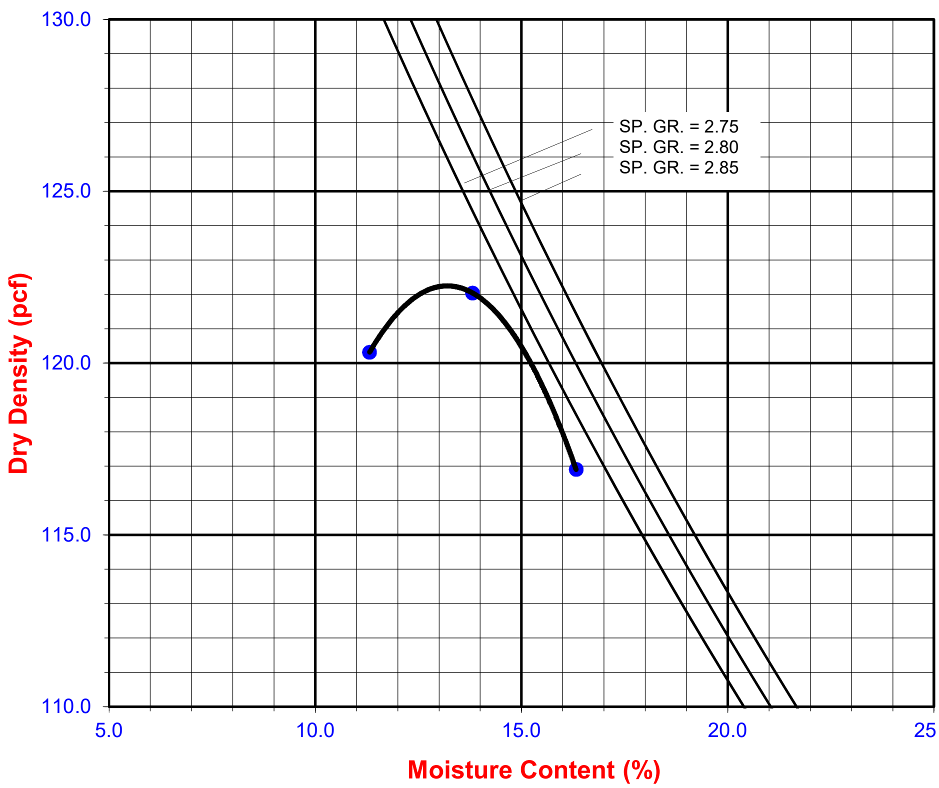
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



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Boring No.	B-2	B-3	B-4	B-5	B-6			
Sample No.	R-3	R-3	R-2	R-4	B-1			
Depth (ft.)	10.0	10.0	5.0	15.0	0-5.0			
Sample Type	Ring	Ring	Ring	Ring	Bulk			
Soil Identification	Brown silty sand (SM)	Brown sandy silt s(ML)	Brown silty sand (SM)	Dark brown silty sand (SM)	Dark brown silty clayey sand with gravel (SC-SM)g			
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00			
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00			
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00			
Sample Dry Weight Determination								
Weight of Sample + Container (g)	903.60	876.20	865.90	497.36	846.87			
Weight of Container (g)	219.40	248.10	236.70	77.31	217.48			
Weight of Dry Sample (g)	684.20	628.10	629.20	420.05	629.39			
Container No.:	610	191	790	936	604			
After Wash								
Method (A or B)	A	A	A	A	A			
Dry Weight of Sample + Cont. (g)	572.70	527.70	632.00	323.72	588.06			
Weight of Container (g)	219.40	248.10	236.70	77.31	217.48			
Dry Weight of Sample (g)	353.30	279.60	395.30	246.41	370.58			
% Passing No. 200 Sieve	**48.4**	**55.5**	**37.2**	**41.3**	**41.1**			
% Retained No. 200 Sieve	51.6	44.5	62.8	58.7	58.9			

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name:	Lewis Pioneer
Project No.:	12604.004
Client Name:	
Tested By:	S. Felter
Date:	04/07/20

