GEOTECHNICAL INVESTIGATION
PROPOSED FIRE STATION 1 (SP-4071),
105 SOUTH WATER STREET,
CITY OF ORANGE, CALIFORNIA

Prepared For:

WLC ARCHITECTS, INC.
8163 Rochester Avenue, Suite 100
Rancho Cucamonga, California 91730

Project No. 12482.001

September 26, 2019

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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To: WLC Architects, Inc.
8163 Rochester Avenue, Suite 100
Rancho Cucamonga, California 91730

Attention: Mr. Kelley Needham

Subject: Geotechnical Investigation, Proposed Fire Station 1 (SP-4071), 105 South Water Street, City of Orange, California

In accordance with our proposal dated March 22, 2019 and your authorization on July 28, 2019, Leighton Consulting, Inc. (Leighton) has conducted a geotechnical investigation for the proposed Fire Station 1 facility (SP-4071), located at 105 South Water Street in the City of Orange, California. The purpose of this study has been to evaluate the subsurface conditions at the site with respect to the proposed fire station development and to provide geotechnical recommendations for design and construction.

Based on this investigation, the proposed development of the fire station is feasible from a geotechnical standpoint. Significant geotechnical issues for this project are those related to the potential for strong seismic shaking and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project.

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

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Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Location and Description

The site contains two (2) parcels located at the southeast corner of Chapman Avenue and South Water Street and at the northwest corner of Almond Avenue and South Water Street, in Orange, California. The site previously contained several buildings and appears to have been vacant since early 2010. Concrete slabs were observed in the northern and western regions with asphalt paved areas located throughout the northern parcel. The site is surrounded by office buildings and single-family residential homes to the north, east and west. The City of Orange Water Division Department of Public Works is located to the south. The parcel located northwest of South Water Street and Almond Avenue is currently occupied by a car dealership lot.

The site and surroundings are relatively flat, with site elevations ranging from about 214 to 219 feet above mean sea level, with drainage to the south. The site location (latitude 33.7873°, longitude -117.8411°) and immediate vicinity are shown on Figure 1, Site Location Map.

1.2 Proposed Improvements

Based on our review of the proposed site plan *Fire Station 1 Headquarters City of Orange Fire Department 105 South Water Street, Orange, CA,* prepared by WLC Architects dated July 30, 2019, the proposed fire station development includes a headquarters/administration building on the western portion and a separate reserve apparatus building on the southeast portion of the site.

We understand that the site will be designed in stages such that the City has the option to construct an operational Fire Station 1 first and the Fire Headquarters building portion added at a later date. The proposed fire station facility is composed of a two-story, approximate 24,300-square-foot building, of which approximately 5,700 square feet make up the main apparatus building portion.

Additional overflow parking will be constructed on the existing site located northwest of the intersection of Almond Avenue and South Water Street. We assume that remedial cuts and fills of 5 feet or less with localized deeper excavations to remove undocumented fill will be required to attain finish grades for the new structures.

1.3 Purpose of Exploration

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

Our geotechnical exploration included hollow-stem auger soil borings, laboratory testing and geotechnical analysis to evaluate existing conditions and develop the recommendations contained in this report. Infiltration testing was conducted to evaluate general infiltration characteristics at the locations and depths tested to support infiltration system design by the civil engineer.

1.4 Scope of Investigation

The scope of our study has included the following tasks:

- Geologic Hazards Review: We reviewed pertinent, readily available geologic and geotechnical literature covering the site. Our review included regional geologic maps and reports available from our in-house library. Key documents reviewed are referenced in Appendix A, *References*.

- Utility Coordination: We contacted Underground Service Alert (USA) prior to our subsurface exploration to have underground utilities located and marked.

- Field Exploration: Our field investigation included drilling, logging, and sampling of five (5) hollow-stem auger borings (LB-1 through LB-5) at representative locations in the areas of the proposed building to depths ranging from approximately 6 feet to 51.5 feet below the existing ground surface (bgs). Additionally, two hollow-stem auger borings (LB-6 and LB-7) were drilled, logged, and sampled in the area of the proposed overflow parking lot to depths of approximately 9 feet bgs. Encountered earth materials were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using a California Ring Sampler. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained from the SPT split-spoon sampler. Representative bulk samples were also collected at shallow depths from the borings.

Two infiltration tests were conducted within borings LB-4 and LB-5 to evaluate general infiltration rates of the subsurface soils with bottom depths of 14 feet bgs and 20.5 feet bgs.

All excavations were backfilled with the soil cuttings. An asphalt concrete patch was placed at the top of LB-6 and LB-7 to match the existing ground surface. Logs of the geotechnical borings are presented in Appendix B, *Exploration Logs*. Approximate boring locations are shown on Figure 2, *Exploration Location Map*.

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

  - In situ moisture content and dry density
  - Proctor Compaction Test
  - Sieve analysis
  - Atterberg Limits
  - Expansion Index
  - Water-soluble sulfate concentration in the soil
  - Resistivity, chloride content and pH

  The in situ moisture content and dry density test results are shown on the boring logs in Appendix B. The other laboratory test results are presented in Appendix C, *Geotechnical Laboratory Test Results*.

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

- Report Preparation: Results of our geotechnical investigation have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed Fire Station development as currently planned.

2.0 FINDINGS

2.1 Geologic Hazards Review

We have reviewed pertinent, readily available geologic and geotechnical literature covering the site. Our review included regional geologic maps and reports available from our library. Documents reviewed are listed in Appendix A, *References*. Potential geologic hazards are discussed in the following sections. Our review has considered California Geological Survey's Note 48, *Checklist of the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*.

2.1.1 Site History

Our review of site history included analysis of historical topographic maps between the dates of 1896 and 2015 and historical aerial photographs between the dates of 1946 and 2016. The purpose of this evaluation was to help understand the origin of the current site profile, former site use, as well as past grading activities.

In its original undeveloped state, up until early 1930, the properties consisted of gently southerly sloping terrain, with the Santiago Creek drainage channel situated approximately 0.20 mile to the east. Between approximately 1940 and 1963, both properties were utilized for agricultural purposes with the western area of proposed parking overflow as orchard and the proposed fire station site as buildings likely associated with the agriculture activities.

While the overall use of the buildings and foundation elements are unknown, structures onsite were not observed in 2010 aerial imagery. It is unknown if all foundation elements were removed and should be anticipated in the subsurface during grading of the site.

2.2 Regional Geologic Conditions

The project site is located in the western part of the Tustin Plain within the Peninsular Ranges geomorphic province west of Santiago Creek drainage. The Peninsular Ranges geomorphic province extends 900 miles southward from the Los Angeles Basin to the tip of Baja California (Yerkes et al., 1965) and is characterized by elongate northwest-trending mountain ranges separated by sediment-floored valleys. The most dominant structural features of the province are the northwest-trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province.

East of the site are the northwest-trending Santa Ana Mountains, a large range which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea. Sediments eroded from the Santa Ana Mountains have been transported by Santiago Creek and the lower reach of the Santa Ana River to build a large, broad alluvial fan known as the Tustin Plain. The Tustin Plain is comprised of relatively flat-lying, unconsolidated to semi-consolidated clastic sediments that are approximately 1,000 to 1,100 feet thick (Singer, 1973; Sprotte et al., 1980a and 1980b). Beneath the site, the near surface, unconsolidated, relatively fine-grained sediments are Holocene age (less than 11,000 years old) and consist of predominately youthful alluvial fan deposits (Sprotte et al., 1980a and 1980b). These sediments in turn are underlain at depth by sedimentary bedrock of Tertiary age.

The surficial geologic units mapped in the vicinity of the site are shown on Figure 3, *Regional Geology Map*.

2.3 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by undocumented fill (Map Symbol: Afu) in the upper five to seven feet, localized deeper fill to seven feet below grade was interpreted in boring LB-5 due to the presence of fresh, mechanically fractured black slaty gravels and cobble size rock fragments. Review of historic aerial imagery indicates former structures were onsite until circa 2010. Foundation elements should be anticipated in the subsurface during grading of the site. Refusal at shallow depth in boring LB-2 was encountered which is within the footprint of a historical structure formerly located onsite (NETR, 2019). The artificial fill is underlain by Quaternary-age old alluvial fan deposits (Map Symbol: Qof) extending to the maximum exploration depth of 51 feet bgs. The overlying undocumented fill (Afu) encountered within our excavations generally consisted of a loose to dense silty sand and sand with gravel and small mechanically fractured cobbles. The native soils (Qof) were generally composed of slightly moist to moist, dense to very dense, well-graded gravel with sand and silt, sand with gravel, and silty sand with small weathered cobbles derived from the sedimentary formations in the Santa Ana Mountains. The in-situ moisture content within the upper approximately 15 feet generally ranged from 2 to 7 percent. More detailed descriptions of the subsurface soil are presented on the boring logs in Appendix B.

2.3.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge or a new structure. Based on our observations and the laboratory test results, the native soil encountered is generally considered slightly compressible. Removal and recompaction of this material under shallow foundations is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential (moisture sensitivity, sometimes referred to as 'hydrocollapse') refers to the potential settlement of a soil under existing stresses upon being wetted. Based upon the dense nature of encountered sands and gravel, the hydrocollapse potential of the onsite soil is expected to be very low.

2.3.2 Expansive Soils

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

A near-surface soil sample from the proposed fire station building area was tested for expansion index. The results of the tests indicated soil with very low expansion potential. Based on these test results, the near surface soil is expected to have a very low expansion potential. The results of the expansion testing are included in Appendix C of this report

2.3.3 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 2016 CBC (CBC, 2016, Chapter 19; and ACI, 2008).

A near-surface soil sample was tested for soluble sulfate content. The result of this test indicated a sulfate content of less than 0.02 percent by weight, indicating negligible sulfate exposure. As such, the soils exposed at pad grade are not expected to pose a significant potential for sulfate reaction with concrete. The results of the chemical analyses are included in Appendix C of this report

2.3.4 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 between 1,000 and 2,000 ohm-cm is considered corrosive, and soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a soil sample was tested during this investigation to determine its minimum resistivity, chloride content, and pH. These tests indicated a minimum resistivity of 1,570 ohm-cm, chloride content of 187 ppm, and pH of 7.1. Based on these results, the onsite soil is considered corrosive to ferrous metals. The results of the chemical analyses are included in Appendix C of this report.

2.4 Groundwater

Groundwater was not encountered in our borings excavated to a maximum depth of 51 feet below the existing ground surface (bgs). The historical high groundwater level in the area was estimated to have been on the order of 172 feet bgs in State Well 04S09W33M001S, located 0.6 miles southeast of the site (CDWR, 2019). The California Geological Survey (1997) Seismic Hazard Zone Report for this region shows the site area as not having historically shallow groundwater levels (greater than 40 feet bgs). Based on this, groundwater has historically been deep, and shallow groundwater is not expected at the site.

Fluctuations of the groundwater level and localized zones of perched water should be anticipated below grade during and following the rainy season. Irrigation of landscaped areas and infiltration of groundwater can also cause a fluctuation of local groundwater levels and may create temporary zones of perched water.

2.5 Faulting and Seismicity

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

2.5.1 Surface Faulting

One of the primary seismic hazards for this region is surface fault rupture. Our assessment of the possible presence of active faulting through the proposed improvement project site included a review of available literature, maps, and aerial photographs.

Our review of available in-house literature indicates that there are no known active faults traversing the site and the site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. Therefore, the potential risk for surface fault rupture through the site is considered low.

The closest known active or potentially active faults are the Elysian Park Blind Thrust and the Puente Hills Blind Thrust fault systems located approximately 9 miles northwest of the project site. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include the Whittier-Elsinore, San Andreas, Sierra Madre, San Jacinto, Newport-Inglewood, Raymond, Puente Hills, Verdugo-Eagle Rock, Elysian Park and Norwalk faults. Active faults within a 60-mile radius from the site are listed in Appendix D.

2.5.2 Seismic Design Parameters

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California, see Figure 4, *Regional Fault and Historical Seismicity Map*. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. Accordingly, design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). The 2016 edition of the California Building Code (CBC) is the current edition of the code. Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced. A summary of the analysis is provided in Appendix D, *Seismic Analysis.*

The following code-based seismic parameters should be considered for design under the 2016 CBC:

Table 1 - 2016 CBC Seismic Design Parameters

Description (2016 CBC reference)	Parameter	Design Value
Site Latitude, degrees		33.7873
Site Longitude, degrees		-117.8411
Site Class Definition (1613A.3.2)		D
Mapped MCE Spect Resp Accel at 0.2s for (Fig 1613.3.1(1)), using USGS	S_S	1.5
Mapped MCE Spect Resp Accel at 1.0s for (Fig 1613.3.1(2)) using USGS	S_1	0.549
Short Period Site Coefficient (Table 1613A.3.3(1))	F_a	1.0
Long Period Site Coefficient (Table 1613A.3.3(2))	F_v	1.5
Adjusted MCE Spectral Response Acceleration at 0.2s Period [=F_aS_S] (Eq. 16-37)	S_{MS}	1.5
Adjusted MCE Spectral Response Acceleration at 1s Period [=F_vS_1] (Eq. 16-38)	S_{M1}	0.823
Design Spectral Response Acceleration at 0.2s Period, 5% damped [=2/3S_{MS}] (Eq. 16-39)	S_{DS}	1.0
Design Spectral Response Acceleration at 1s Period, 5% damped [=2/3S_{M1}] (Eq 16-40)	S_{D1}	0.549
Is S_1 greater than or equal to 0.75?		No
Seismic Design Category [="D" if S_1<0.75] (1613A.2.5)		D

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2.5.3 Seismic Parameters for Geotechnical Evaluation

Based on ASCE 7-10 Equation 11.8-1, the F_{PGA} is 1.0, the PGA is 0.515g, and the PGA_M is 0.51g. This is the value used for seismic analysis of the onsite soils. As an added check, PGA and hazard deaggregation were also estimated using the United States Geological Survey's (USGS) 2008 Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a PGA of 0.58g with magnitude of approximately 6.9 (M_W) at a distance on the order of 12.8 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years); 2/3 of this value is 0.39g. Results are included in Appendix D. This is not an exhaustive site-specific analysis, yet is useful in evaluating the general seismic potential at the site as an added check.