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Leighton

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

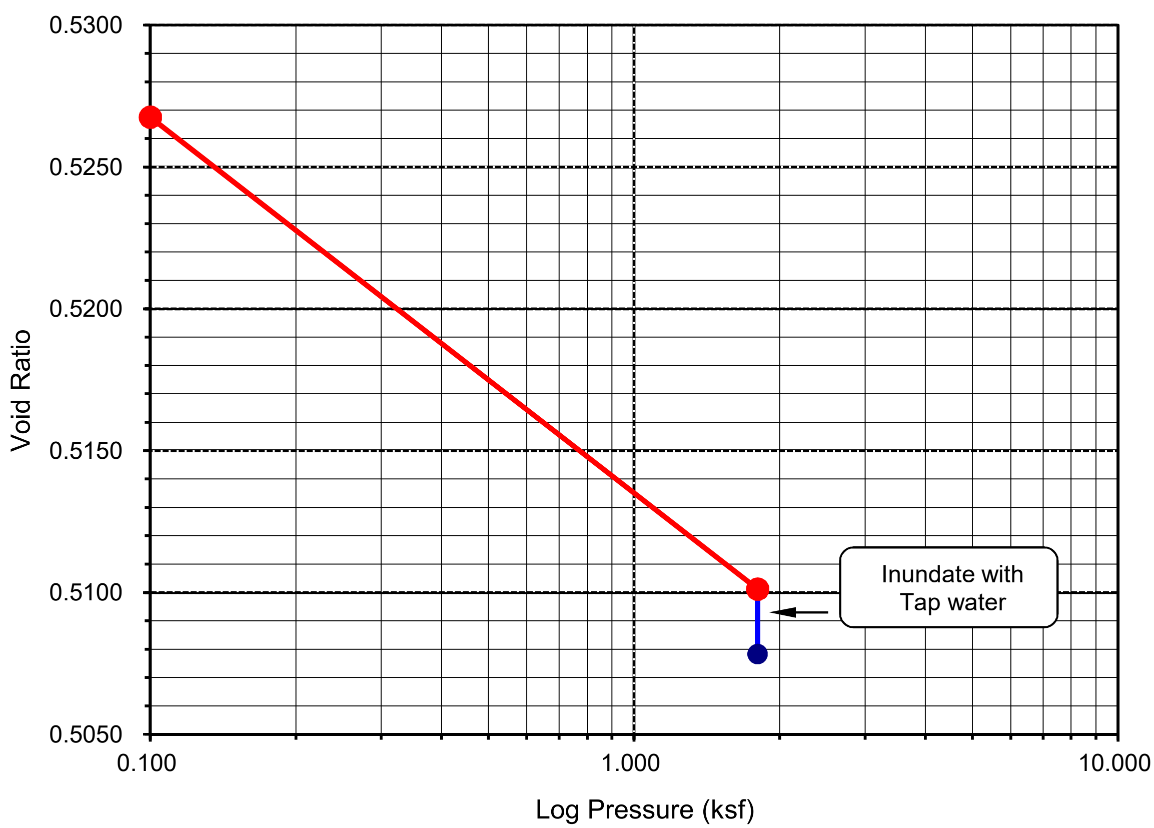
Project Name:	Lewis Pioneer	Tested By:	O. Figeuroa	Date:	04/13/20
Project No.:	12064.004	Checked By:	A. Santos	Date:	04/17/20
Boring No.:	B-5	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	15.0		
Sample Description:	Dark brown silty sand (SM)				

Initial Dry Density (pcf):	110.4	Final Dry Density (pcf):	111.8
Initial Moisture (%):	10.88	Final Moisture (%) :	17.3
Initial Length (in.):	1.0000	Initial Void Ratio:	0.5268
Initial Dial Reading:	0.2862	Specific Gravity(assumed):	2.70
Diameter(in):	2.415	Initial Saturation (%)	55.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2862	1.0000	0.00	0.00	0.5268	0.00
1.800	0.2734	0.9872	0.19	-1.28	0.5101	-1.09
H2O	0.2719	0.9857	0.19	-1.43	0.5078	-1.24

Percent Swell (+) / Settlement (-) After Inundation = -0.15

Void Ratio - Log Pressure Curve



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APPENDIX D

SUMMARY OF SEISMIC ANALYSIS

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Liquefaction Susceptibility Analysis: SPT Method

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

Project: Residential Development (Former Pioneer School) **Leighton**

Project No.: 12064.004

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)	
B-1	200	150	0		-150
B-2	200	150	0		-150
B-3	200	150	0		-150
B-4	200	150	0		-150
B-5	200	150	0		-150
B-6	200	150	0		-150
					0
					0

General Parameters:		
a_{max} =	0.77g	MCE
M_W =	7.7	
MSF eq:	1	(Idriss, 2001)
MSF =	0.93	
Hammer Efficiency =	83	%
C_E =	1.38	
C_B =	1	
$C_{S(SPT)}$ =	1.2	
$C_{S(ring)}$ =	1	
Rod Stickup (feet) =	3	
Ring sample correction =	0.65	

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Summary of Liquefaction Susceptibility Analysis: SPT Method

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

Project: Residential Development (Former Pioneer School)

Project No.: 12064

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement (in.)**
B-1	0 to 4	3	4		20	120	23	2	1	15.0	360	26.4	32.1	>Range	360	0.50	0.53	**NonLiq**	32.1	0.09		0.04	0.2
B-1	4 to 8	5	4		20	120	40	2	1	26.0	600	45.9	53.1	>Range	600	0.50	0.53	**NonLiq**	53.1	0.04		0.02	0.1
B-1	8 to 13	10	5		20	120	63	2	1	41.0	1200	63.5	72.2	>Range	1200	0.49	0.52	**NonLiq**	72.2	0.03		0.02	0.1
B-1	13 to 18	15	5		3	120	41	2	1	26.7	1800	33.7	33.7	>Range	1800	0.48	0.52	**NonLiq**	33.7	0.14		0.08	0.1
B-1	18 to 22	20	5		3	120	70	2	1	45.5	2400	55.8	55.8	>Range	2400	0.48	0.51	**NonLiq**	55.8	0.04		0.02	0.0
B-2	0 to 4	3	4		20	120	5	2	1	3.3	360	5.7	9.8	0.111	360	0.50	0.53	**NonLiq**	9.8	3.26		1.57	2.9
B-2	4 to 8	5	4		20	120	11	2	1	7.2	600	12.6	17.2	0.183	600	0.50	0.53	**NonLiq**	17.2	1.35		0.57	1.4
B-2	8 to 13	10	5		20	120	13	2	1	8.5	1200	13.1	17.8	0.189	1200	0.49	0.52	**NonLiq**	17.8	1.17		0.70	0.8
B-2	13 to 18	15	5		3	120	41	2	1	26.7	1800	33.7	33.7	>Range	1800	0.48	0.52	**NonLiq**	33.7	0.14		0.08	0.1
B-2	18 to 22	20	5		3	120	82	2	1	53.3	2400	65.3	65.3	>Range	2400	0.48	0.51	**NonLiq**	65.3	0.04		0.02	0.0
B-3	0 to 4	3	4		20	120	6	2	1	3.9	360	6.9	11.0	0.122	360	0.50	0.53	**NonLiq**	11.0	1.67		0.80	3.0
B-3	4 to 8	5	4		20	120	11	2	1	7.2	600	12.6	17.2	0.183	600	0.50	0.53	**NonLiq**	17.2	1.35		0.57	2.2
B-3	8 to 13	10	5		20	120	11	2	1	7.2	1200	11.1	15.6	0.166	1200	0.49	0.52	**NonLiq**	15.6	1.29		0.77	1.6
B-3	13 to 18	15	5		5	120	78	2	1	50.7	1800	64.2	64.2	>Range	1800	0.48	0.52	**NonLiq**	64.2	0.03		0.02	0.8
B-3	18 to 23	20	5		3	120	100	2	1	65.0	2400	79.7	79.7	>Range	2400	0.48	0.51	**NonLiq**	79.7	0.03		0.02	0.8
B-3	23 to 28	25	5		3	120	100	1	1.2	120.0	3000	131.6	131.6	>Range	3000	0.47	0.51	**NonLiq**	131.6	0.03		0.02	0.8
B-3	28 to 33	30	5		60	120	32	2	1	20.8	3600	21.9	31.3	>Range	3600	0.47	0.50	**NonLiq**	31.3	0.19		0.11	0.8
B-3	33 to 38	35	5		60	120	16	1	1.2	19.2	4200	18.7	27.5	0.352	4200	0.45	0.48	**NonLiq**	27.5	0.43		0.26	0.7
B-3	38 to 43	40	5		30	120	31	2	1	20.2	4800	18.4	25.9	0.311	4800	0.43	0.46	**NonLiq**	25.9	0.50		0.30	0.4
B-3	43 to 48	45	5		20	120	100	1	1.2	120.0	5400	103.2	115.0	>Range	5400	0.41	0.43	**NonLiq**	115.0	0.02		0.01	0.1
B-3	48 to 52	50	5		20	120	57	2	1	37.1	6000	30.2	36.3	>Range	6000	0.38	0.41	**NonLiq**	36.3	0.20		0.11	0.1
B-4	0 to 4	3	4		20	120	6	2	1	3.9	360	6.9	11.0	0.122	360	0.50	0.53	**NonLiq**	11.0	1.67		0.80	3.7
B-4	4 to 8	5	4		20	120	5	2	1	3.3	600	5.7	9.8	0.111	600	0.50	0.53	**NonLiq**	9.8	3.94		1.65	2.9
B-4	8 to 13	10	5		20	120	25	2	1	16.3	1200	25.2	30.8	>Range	1200	0.49	0.52	**NonLiq**	30.8	0.27		0.16	1.3
B-4	13 to 18	15	5		20	120	10	2	1	6.5	1800	8.2	12.5	0.136	1800	0.48	0.52	**NonLiq**	12.5	1.86		1.11	1.1
B-4	18 to 22	20	5		20	120	100	2	1	65.0	2400	79.7	89.6	>Range	2400	0.48	0.51	**NonLiq**	89.6	0.03		0.02	0.0