2.5.4 Historical Seismicity

Figure 4, *Regional Fault and Historical Seismicity Map* shows recorded historical regional seismic events (those that have been recorded since the mid 1700s) with respect to the site. Based on this map, it appears that the site has been exposed to relatively significant seismic events; however, this site does not appear to have experienced more severe seismicity than compared to much of southern California in general. We are unaware of documentation indicating that past earthquake damage in the site vicinity has been significantly worse than for the majority of southern California. In addition, we are unaware of damage in the site vicinity as the result of liquefaction, lateral spreading, or other related phenomenon.

We also performed an evaluation of site historical seismicity with respect to significant past earthquakes (those recorded from the 1800s with magnitudes 5 or greater) using the EQSEARCH computer program (Blake, 2011; see Appendix D). This is a relatively simple analysis, based on epicenters, and does not include more complex characteristics of earthquakes, such as rupture length and direction; however, it gives an idea of past seismicity at the site. This analysis suggests that the largest ground acceleration at the site generated from the magnitude 6.3Mw 1933 Long Beach Earthquake along the Newport Inglewood Fault is estimated to have been roughly 0.16g.

2.6 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.

2.6.1 Liquefaction Potential

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

As shown on the Seismic Hazard Zones map for the Orange Quadrangle (CGS, 1998), the project site is **not** located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*).

We have evaluated liquefaction potential of the soil encountered in our borings assuming a historic high groundwater depth deeper than 50 feet. Our analysis was based on the modified Seed Simplified Procedure as detailed by Youd et al. (2001) and Martin and Lew (1999), which compares the seismic demand on a soil layer (Cyclic Stress Ratio, or CSR) to the capacity of the soil to resist liquefaction (Cyclic Resistance Ratio, or CRR), (Youd et al, 2001). A minimum required factor of safety of 1.3 was used in our analysis, with factor of safety defined as CRR/CSR. As required, our analysis assumes that the design earthquake would occur while the groundwater is at its estimated historically highest level. In the SPT method, soil resistance to liquefaction is estimated based on several factors, including SPT sampling blow counts normalized and corrected for several factors including fines content, and overburden pressure. Soil plasticity and moisture content are also considered in an evaluation of liquefaction. Parameters utilized in our analysis include Standard Penetration Test (SPT) results from the borings, visual descriptions of soil samples retrieved, and geotechnical laboratory test results.

Based on our analysis, the nature of the onsite soils, and the historically deep groundwater level, the potential for liquefaction at the site is considered very low.

2.6.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. 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 (1987), and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). The results of our analyses suggest that the onsite soils are susceptible to less than an 1-inch of seismic settlement based on the MCE. Differential settlement due to seismic loading is assumed to be less than ½ inch over a horizontal distance of 40 feet based on the MCE. A summary of seismic settlement analysis is included in Appendix D.

2.6.3 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the location of the site and distance from contained water facilities, seiches and tsunamis are not a hazard to the site.

2.7 Slope Stability and Landslides

The potential for seismically induced landsliding to occur at the site is considered low due to the absence of slopes at the site. In addition, based on review of the

Seismic Hazard Zones Map for the Orange Quadrangle (CGS, 1998), the site is **not** located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (Figure 5, *Seismic Hazard Map*). Proposed slopes, while not anticipated, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

2.8 Flooding and Dam Inundation Potential

The site is not located within the 100-year or 500-year flood plain based on the Federal Emergency Management Agency (FEMA) flood maps (see Figure 7, *Flood Hazard Zone Map*).

Flooding can also result from the failure of dams. Based on our review of dam inundation data by the California Office of Emergency Services (OES), the site is not located near dams or in an area shown as susceptible to dam inundation, see Figure 6, *Dam Inundation Map.*

2.9 Infiltration Testing

Infiltration tests was conducted in two of the excavated borings (LB-4 and LB-5) to estimate the infiltration rate of the onsite soils at the depths tested. The infiltration test was conducted at bottom depths of approximately 14 and 20.5 feet below the existing ground surface.

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 or gravel is placed in the boring bottom to support temporary perforated well casing pipe and a float valve. In addition, coarse sand is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float valve, lowered into the boring inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a relatively constant water head in the boring. The incremental infiltration rate as measured during intervals of the test is defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface. The test was conducted based on the USBR 7300-89 test method.

Raw infiltration rates for the well permeameter tests may be assumed to be about 4.5 in/hour within the gravel layer generally encountered at a depth of 15 to 20 feet bgs, but should be considered negligible in the clayey sand layer at a depth of approximately 10 feet in boring LB-4. These are raw values and do not include a factor of safety or correction. Results of infiltration testing are provided in Appendix B. Further discussion on infiltration testing and recommendations are included in Section 3.9.

2.10 Other Potential Hazards Listed on CGS Note 48

The following naturally occurring hazards are not believed to exist at the site nor in the region: methane gas, hydrogen-sulfide gas, tar seeps, volcanic eruption, radon-22 gas, and naturally occurring asbestos in geologic formations associated with serpentine.

We are unaware of significant subsidence or damage from subsidence near the site due to groundwater withdrawal.

3.0 CONCLUSIONS AND RECOMMENDATIONS

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

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, General Earthwork Recommendations, 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, as should trees and their root systems. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

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

3.1.2 Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. For the proposed fire station building and apparatus building constructed with shallow foundations, we recommend that onsite soils be overexcavated and recompacted to a minimum depth of 2 feet below the bottom of the proposed footings or 5 feet below existing grade, whichever is deeper. In addition, existing undocumented artificial fill in structural areas should be removed to undisturbed native alluvial soil. Where feasible, overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of the proposed footings, or a distance equal to the depth of overexcavation, whichever is greater.

Local conditions, such as those interpreted in boring LB-5 may require that deeper overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and areas to receive fill should be overexcavated to a minimum depth of 18 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper. Overexcavation for site walls should extend a minimum 2 feet below the bottom of the wall footings.

All excavation or removal bottoms should be observed by a representative of the geotechnical engineer prior to placement of fill or other improvements to determine that geotechnically suitable soil is exposed. The overexcavation in the building area may also require observation by the City Grading Inspector prior to fill placement. Excavation bottoms observed to be suitable for fill placement or other improvements should be scarified to a depth of at least 8 inches, moisture-conditioned as necessary to achieve a moisture content approximately 2 to 3 percentage points above the optimum moisture content, and then compacted to a minimum of 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor).

Once final development plans are completed and building loads have been calculated this information should be provided to Leighton for geotechnical review to ensure our recommendations have been properly interpreted and remain appropriate for the project as currently proposed.

3.1.3 Fill Placement and Compaction

The onsite soil is geotechnically suitable for use as compacted structural fill, provided it is free of debris and oversized material (cobbles) (greater than 6 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

Based upon the anticipated conceptual plan, site grading is not expected to require significant cut or fill; however, excavations as deep as 5 to 6 feet with localized deeper excavation should be expected for the removal and reworking of all undocumented fill and overexcavation of building foundations. All fill soil should be placed in thin, loose lifts, moisture-conditioned as necessary to achieve a moisture content approximately 2 to 3 percentage points above the optimum moisture content, and then compacted to a minimum of 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor). Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 Import Fill Soil

If import soil is to be placed as fill, it 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, environmental unsuitability etc.

3.1.5 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered, sampling blow counts, and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage and Subsidence	
Shrinkage	Approximately 10 +/- 3 percent
Subsidence (overexcavation bottom processing)	Approximately 0.1 foot

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

3.2 Foundation Recommendations

The following recommendations are based on soils with a very low expansion potential. The structural engineer should design the footing reinforcement in accordance with current California Building Code (CBC) requirements. Local agencies, the structural engineer or the CBC may have requirements that are more stringent.

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1.2.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment depth and width per the 2016 CBC. These minimums include a depth and width of 12 inches.

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 200 psf per foot increase in depth or width to a maximum allowable bearing pressure of 4,000 psf. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.40. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The maximum passive resistance should not exceed 3,500 psf. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

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

3.2.5 Settlement Estimates

The recommended allowable bearing capacity 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 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a very low expansion potential. Laboratory testing should be conducted at finish grade to evaluate the Expansion Index (EI) of near-surface subgrade soils. 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. Slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 3 percentage points above optimum moisture content to a minimum depth of 18 inches prior to placing steel or concrete.

- Concrete Thickness: Thickness of 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. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Crack control joints should be placed at 13 feet on center or less, forming approximately square panels.

  For the apparatus bay, the slab should be a minimum of 8 inches thick and underlain by 6 inches of aggregate base. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Construction joints should be designed by the structural engineer, but should be spaced no more than 13 feet on center, forming square sections.

- Moisture Vapor Retarder: We recommend a minimum of a 15-mil vapor retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. Since moisture will otherwise be transmitted up from the soil through the concrete, it is important that an intact vapor retarder be installed. We recommend that the vapor retarder intended for the specific conditions present be used and meet the requirements of ASTM E1745 and installed per ASTM E1643. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether or not a sand blotter layer should be placed over the vapor retarder. If sand is placed on top of the vapor retarder, the contractor should not allow the sand to become wet prior to concrete placement (e.g., sand should not be placed if rain is expected). Sharp objects, such as gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a stronger vapor retarder intended for the specific conditions present can be used. *Mechanically fractured gravel and small cobbles observed during drilling and sampling resulted in angular sharp fragments that could puncture the barrier.*

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, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

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

Leighton does not practice in the field of moisture vapor transmission recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person (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 most recent edition of the California Building Code (CBC). The seismic design parameters listed in Table 1 of Section 0 of this report should be considered for the seismic analysis of the subject site.

3.5 Lateral Earth Pressures

The following retaining wall recommendations are included for design consideration of walls with a height less than 6 feet. 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 8, *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. Retaining wall locations and configurations are unknown at the time of this report.

Table 2 - Retaining Wall Design Parameters

Static Equivalent Fluid Pressure (pcf)	
Condition	Level Backfill
Active	40
At-Rest (drained, compacted-fill backfill)	60
Passive (ultimate)	360 (Max. 5,000 psf)

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 0.40 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time. 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 at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

We recommend that the wall designs for walls 6 feet tall or taller be checked seismically using an *additive seismic* Equivalent Fluid Pressure (EFP) of 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 200 psf per foot increase in width or depth to a maximum allowable bearing pressure of 4,000 psf.

3.6 Cement Type and Corrosion Protection

Based on the results of laboratory testing (Appendix C), concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. Concrete should be designed in accordance with ACI 318-14, Section 4.2 (ACI, 2014), adopted by the 2016 CBC (Section 1904A.2).

Based on our laboratory testing, the onsite soil is considered corrosive to ferrous metals. Metallic utilities should be avoided, or typical corrosion protection of underground metallic utilities should be considered. Corrosion information presented in this report should be provided to your underground utility contractors.

3.7 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 40 for compacted silty sand subgrade soils, preliminary flexible pavement sections may consist of the following for the Traffic Indices (TI) indicated.

Table 3 – Hot Mix Asphalt (HMA) Pavement Sections

Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base (AB) Thickness (inches)
5 or less (auto access)	3.0	4.0
7 (truck access)	4.0	4.0

For fire truck (60,000-pound “apparatus”) lanes, asphalt pavements designed for a TI=7 are recommended. However, note that undisturbed apparatus outrigger loads could cause local asphalt pavement punching damage. When possible, outrigger loads should be distributed over asphalt pavements with planks and plywood. Otherwise, areas where outrigger loads are anticipated could be paved with 8-inch-thick concrete as described below.

Portland cement concrete pavement sections were calculated in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for three Traffic Indices (TIs) are presented below.

Table 4 – Portland Cement Concrete Pavement Sections

Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base (AB) Thickness (inches)
5 or less (auto access)	6.0	4.0
7 (truck access)	8.0	6.0

We have assumed that this Portland cement concrete will have a compressive strength of at least 4,000 psi. Reinforcement should be specified by the structural engineer, but should be a minimum of #3 rebar at 18 inches on center each way. The PCC pavement sections should be provided with crack-control joints spaced no more than 13 feet on center each way. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that sections be as nearly square as possible.