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Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
B-5	0 to 4	3	4		20	120	7	2	1	4.6	360	8.0	12.3	0.134	360	0.50	0.53	**NonLiq**	12.3	1.56		0.75	4.1
B-5	4 to 8	5	4		20	120	17	2	1	11.1	600	19.5	24.7	0.285	600	0.50	0.53	**NonLiq**	24.7	0.67		0.28	3.4
B-5	8 to 13	10	5		20	120	12	2	1	7.8	1200	12.1	16.7	0.177	1200	0.49	0.52	**NonLiq**	16.7	1.22		0.73	3.1
B-5	13 to 18	15	5		20	120	7	2	1	4.6	1800	5.8	9.8	0.112	1800	0.48	0.52	**NonLiq**	9.8	3.72		2.23	2.4
B-5	18 to 22	20	5		20	120	33	2	1	21.5	2400	26.3	32.0	>Range	2400	0.48	0.51	**NonLiq**	32.0	0.26		0.14	0.1
B-6	0 to 4	3	4		20	120	9	2	1	5.9	360	10.3	14.8	0.158	360	0.50	0.53	**NonLiq**	14.8	1.36		0.65	1.8
B-6	4 to 8	5	4		40	120	11	2	1	7.2	600	12.6	20.1	0.217	600	0.50	0.53	**NonLiq**	20.1	0.84		0.35	1.2
B-6	8 to 13	10	5		40	120	51	2	1	33.2	1200	51.4	66.7	>Range	1200	0.49	0.52	**NonLiq**	66.7	0.03		0.02	0.8
B-6	13 to 18	15	5		20	120	36	2	1	23.4	1800	29.6	35.6	>Range	1800	0.48	0.52	**NonLiq**	35.6	0.13		0.08	0.8
B-6	18 to 23	20	5		3	120	51	2	1	33.2	2400	40.6	40.6	>Range	2400	0.48	0.51	**NonLiq**	40.6	0.06		0.04	0.7
B-6	23 to 28	25	5		3	120	75	1	1.2	90.0	3000	98.7	98.7	>Range	3000	0.47	0.51	**NonLiq**	98.7	0.03		0.02	0.7
B-6	28 to 33	30	5		3	120	100	2	1	65.0	3600	68.5	68.5	>Range	3600	0.47	0.50	**NonLiq**	68.5	0.03		0.02	0.7
B-6	33 to 38	35	5		3	120	84	1	1.2	100.8	4200	98.3	98.3	>Range	4200	0.45	0.48	**NonLiq**	98.3	0.03		0.02	0.7
B-6	38 to 43	40	5		60	120	30	2	1	19.5	4800	17.8	26.3	0.321	4800	0.43	0.46	**NonLiq**	26.3	0.49		0.29	0.7
B-6	43 to 48	45	5		20	120	38	1	1.2	45.6	5400	39.2	46.0	>Range	5400	0.41	0.43	**NonLiq**	46.0	0.05		0.03	0.4
B-6	48 to 52	50	5		3	120	44	2	1	28.6	6000	23.3	23.3	0.262	6000	0.38	0.41	**NonLiq**	23.3	0.61		0.33	0.3