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 8 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 Rate: We recommend an unfactored (small-scale) infiltration rate of 4.5 inches per hour be used for preliminary design for an infiltration system designed at a depth of 15 to 20 feet below the existing grade within the natural gravel layer. The infiltration chamber may be deepened by excavating trenches in the bottom of the infiltration chamber excavation for the length of the excavation, and backfilling these trenches with ASTM C33 Fine Aggregate (washed concrete sand). Leighton should observe the soil in the excavation to confirm these recommendations.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with the Orange County guidelines, 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 2 for buried chambers, and at least 3 for open basins or for conditions where retained water will be exposed to the open atmosphere, 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 infiltration facility 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 specific locations and invert depths of proposed facilities. Further testing may be needed based on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations: The periodic flow of water carrying sediments into the infiltration facility, plus the introduction of wind-blown sediments and sediments from erosion of basin side walls, can eventually cause the bottom of the facility to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate. 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 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 chambers.

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.

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. Contractors should be advised that sand and gravelly fill soils should be considered Type C soils as defined in the California Construction Safety Orders.

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

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

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

3.10 Trench Backfill

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

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

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

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment.

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

3.11 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems. Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

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

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

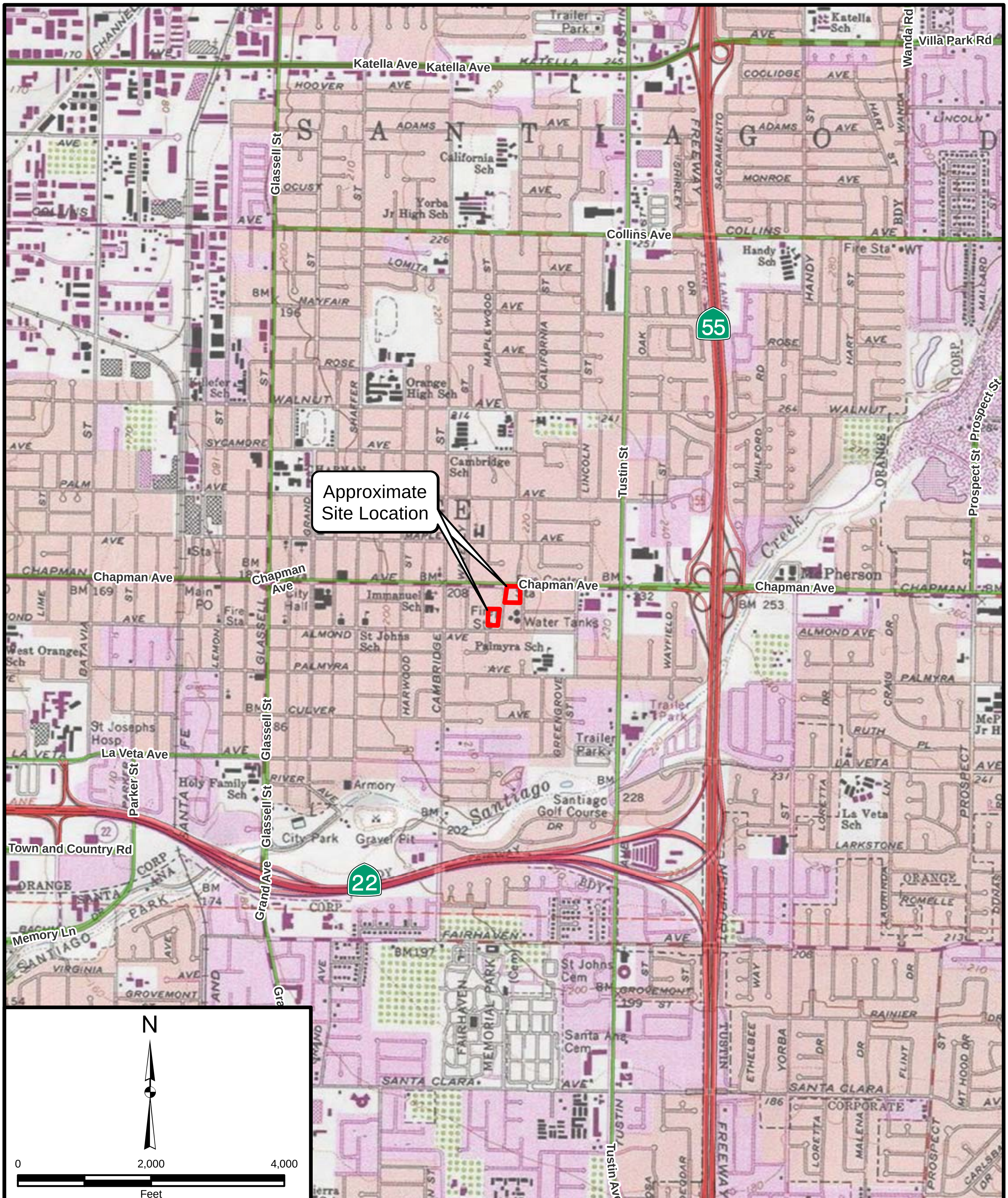
Geotechnical observation and testing should be provided:

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

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton Consulting, Inc. will provide geotechnical observation and testing during construction. Please refer to the GBA “Important Information about This Geotechnical Engineering Report” presented on at the end of this report.

This report was prepared for the sole use of WLC Architects, Inc. for application to the design of the proposed City of Orange Fire Station 1 in accordance with generally accepted geotechnical engineering practices at this time in California.



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Scale:1 " = 2,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: (kmanchikanti)

SITE LOCATION MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 1

Leighton

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Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: (kmanc)

EXPLORATION LOCATION MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 2

Leighton

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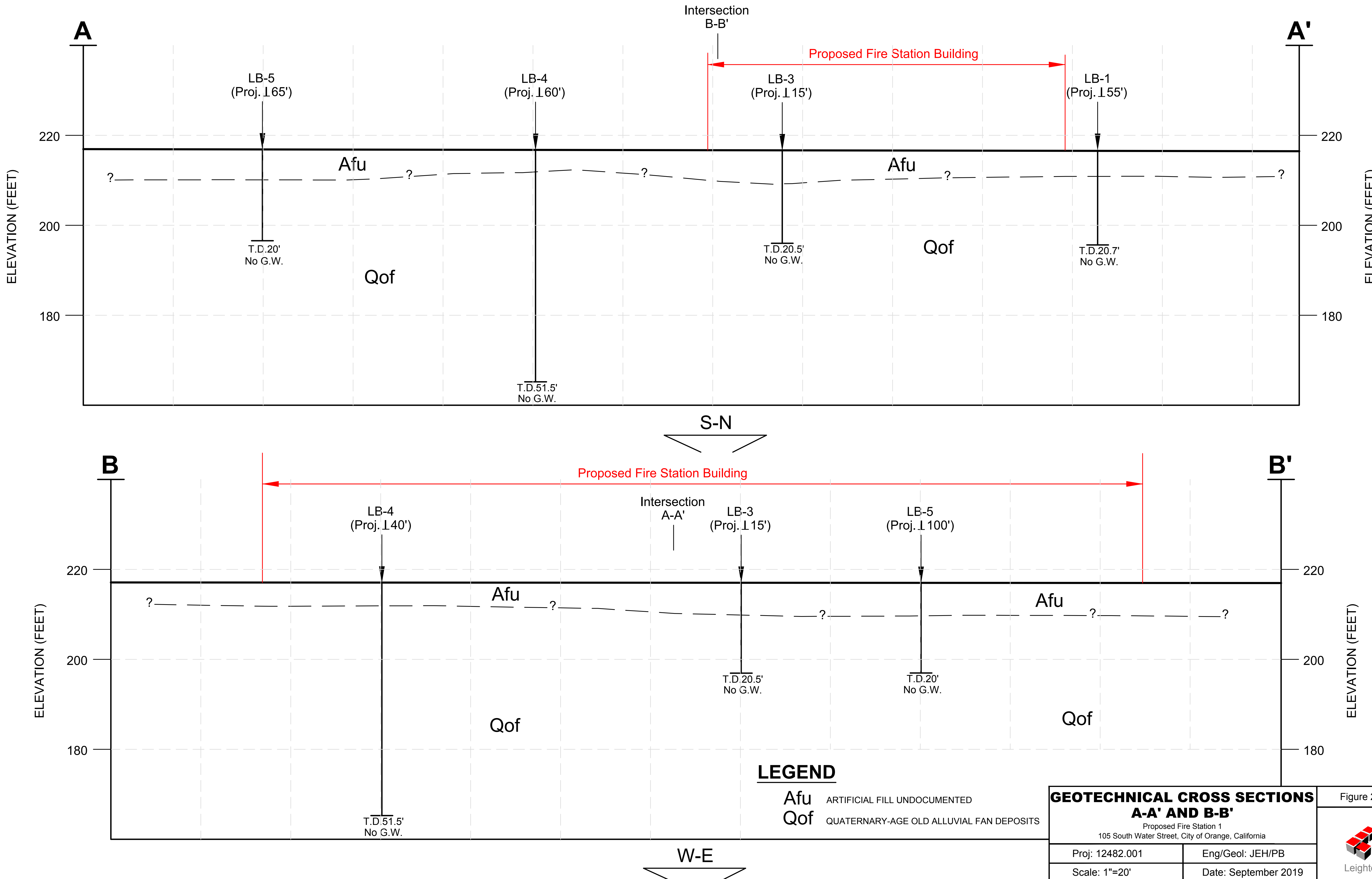


LEGEND

Afu ARTIFICIAL FILL UNDOCUMENTED

Qof QUATERNARY-AGE OLD ALLUVIAL FAN DEPOSITS

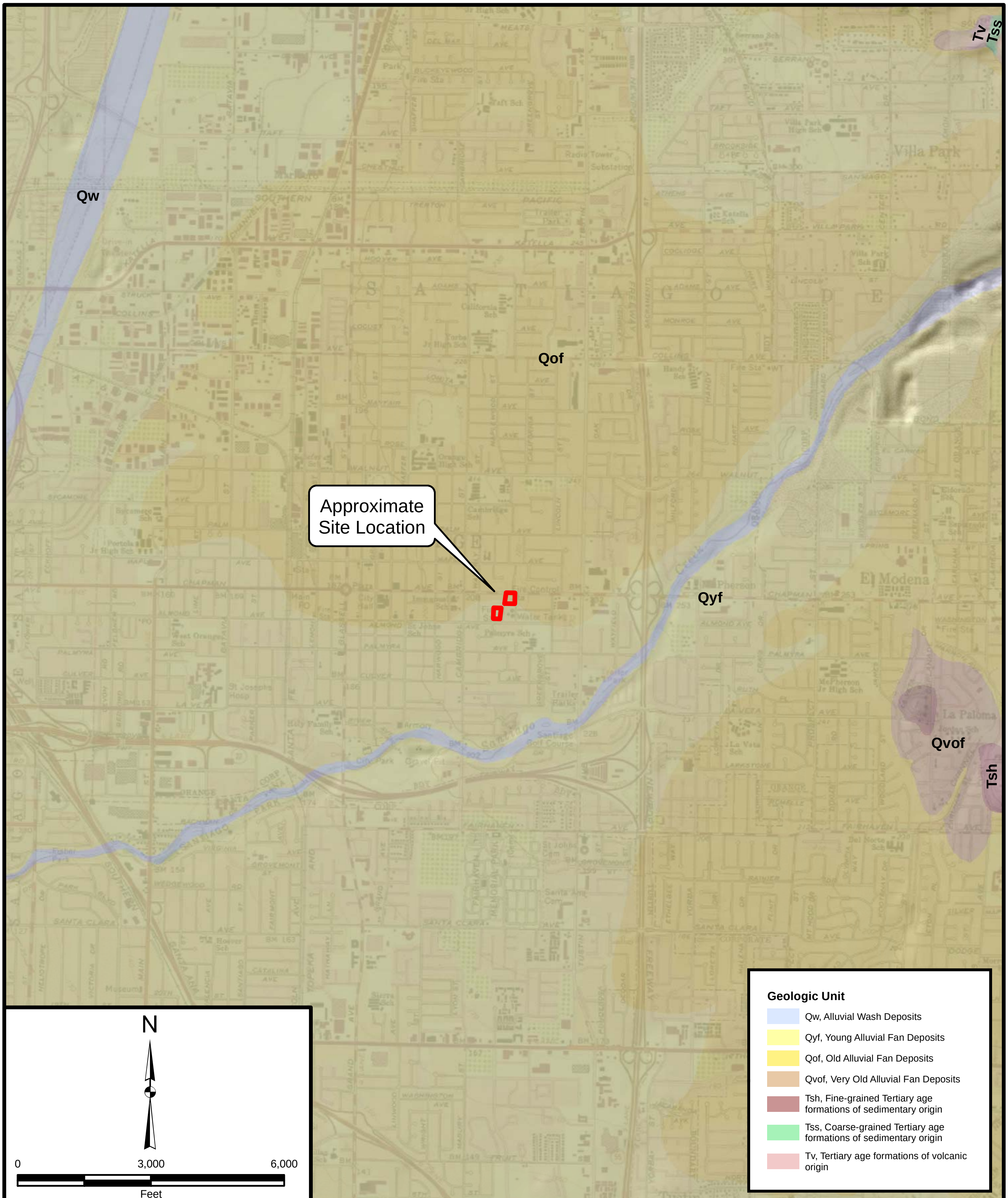
GEOTECHNICAL CROSS SECTIONS A-A' AND B-B'