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Latitude, Longitude: 34.08019, -117.91009

Date	4/16/2020, 2:30:46 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.658	MCE_R ground motion. (for 0.2 second period)
S_1	0.61	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.658	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.105	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	0.702	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.772	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.658	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.81	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.128	Factored deterministic acceleration value. (0.2 second)
S1RT	0.61	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.673	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.662	Factored deterministic acceleration value. (1.0 second)
PGAd	0.851	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.916	Mapped value of the risk coefficient at short periods
C_{R1}	0.907	Mapped value of the risk coefficient at a period of 1 s

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Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Latitude

Decimal degrees

34.08019

Longitude

Decimal degrees, negative values for western longitudes

-117.91009

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

Return period in years

2475

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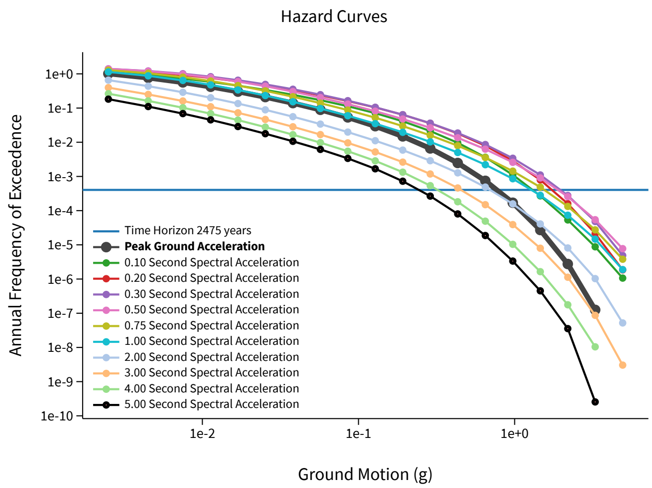
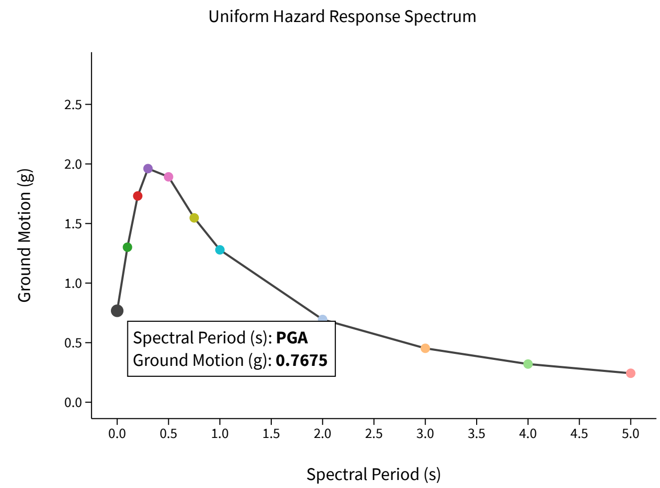
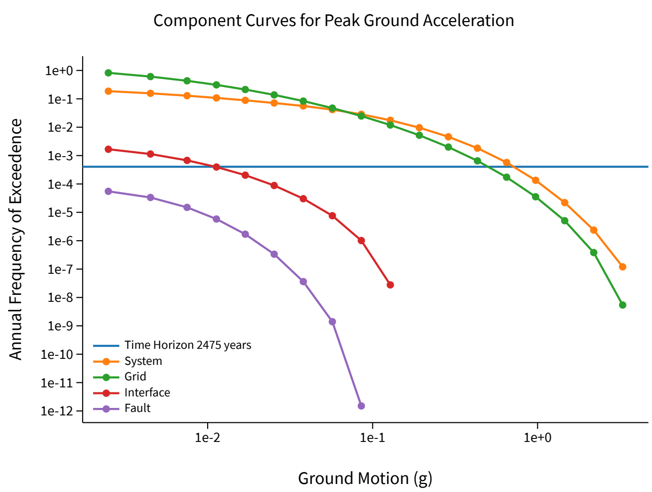
Hazard Curve