Proposed Fire Station 1
105 South Water Street, City of Orange, California

Proj: 12482.001	Eng/Geol: JEH/PB
Scale: 1"=20'	Date: September 2019

Figure 2A

Leighton



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Scale:1 " = 3,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton, USGS
Author: (kmanchikanti)

REGIONAL GEOLOGY MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 3

Leighton

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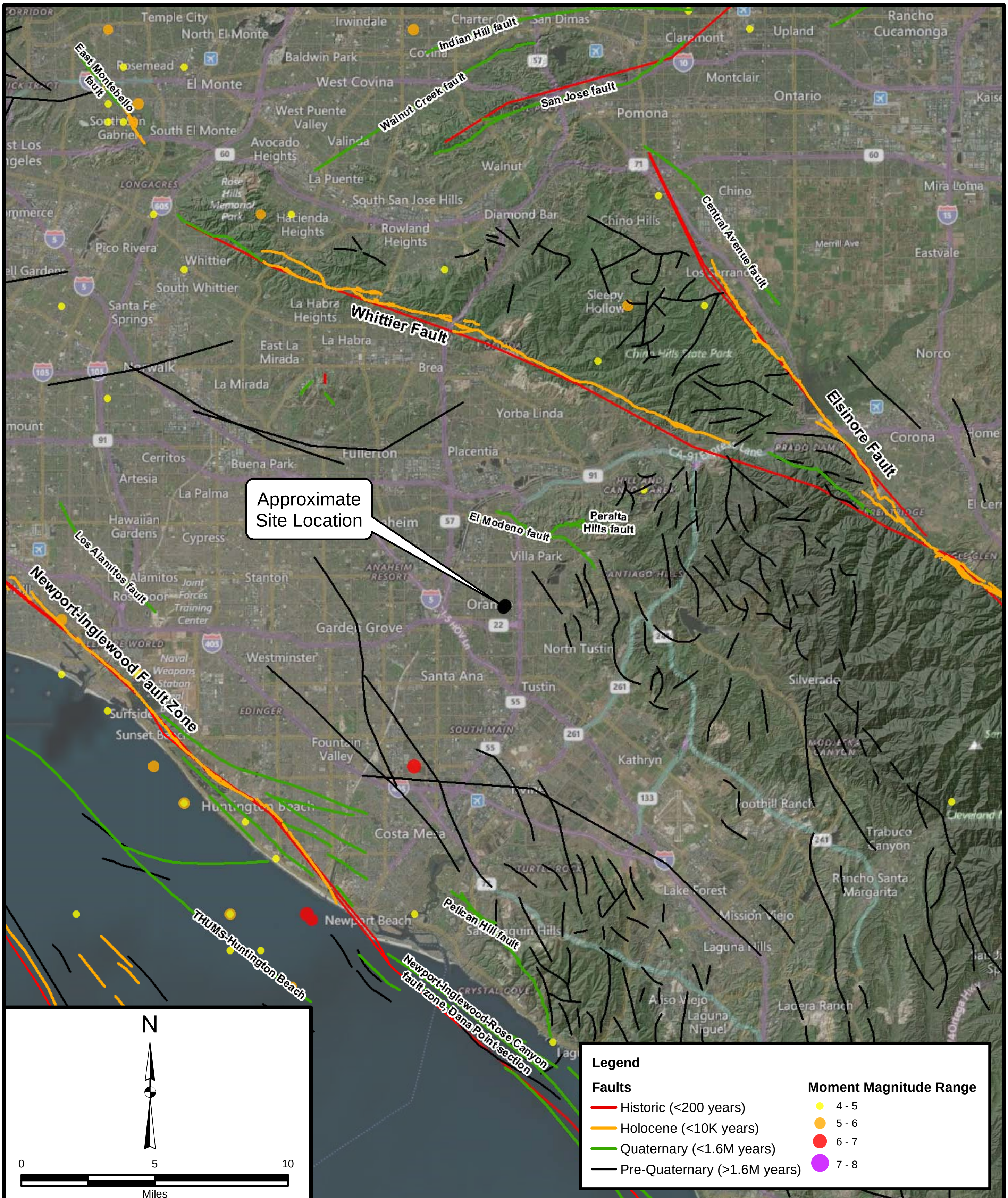

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Scale:1 " = 5 miles	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (kmanchikanti)

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Fire Station 1

105 South Water Street, City of Orange, California

Figure 4

Leighton

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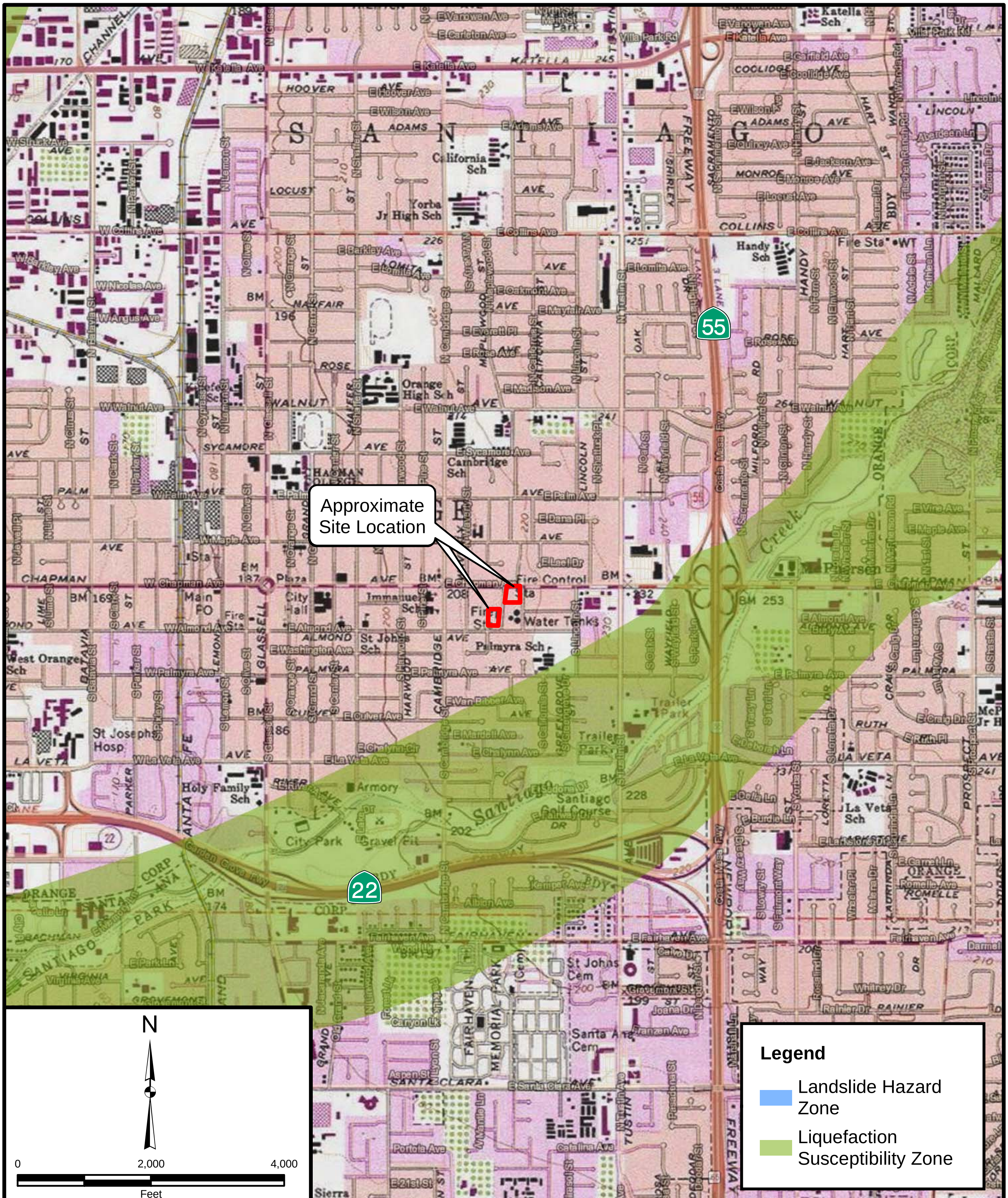

Project: 12482.001	Eng/Geol: JDH/PB
Scale:1 " = 2,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (kmanchikanti)

SEISMIC HAZARD MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 5

Leighton

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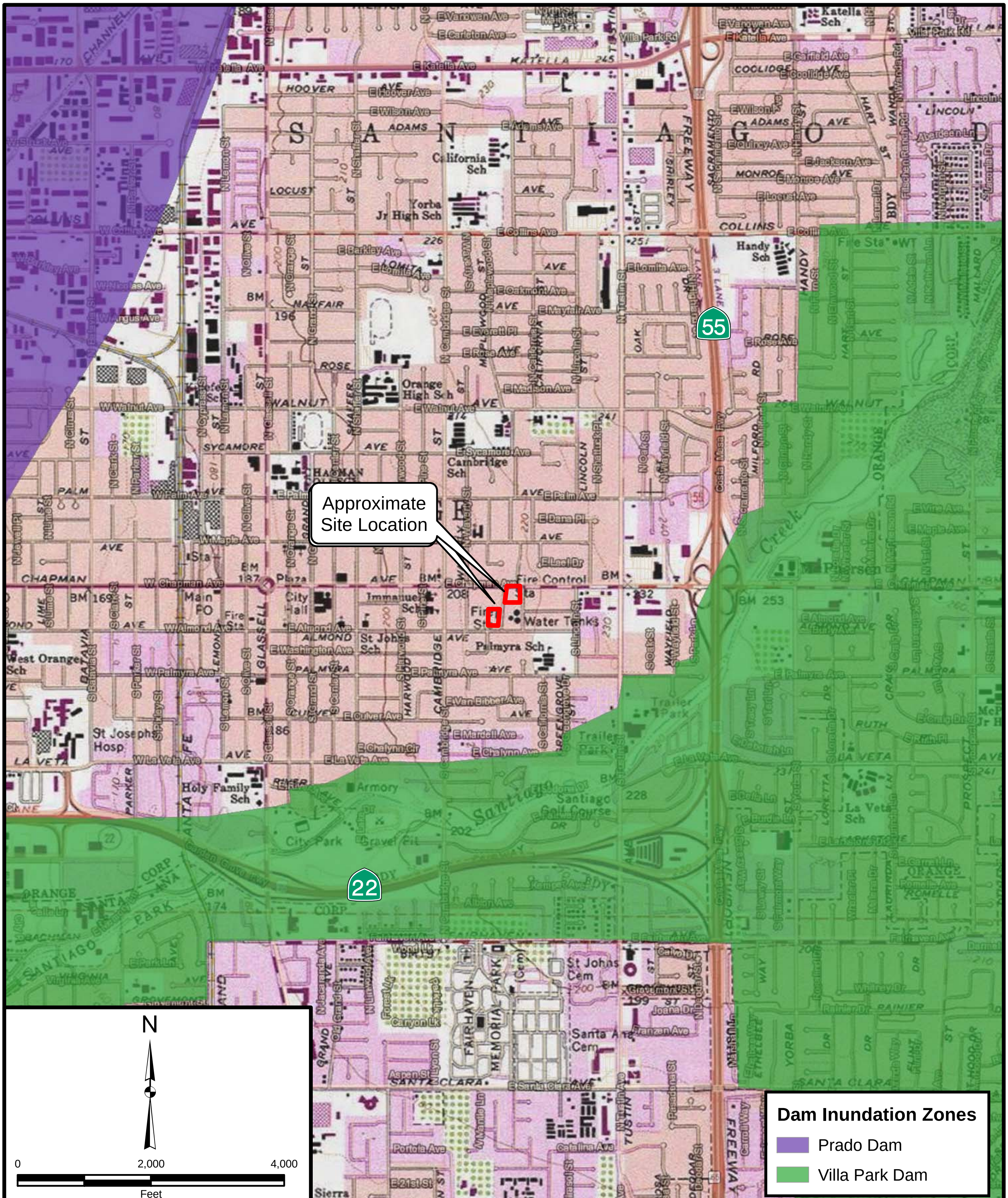

Project: 12482.001	Eng/Geol: JDH/PB
Scale:1 " = 2,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (kmanchikanti)