[View Raw Data]()

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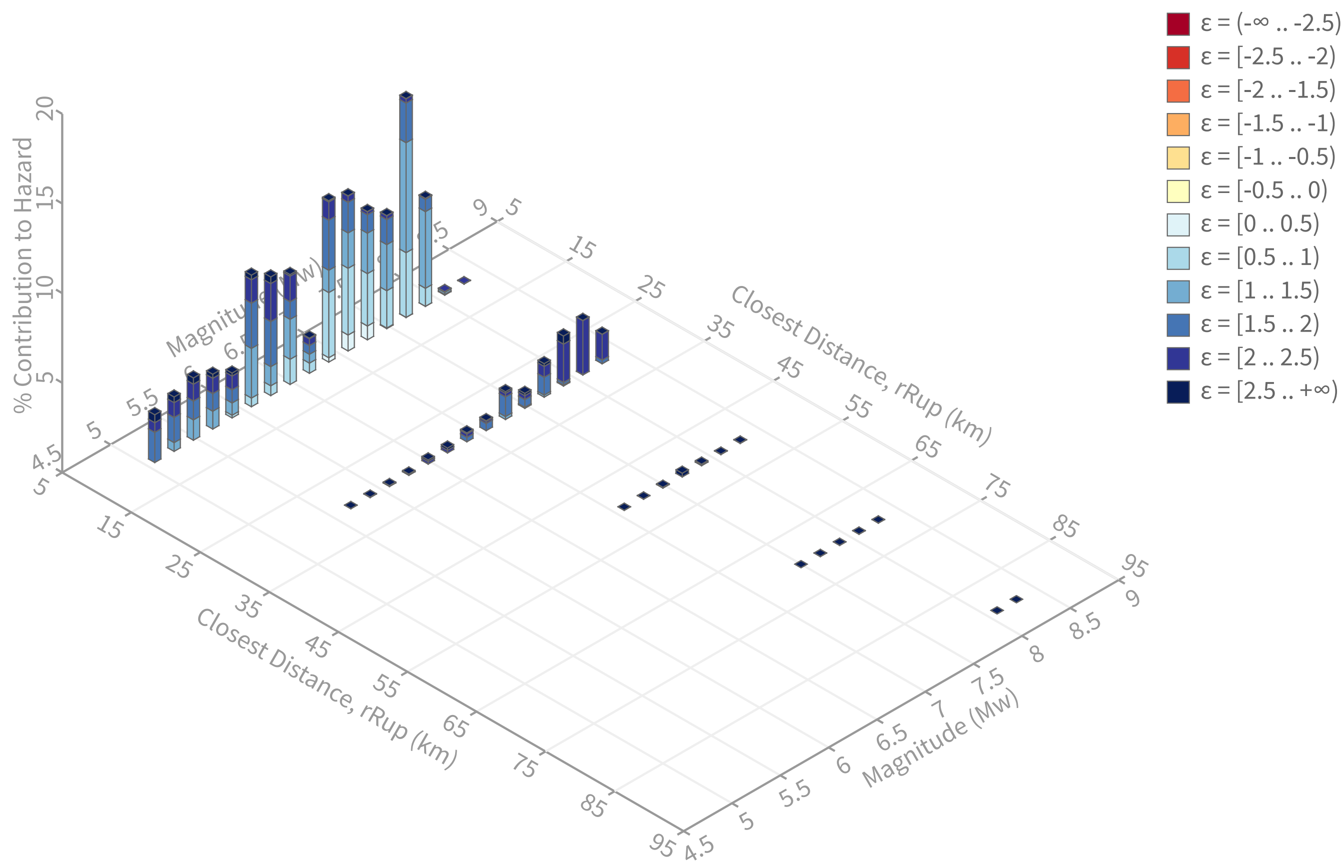
^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.76751153 g