DAM INUNDATION MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 6

Leighton

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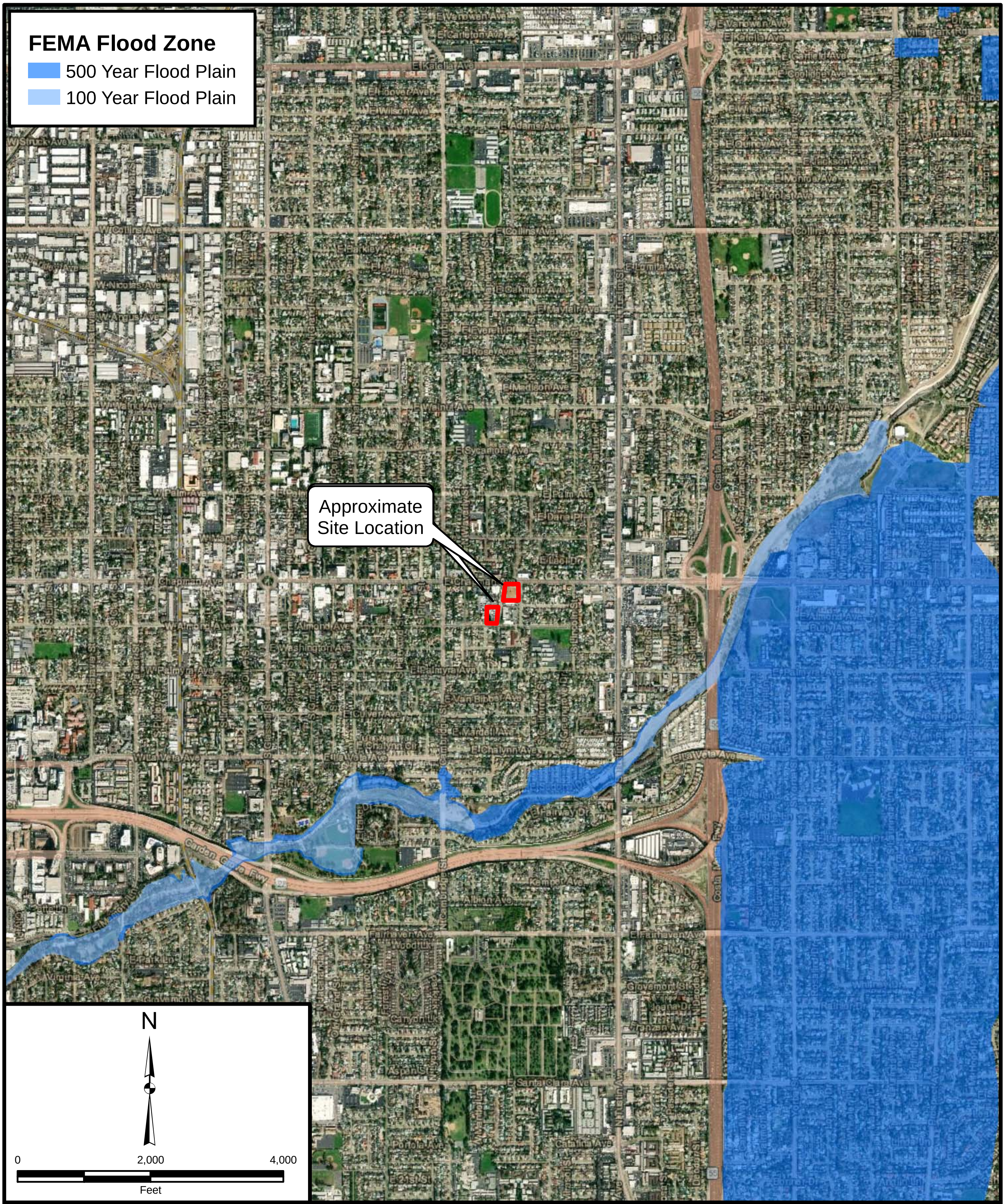

Project: 12482.001	Eng/Geol: JDH/PB
Scale:1 " = 2,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton; FEMA
Author: Leighton Geomatics (kmanchikanti)

FLOOD HAZARD ZONE MAP

Proposed Fire Station 1
105 South Water Street
City of Orange, California

Figure 7

Leighton

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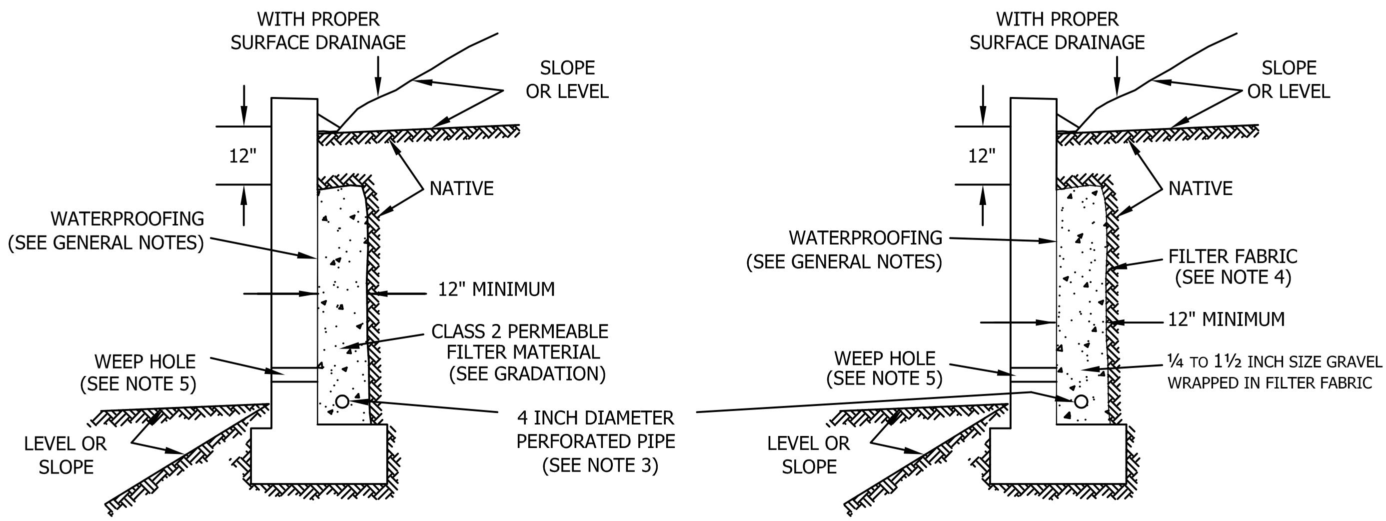
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 8

Important Information about This

Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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

APPENDIX A

REFERENCES

APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING AND INFILTRATION LOGS

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration 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			B-1					@Surface: gravel, sand **Artificial Fill, undocumented (Afu)**	EI
215				R-1	8 14 17	111	3	SM	@2.5' SILTY SAND (SM), medium dense, orange brown, moist, fine sand, 30% fines (field estimate), 10% gravel (field estimate) subround, subangular, fine gravel, mechanically fractured	
	5			R-2	11 23 20			SM	**Quaternary Old Alluvial Fan (Qof)** @5' SILTY SAND with gravel (SM), medium dense, orange brown, moist, fine sand, fine to medium gravel, angular due to mechanical fracturing (soil cuttings)	
210										
	10			R-3	50/5"	122	1	GW-GM	@10' GRAVEL with silt and sand (GW-GM), dense, orange brown, moist, fine sand, fine to coarse gravel and cobbles, no recovery	
205										
	15			R-4	41 50/5"	121	7	SP-SM	@15' SAND with silt to silty sand with gravel (SP-SM), dense, orange brown, moist, fine to medium sand, oxidized throughout, fine angular gravel, due to mechanical fracturing	
200										
	20			S-1	18 50/2.5"			SP-SM	@20' SAND with silt to silty sand with gravel (SP-SM), very dense, orange brown, moist, fine to medium sand, oxidized throughout, fine angular gravel, angular due to mechanical fracturing, low recovery	
195									**Total Depth: 21 feet** **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	25									
190										
	30									

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

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

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

GEOTECHNICAL BORING LOG LB-2

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration Location Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1	Bulk					@Surface: 2 inches Asphalt Concrete **Artificial Fill, undocumented (Afu)**	
215				R-1	Driven	23 23 25	97	3	SM	@2.5' SILTY SAND with gravel (SM), medium dense, orange brown, moist, fine sand, fine to medium gravel, subangular to subround, majority of gravel in cuttings, no recovery	
	5			R-2	Driven	25 50/3"			GP	**Quaternary Old Alluvial Fan (Qof)** @5' GRAVEL with sand (GP), dense, grayish brown, slightly moist, fine sand, fine to medium gravel, subangular to subround, mechanically fractured gravel, low recovery	
210										**Drilling refusal at 6 feet** **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	10										
205											
	15										
200											
	20										
195											
	25										
190											
	30										

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

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

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

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

GEOTECHNICAL BORING LOG LB-3

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration 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.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1	Bulk					@Surface: Gravel and sand **Artificial Fill, undocumented (Afu)**	MD, CR
215				R-1	Driven	8 6 6	112	7	SM	@2.5' SILTY SAND (SM), loose, brown, moist, fine sand, gravel, fine gravel, angular to subangular gravel, 28% fines	-200
	5			R-2	Driven	7 11 15	124	2	SM	@5' SILTY SAND (SM), medium dense, brown, moist, fine sand, no recovery, cuttings same as @2.5'	
210										**Quaternary Old Alluvial Fan (Qof)**	
				R-3	Driven	10 15 18			SP	@8' SAND with gravel (SP), medium dense, light brown to grayish brown, slightly moist, fine sand, angular to subangular fine gravel	
	10			R-4	Driven	10 18 32			SP	@10' SAND with gravel (SP), dense, light brown to grayish brown, slightly moist, fine to medium sand, fine to medium subangular gravel due to mechanical fracturing, clay with gravel in shoe	
205											
	15			R-5	Driven	50/3"			GW-GM	@15' GRAVEL with silt and sand (GW-GM), very dense, no recovery, sand with gravel in cuttings	
200											
	20			S-1	Driven	50/5"			GW-GM	@20' GRAVEL with silt and sand (GW-GM), very dense, no recovery, sand with gravel in cuttings	
195										**Total Depth: 20.5 feet** **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	25										
190											
	30										

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

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

GEOTECHNICAL BORING LOG LB-4

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration 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.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1	Bulk					@Surface: sand, gravel **Artificial Fill, undocumented (Afu)**	
215				R-1	Driven	19 12 9			SM	@2.5' SILTY SAND with gravel (SM), medium dense, brown	
	5			R-2	Driven	13 20 27	136	3	GP	**Quaternary Old Alluvial Fan (Qof)** @5' SANDY GRAVEL (GP), medium dense, grayish brown, slightly moist, fine sand, rounded gravels, with few mechanically fractured during sampling, cobble-sized slaty bedrock fragments @8' gravel, hard drilling	
210											
	10			R-3	Driven	32 32 18	122	7	SC	CLAYEY SAND with gravel (SC), medium dense, reddish brown, moist, low to medium plasticity, 31% fines	-200
205											
	15			R-4	Driven	30 47 50/5"	108	4	GW-GM	@15' GRAVEL with silt and sand (GW-GM), very dense, reddish brown, moist, fine, subangular, well graded	SA
200											
	20			R-5	Driven	50/5"	115	4	GW-GM	@20' GRAVEL with silt and sand (GW-GM), very dense, reddish brown, moist, fine, subangular, well graded	
195											
	25			S-1	Driven	32 49 50/4"			SP	@25' SAND (SP), very dense, reddish brown, moist, fine to medium, subangular, trace silt	
190											
	30										

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

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

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

GEOTECHNICAL BORING LOG LB-4

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration 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.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-6		50/2"			GW-GM	@30' GRAVEL with silt and sand (GW-GM), very dense, brown, no recovery	
185											
	35			S-2		11 18 10			CL	@35' CLAY (CL), very stiff, orange brown, moist, low to medium plasticity	
180											
	40			R-7		5 10 14	105	21	CL	@40' CLAY (CL), stiff, orange brown, moist, low to medium plasticity, with some gravel, 81% fines	-200, AL
175											
	45			S-3		11 14 50/5"			SP	@45' SAND (SP), very dense, light brown, moist, fine grained	
170											
	50			R-8		20 24 23	119	11	CL	@50' CLAY (CL), very stiff, orange brown, moist, low plasticity	
165										**Total Depth: 51.5 feet** **No groundwater encountered** **Caved at 30'** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	55										
160											
	60										

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

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

GEOTECHNICAL BORING LOG LB-5

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	217'
Location	See Figure 2 Exploration 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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
217	0		B-1					@Surface: 3.5 inches Asphalt Concrete **Artificial Fill, undocumented (Afu)**	
215	2.5		R-1	11 29 30	127	3	SP	@2.5' SAND with gravel (SP), dense, light orange brown, moist, fine to medium, subangular, 4% fines	-200
	5		R-2	26 36 44	119	1	GP	@5' SANDY GRAVEL (GP), dense, grayish orange brown, moist, fine to medium, subangular slaty rock fragments @6' gravel, hard drilling, abundant mechanically fractured gravel, small cobble-sized slaty rock fragments	
210	7							**Quaternary Old Alluvial Fan (Qof)**	
	10		R-3	21 50/4"	121	3	GP	@10' SANDY GRAVEL with cobbles (GP), dense, orange brown, moist, abundant mechanically fractured rock fragments	
205									
	15		R-4	50/3"			GW-GM	@15' GRAVEL with silt and sand (GW-GM), very dense, reddish brown, moist, fine to medium, subangular, no recovery	
200									
	20		S-1	50/1"			GW-GM	@20' GRAVEL with silt and sand (GW-GM), very dense, reddish brown, moist, fine to medium, subangular, no recovery	
195								**Total Depth: 20 feet** **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	25								
190									
	30								

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

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

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

GEOTECHNICAL BORING LOG LB-6

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	216'
Location	See Figure 2 Exploration 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.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0									@Surface: 6 inches Asphalt Concrete	
215				B-1					SC	**Artificial Fill, undocumented (Afu)**	
				R-1		3 4 11	117	12	SC	@2.5 CLAYEY SAND (SC), stiff, brown, moist, fine, low plasticity, 43% fines	-200, AL
	5			R-2		50/5"			SP	**Quaternary Old Alluvial Fan (Qof)** @ 5' No Recovery	
210				R-3		17 24 29			SP	@ 6.5' SAND with gravel (SP), dense, light brown, slightly moist, fine, subangular	
										Total Depth: 8' **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
	10										
205											
	15										
200											
	20										
195											
	25										
190											
	30										

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

TYPE OF TESTS:

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

GEOTECHNICAL BORING LOG LB-7

Project No.	12482.001	**Date Drilled**	8-8-19
Project	Proposed Fire Station 1	**Logged By**	MM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	216'
Location	See Figure 2 Exploration 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.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1	Bulk					@Surface: 3 inches Asphalt Concrete **Artificial Fill, undocumented (Afu)**	
215				R-1	Driven				SM	@2.5' SILTY SAND with gravel (SM), light brown, moist, fine, subangular	
210	5									**Quaternary Old Alluvial Fan (Qof)**	
				R-2	Driven	27 35 42	100	10	SM	@6' SAND with gravel (SP), dense, light brown, fine, subangular	
				R-3	Driven	18 24 38			SP	@7.5' SAND with gravel (SP), dense, light brown, fine to medium, subangular	
205	10									**Total Depth: 9'** **No groundwater encountered** **Backfilled with soil cuttings and tamped upon completion of drilling**	
200	15										
195	20										
190	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. ***

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

Leighton

Project: 12482
Exploration #/Location: LB-4
Depth Boring drilled to (ft): 50
Tested by: JK
USCS Soil Type in test zone:
Weather (start to finish): Cloudy
Liquid Used/pH: H2O
Measured boring diameter: 9 in. 4.5 in. Well Radius
Approx Depth to GW below GS: 60 ft
Well Prep: Caved to 30', Backfilled to 15', Added Bentonite, Gravel to 14'

Initial estimated Depth to Water Surface (in.): 127
Average depth of water in well, "h" (in.): 41
approx. h/r: 9.1
Tu (Fig. 8) (ft): 49.4
Tu>3h?: yes, OK
Cross-sectional area for vol calcs (in.^2): 63.6

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	14. ft	0. in.	168	
Pilot Tube stickup (+ is above ground)	0. ft	10. in.	10	
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	4. in.	112	102 Depth below GS (in.)
Float Assembly ID		DHVA		
Float assembly Extension length (in.)		34		

Flow Meter:
Meter ID SN18003236
Meter Colo Black
Meter Unit Gallons
DL ID 1
0.05 gallons/pulse

Field Data / Calculations

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Interval Pulse Count	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	Start time:	Gallons		ft	in.																
-																					
8/8/19	10:30	593.5		10	8.5				6.3E+07	118.5	49.5										
8/8/19	10:55	595.2		10	7.4			25	6.3E+07	117.4	50.6	1.1	50	393	-70	323	13	775	0.9	0.10	**0.48**
8/8/19	11:26	595.4		10	9.9			31	6.3E+07	119.9	48.1	-2.5	49	46	159	205	7	397	0.9	0.05	**0.25**
8/8/19	11:53	595.4		10	11.8			27	6.3E+07	121.8	46.2	-1.9	47	0	121	121	4	268	0.921	0.04	**0.18**
8/8/19	12:24	595.4		11	2.1			31	6.3E+07	124.1	43.9	-2.3	45	0	146	146	5	283	0.9	0.05	**0.20**
8/8/19	12:50	595.45		11	3.2			26	6.3E+07	125.2	42.8	-1.1	43	12	70	81	3	188	0.9	0.03	**0.13**
8/8/19	13:15	595.45		11	4.3			25	6.3E+07	126.3	41.7	-1.1	42	0	70	70	3	168	0.9	0.03	**0.12**
8/8/19	13:45	595.45		11	5.9			30	6.3E+07	127.9	40.1	-1.6	41	0	102	102	3	203	0.9	0.04	**0.15**
8/8/19	14:15	595.54		11	6.2			30	6.3E+07	128.2	39.8	-0.3	40	21	19	40	1	80	0.9	0.01	**0.06**
8/8/19	14:30	595.7		11	6.1			15	6.3E+07	128.1	39.9	0.1	40	37	-6	31	2	122	0.9	0.02	**0.09**

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

Leighton

Project: 12482

Exploration #/Location: *LB-5*

Depth Boring drilled to (ft): 30

Tested by: JK

USCS Soil Type in test zone:

Weather (start to finish): Sunny

Liquid Used/pH:

Measured boring diameter: 10 in. 5 in. Well Radius

Approx Depth to GW below GS: 60 ft

Well Prep: Drilled to 30', Caved to 25', Backfilled to 21', Added Bentonite, Gravel to 20.5'

Initial estimated Depth to Water Surface (in.): 212

Average depth of water in well, "h" (in.): 34

approx. h/r: 6.9

Tu (Fig. 8) (ft): 42.4

Tu>3h?: yes, OK

Cross-sectional area for vol calcs (in.^2): 78.5

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	20. ft	6. in.	246	
Pilot Tube stickup (+ is above ground)	0. ft	4. in.	4	
Depth to top of sand outside of casing from top of pilot tube				
Depth to top of float assembly from top of pilot tube	15. ft	8. in.	188	184 Depth below GS (in.)
Float Assembly ID		E		
Float assembly Extension length (in.)		34		

Flow Meter:

Meter ID: SN18003242

Meter Color: Black

Meter Unit: Gallons

DL ID: 2

0.05 gallons/pulse

Field Data | **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube): ft	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	Start time:	Gallons																			
-																					
8/8/19	12:03	350.4		18	0.6				6.3E+07	212.6	33.4										
8/8/19	12:26	364.2		17	11.2			23	6.3E+07	211.2	34.8	1.4	34	3188	-110	3078	134	8029	0.9	1.71	**6.44**
8/8/19	12:51	375.8		17	11.6			25	6.3E+07	211.6	34.4	-0.4	35	2680	31	2711	108	6506	0.9	1.43	**5.15**
8/8/19	13:16	385.4		18	1.3			25	6.3E+07	213.3	32.7	-1.7	34	2218	133	2351	94	5643	0.921	1.35	**4.59**
8/8/19	13:46	398.5		18	1.4			30	6.3E+07	213.4	32.6	-0.1	33	3026	8	3034	101	6068	0.9	1.44	**5.07**
8/8/19	14:16	416.8		18	1.2			30	6.3E+07	213.2	32.8	0.2	33	4227	-16	4212	140	8423	0.9	1.97	**7.02**
8/8/19	14:45	437.2		17	11.5			29	6.3E+07	211.5	34.5	1.7	34	4712	-133	4579	158	9474	0.9	2.04	**7.69**
8/8/19	15:00	448.6		17	10.8			15	6.3E+07	210.8	35.2	0.7	35	2633	-55	2578	172	10314	0.9	2.17	**8.10**
8/8/19	15:15	459.3		17	10.7			15	6.3E+07	210.7	35.3	0.1	35	2472	-8	2464	164	9855	0.9	2.07	**7.66**
8/8/19	15:30	468.9		17	11.4			15	6.3E+07	211.4	34.6	-0.7	35	2218	55	2273	152	9090	0.9	1.98	**7.12**

APPENDIX C

LABORATORY TEST RESULTS

Leighton

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
LB-1	2.5							3.2	110.5		
LB-1	10.0							0.6	121.7		
LB-1	15.0							6.5	120.8		
LB-2	2.5							3.0	96.9		
LB-3	2.5							7.0	112.4		
LB-3	5.0							1.8	123.5		
LB-4	5.0							2.6	136.5		
LB-4	10.0							7.4	121.6		
LB-4	15.0							3.5	107.9		
LB-4	20.0							3.5	114.9		
LB-4	40.0							21.1	105.4		
LB-4	50.0							11.5	118.7		
LB-5	2.5							3.0	126.6		
LB-5	5.0							0.9	118.5		
LB-5	10.0							3.3	120.5		
LB-6	2.5							11.7	116.9		
LB-7	6.0							10.5	99.7		

US_LAB_SUMMARY 12482.01 BORING LOGS DRAFT.GPJ ROCKLOG2012.GDT 9/6/19

Leighton

Summary of Laboratory Results

Project Name: WLC Orange FS1

Project Number: 12482.001

Date: 9/6/2019 2:38:21 PM

Figure No. 1

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	WLC/Orange FS 1	Tested By:	O. Figueroa	Date:	09/05/19
Project No.:	12482.001	Input By:	G. Bathala	Date:	09/06/19
Boring No.:	LB-3	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive 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
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	16.5
#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3852	3945	3995	3901		
Weight of Mold (g)	1817	1817	1817	1817		
Net Weight of Soil (g)	2035	2128	2178	2084		
Wet Weight of Soil + Cont. (g)	395.3	442.6	435.5	459.9		
Dry Weight of Soil + Cont. (g)	385.1	420.8	405.4	419.4		
Weight of Container (g)	62.3	39.2	39.4	39.8		
Moisture Content (%)	3.16	5.71	8.22	10.67		
Wet Density (pcf)	135.1	141.3	144.6	138.4		
Dry Density (pcf)	131.0	133.7	133.6	125.0		

Maximum Dry Density (pcf)	134.4	**Optimum Moisture Content (%)**	7.1
Corrected Dry Density (pcf)	139.0	**Corrected Moisture Content (%)**	6.1

☐ **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

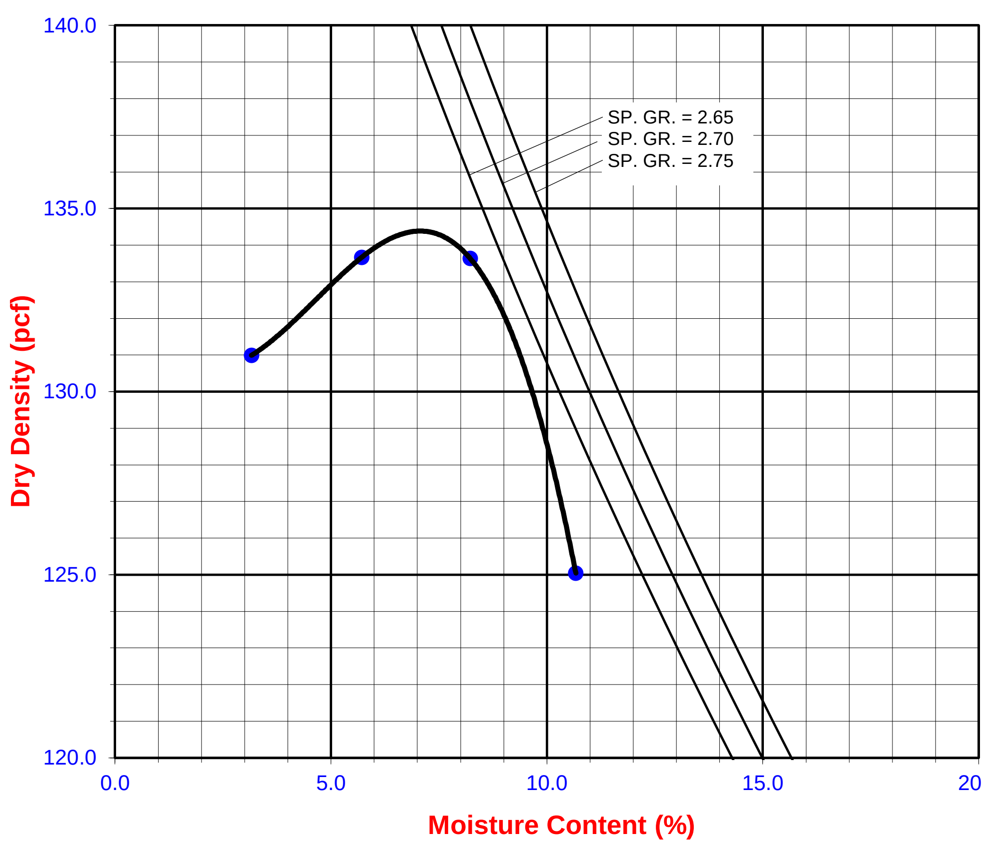
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-3	LB-4	LB-4	LB-5	LB-6			
Sample No.	R-1	R-3	R-7	R-1	R-1			
Depth (ft.)	2.5	10.0	40.0	2.5	2.5			
Sample Type	Ring	Ring	Ring	Ring	Ring			
Soil Identification	Brown silty sand with gravel (SM)g	Brown clayey sand with gravel (SC)g	Brown lean clay with sand (CL)s	Brown poorly-graded sand with gravel (SP)g	Brown clayey sand (SC)			
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)	769.70	1024.80	627.10	867.90	699.50			
Weight of Container (g)	108.70	99.80	108.40	107.80	108.80			
Weight of Dry Sample (g)	661.00	925.00	518.70	760.10	590.70			
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A			
Dry Weight of Sample + Cont. (g)	584.00	740.30	205.10	834.20	442.40			
Weight of Container (g)	108.70	99.80	108.40	107.80	108.80			
Dry Weight of Sample (g)	475.30	640.50	96.70	726.40	333.60			
% Passing No. 200 Sieve	**28.1**	**30.8**	**81.4**	**4.4**	**43.5**			
% Retained No. 200 Sieve	71.9	69.2	18.6	95.6	56.5			