Recovered targets

Return period: 2953.4633 yrs
Exceedance rate: 0.00033858555 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.11 %

Mean (over all sources)

m: 6.93
r: 13.35 km
ε_0: 1.5 σ

Mode (largest m-r bin)

m: 7.72
r: 11.75 km
ε_0: 1.19 σ
Contribution: 12.2 %

Mode (largest m-r-ε_0 bin)

m: 7.72
r: 11.53 km
ε_0: 1.25 σ
Contribution: 6.13 %

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 .. +∞]

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Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							39.63
San Jose [2]		5.18	6.96	0.77	117.881°W	34.043°N	147.08	6.19
Sierra Madre [2]		8.73	7.67	1.17	117.903°W	34.157°N	4.23	6.16
Puente Hills (Coyote Hills) [0]		11.92	7.26	0.90	117.868°W	33.919°N	167.79	5.42
Whittier alt 2 [5]		13.48	7.27	1.52	117.963°W	33.966°N	200.99	3.78
San Andreas (Mojave S) [12]		38.50	8.05	2.27	117.720°W	34.389°N	26.88	3.73
Richfield [1]		13.64	6.17	1.51	117.870°W	33.882°N	170.48	3.06
Raymond [0]		12.16	7.13	1.66	117.991°W	34.166°N	322.01	2.53
Puente Hills (LA) [0]		17.66	7.17	1.70	118.116°W	33.990°N	242.34	1.39
Chino alt 2 [0]		15.56	6.84	2.01	117.751°W	34.030°N	110.83	1.27
Compton [0]		22.52	7.37	1.72	118.112°W	33.746°N	206.67	1.21
UC33brAvg_FM31	System							38.46
San Jose [2]		5.18	6.96	0.77	117.881°W	34.043°N	147.08	6.29
Sierra Madre [2]		8.73	7.66	1.17	117.903°W	34.157°N	4.23	6.17
Puente Hills [0]		11.96	7.43	0.86	117.914°W	33.943°N	181.34	5.25
San Andreas (Mojave S) [12]		38.50	8.05	2.27	117.720°W	34.389°N	26.88	3.72
Whittier alt 1 [6]		13.12	6.85	1.69	117.961°W	33.966°N	200.22	3.64
Raymond [0]		12.16	7.12	1.67	117.991°W	34.166°N	322.01	2.33
Chino alt 1 [0]		14.96	6.47	2.15	117.752°W	34.028°N	111.82	1.58
Puente Hills [1]		11.97	7.09	0.87	117.957°W	33.944°N	195.78	1.09
Compton [0]		22.52	7.27	1.77	118.112°W	33.746°N	206.67	1.04
UC33brAvg_FM31 (opt)	Grid							10.98
PointSourceFinite: -117.910, 34.103		5.63	5.67	1.40	117.910°W	34.103°N	0.00	2.83
PointSourceFinite: -117.910, 34.103		5.63	5.67	1.40	117.910°W	34.103°N	0.00	2.83
PointSourceFinite: -117.910, 34.157		9.45	5.74	1.94	117.910°W	34.157°N	0.00	1.39
PointSourceFinite: -117.910, 34.157		9.45	5.74	1.94	117.910°W	34.157°N	0.00	1.39
UC33brAvg_FM32 (opt)	Grid							10.93
PointSourceFinite: -117.910, 34.103		5.64	5.67	1.40	117.910°W	34.103°N	0.00	2.76
PointSourceFinite: -117.910, 34.103		5.64	5.67	1.40	117.910°W	34.103°N	0.00	2.76
PointSourceFinite: -117.910, 34.157		9.36	5.78	1.92	117.910°W	34.157°N	0.00	1.38
PointSourceFinite: -117.910, 34.157		9.36	5.78	1.92	117.910°W	34.157°N	0.00	1.38

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APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

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

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

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

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

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

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

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

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

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

2.0 Preparation of Areas to be Filled

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

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

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

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

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

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

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

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

3.0 Fill Material

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

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

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

4.0 Fill Placement and Compaction

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

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

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

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

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

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

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

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

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

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

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