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: WLC/Orange FS 1
Project No.: 12482.001
Client Name:
Tested By: S. Felter Date: 08/19/19

Leighton

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

ASTM D 6913

Project Name:	WLC/Orange FS 1	Tested By:	S. Felter	Date:	08/19/19
Project No.:	12482.001	Checked By:	G. Bathala	Date:	09/06/19
Boring No.:	LB-4	Depth (feet):	15.0		
Sample No.:	R-4				
Soil Identification:	Brown well-graded gravel with silt and sand (GW-GM)s				

		Moisture Content of Total Air - Dry Soil	
Container No.:	A-15	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	893.6	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	107.2	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	786.4	Moisture Content (%)	0.0

After Wet Sieve		
	Container No.	A-15
	Wt. of Dry Soil + Container (g)	842.9
	Wt. of Container (g)	107.2
	Dry Wt. of Soil Retained on # 200 Sieve (g)	735.7

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5	0.0	100.0
1"	25.0	59.8	92.4
3/4"	19.0	112.3	85.7
1/2"	12.5	250.7	68.1
3/8"	9.5	324.1	58.8
#4	4.75	453.6	42.3
#8	2.36	548.3	30.3
#16	1.18	612.3	22.1
#30	0.600	665.1	15.4
#50	0.300	704.4	10.4
#100	0.150	723.1	8.0
#200	0.075	735.1	6.5
PAN			

GRAVEL: **58 %**

SAND: **35 %**

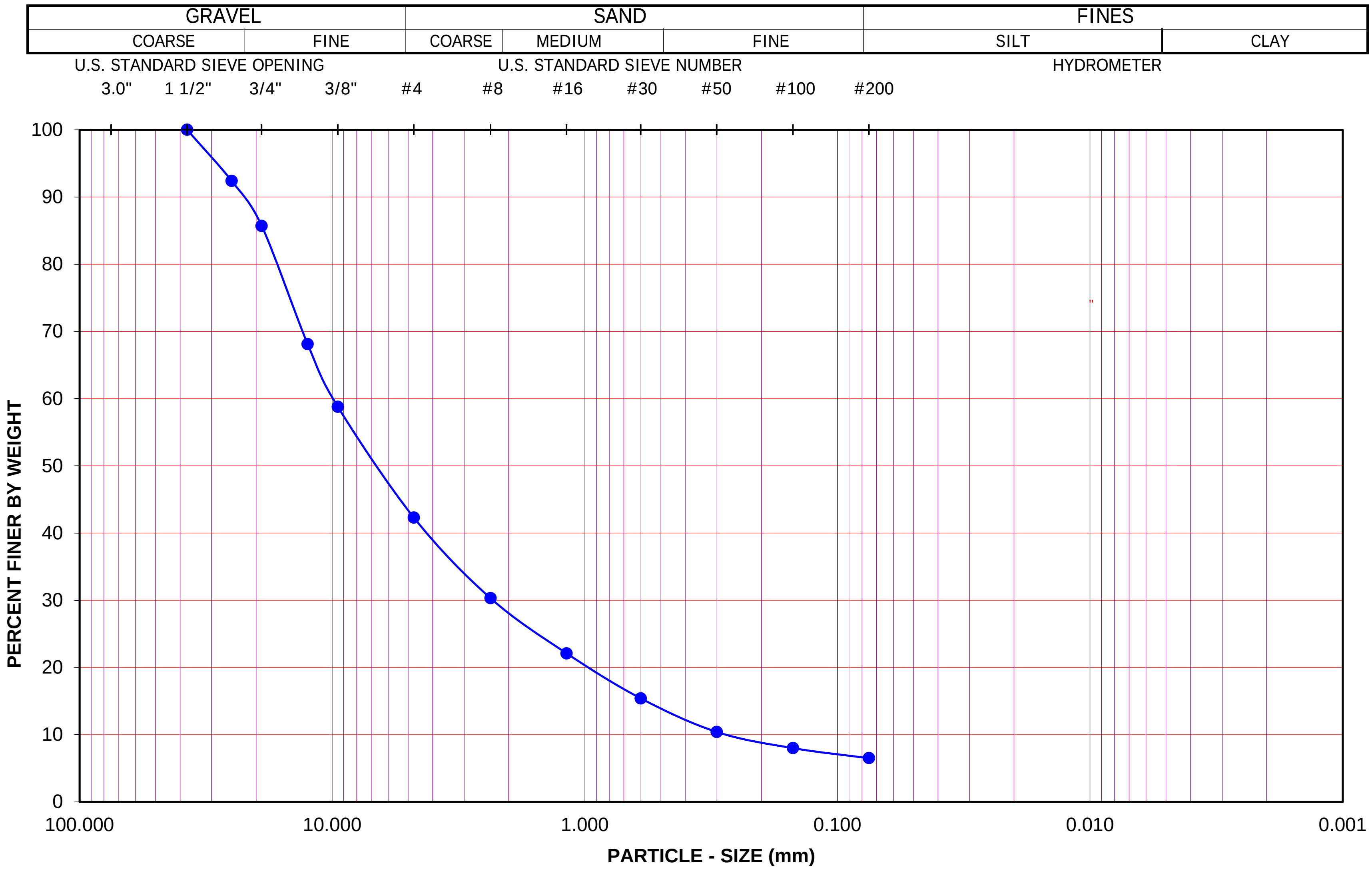
FINES: **7 %**

GROUP SYMBOL: **(GW-GM)s**

Cu = D60/D10 = 35.71

Cc = $(D30)^2/(D60*D10)$ = 2.06

Remarks:



Project Name: WLC/Orange FS 1

Project No.: 12482.001

Boring No.: LB-4

Sample No.: R-4

Depth (feet): 15.0

Soil Type : (GW-GM)s

Soil Identification: Brown well-graded gravel with silt and sand (GW-GM)s

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 58 : 35 : 7

Sep-19

Sieve LB-4, R-4 @ 15.xls

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	WLC/Orange FS 1	Tested By:	S. Felter	Date:	08/20/19
Project No. :	12482.001	Input By:	G. Bathala	Date:	09/06/19
Boring No.:	LB-4	Checked By:	G. Bathala		
Sample No.:	R-7	Depth (ft.)	40.0		
Soil Identification:	Brown lean clay with sand (CL)s				

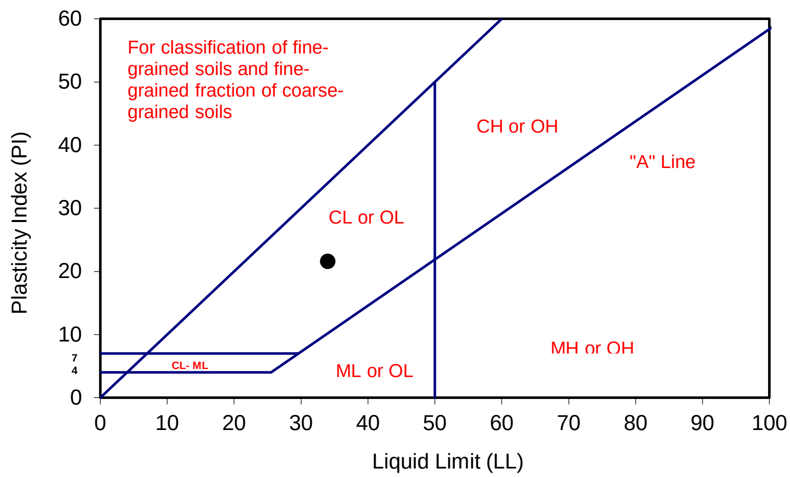
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	27	18	
Wet Wt. of Soil + Cont. (g)	10.19	10.14	20.91	20.14	21.78	
Dry Wt. of Soil + Cont. (g)	9.17	9.16	16.08	15.40	16.37	
Wt. of Container (g)	1.09	1.10	1.03	1.10	1.12	
Moisture Content (%) [Wn]	12.62	12.16	32.09	33.15	35.48	

Liquid Limit	**34**
Plastic Limit	**12**
Plasticity Index	**22**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 10.22

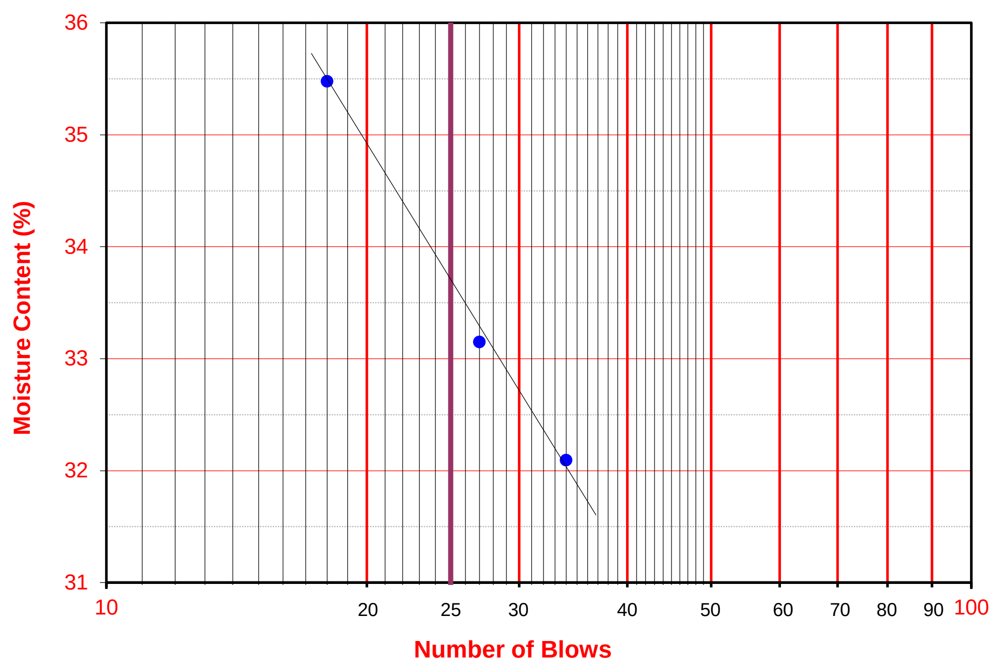
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

[ ]	Wet Preparation Multipoint - Wet
[X]	Dry Preparation Multipoint - Dry
[X]	Procedure A Multipoint Test
[ ]	Procedure B One-point Test



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	WLC/Orange FS 1	Tested By:	S. Felter	Date:	08/20/19
Project No. :	12482.001	Input By:	G. Bathala	Date:	09/06/19
Boring No.:	LB-6	Checked By:	G. Bathala		
Sample No.:	R-1	Depth (ft.)	2.5		
Soil Identification:	Brown clayey sand (SC)				

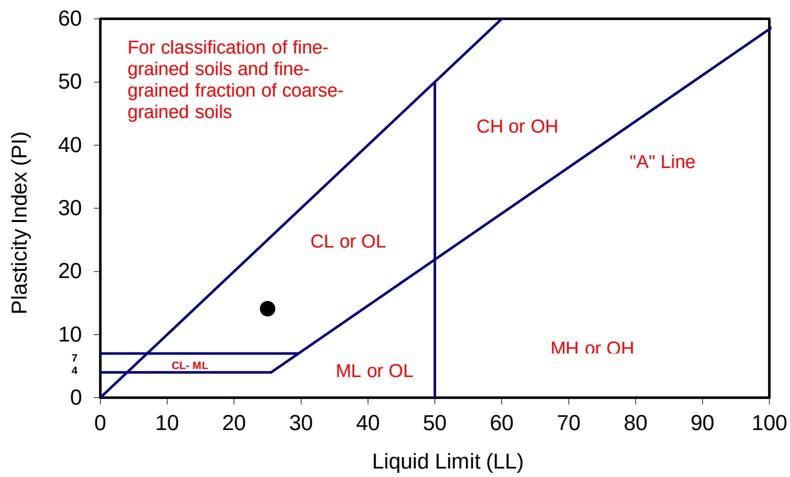
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	22	17	
Wet Wt. of Soil + Cont. (g)	10.14	10.15	22.28	22.14	21.95	
Dry Wt. of Soil + Cont. (g)	9.26	9.25	18.19	17.92	17.65	
Wt. of Container (g)	1.11	1.11	1.12	1.14	1.10	
Moisture Content (%) [Wn]	10.80	11.06	23.96	25.15	25.98	

Liquid Limit	**25**
Plastic Limit	**11**
Plasticity Index	**14**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 3.65

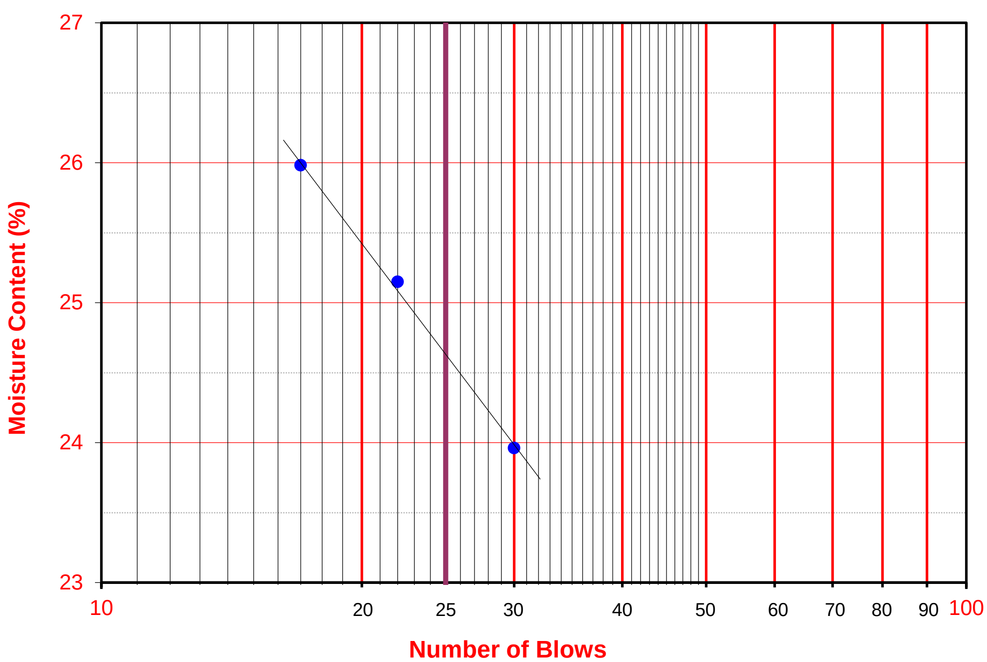
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

[ ]	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
[ ]	Procedure B One-point Test



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	WLC/Orange FS 1	Tested By:	S. Felter	Date:	09/05/19
Project No.:	12482.001	Checked By:	G. Bathala	Date:	09/06/19
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Brown poorly-graded sand with silt (SP-SM)				

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

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0010
Wt. Comp. Soil + Mold	(g)	610.00	436.98
Wt. of Mold	(g)	191.60	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	842.70	628.58
Dry Wt. of Soil + Cont.	(g)	780.30	579.04
Wt. of Container	(g)	0.00	191.60
Moisture Content	(%)	8.00	12.79
Wet Density	(pcf)	126.2	131.7
Dry Density	(pcf)	116.9	116.8
Void Ratio		0.443	0.444
Total Porosity		0.307	0.307
Pore Volume	(cc)	63.5	63.7
Degree of Saturation (%) [S meas]		**48.8**	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.)
09/05/19	7:57	1.0	0	0.4670
09/05/19	8:07	1.0	10	0.4670
Add Distilled Water to the Specimen				
09/05/19	14:02	1.0	355	0.4680
09/06/19	6:28	1.0	1341	0.4680
09/06/19	7:44	1.0	1417	0.4680

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

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	WLC/Orange FS 1	Tested By :	GEB/GB	Date:	09/04/19
Project No. :	12482.001	Input By:	G. Bathala	Date:	09/06/19

Boring No.	LB-3			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SC-SM)g			
Wet Weight of Soil + Container (g)	130.22			
Dry Weight of Soil + Container (g)	126.87			
Weight of Container (g)	39.58			
Moisture Content (%)	3.84			
Weight of Soaked Soil (g)	100.10			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	304			
Crucible No.	12			
Furnace Temperature (°C)	860			
Time In / Time Out	10:50/11:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.7428			
Wt. of Crucible (g)	20.7384			
Wt. of Residue (g) (A)	0.0044			
PPM of Sulfate (A) x 41150	181.06			
PPM of Sulfate, Dry Weight Basis	**188**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	2.0			
PPM of Chloride (C -0.2) * 100 * 30 / B	180			
PPM of Chloride, Dry Wt. Basis	**187**			

pH TEST, DOT California Test 643

pH Value	**7.15**			
Temperature °C	20.1			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	WLC/Orange FS 1	Tested By :	O. Figueroa Date: 09/06/19
Project No. :	12482.001	Input By:	G. Bathala Date: 09/06/19
Boring No.:	LB-3	Depth (ft.) :	0-5
Sample No. :	B-1		

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

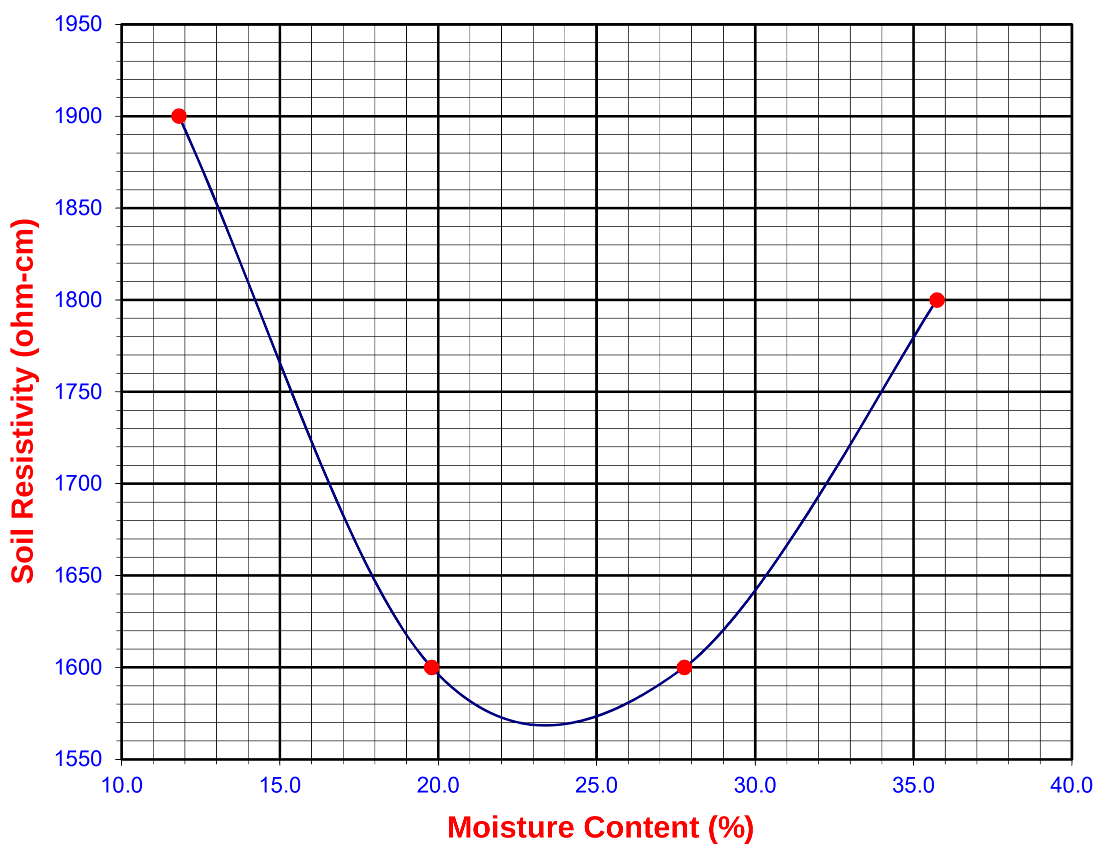
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	11.81	1900	1900
2	20	19.79	1600	1600
3	30	27.76	1600	1600
4	40	35.74	1800	1800
5				

Moisture Content (%) (MCi)	3.84
Wet Wt. of Soil + Cont. (g)	130.22
Dry Wt. of Soil + Cont. (g)	126.87
Wt. of Container (g)	39.58
Container No.	
Initial Soil Wt. (g) (Wt)	130.20
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	
1570	**23.4**	**188**	**187**	**7.15**	**20.1**



APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

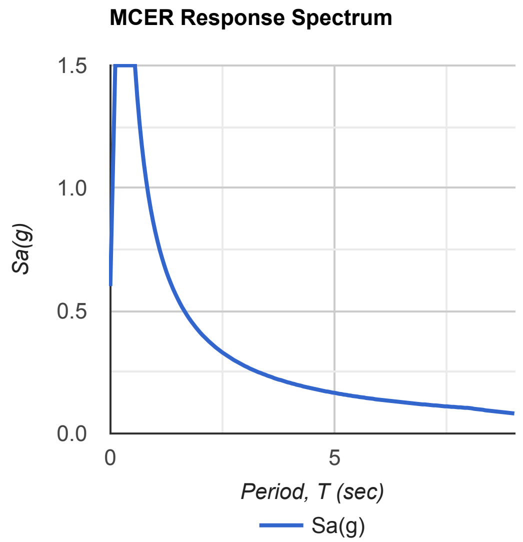
Leighton

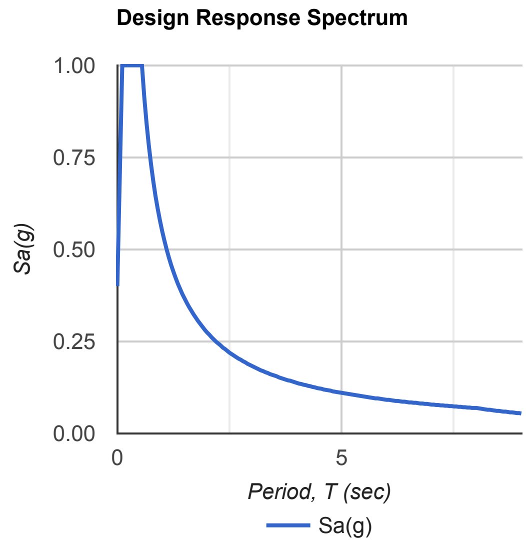
Latitude, Longitude: 33.7873345, -117.84107526

Date	8/20/2019, 11:05:16 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.549	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	0.823	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.549	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.515	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.515	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.504	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.443	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.549	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.511	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.515	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	1.042	Mapped value of the risk coefficient at short periods
C_{R1}	1.073	Mapped value of the risk coefficient at a period of 1 s





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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2008 (v3.3.

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.7873345

Time Horizon

Return period in years

2475

Longitude

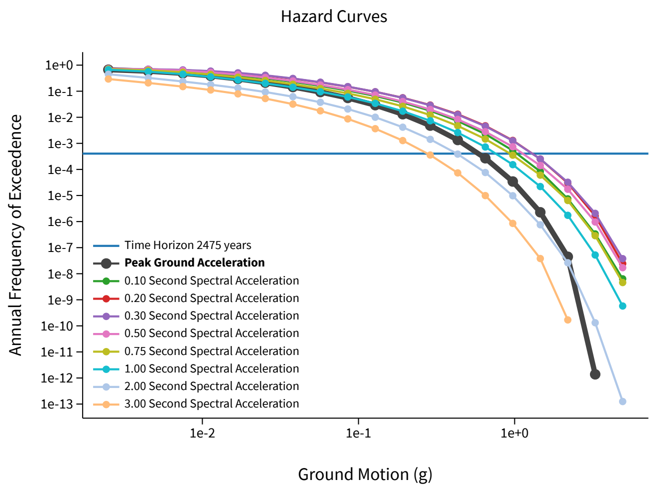
Decimal degrees, negative values for western longitudes

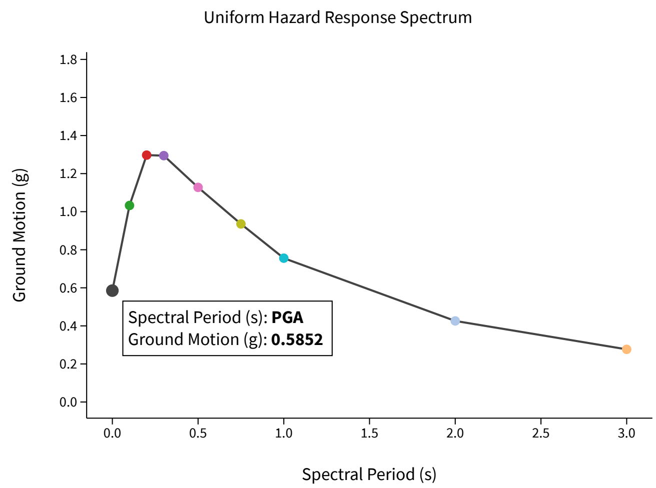
-117.84107526

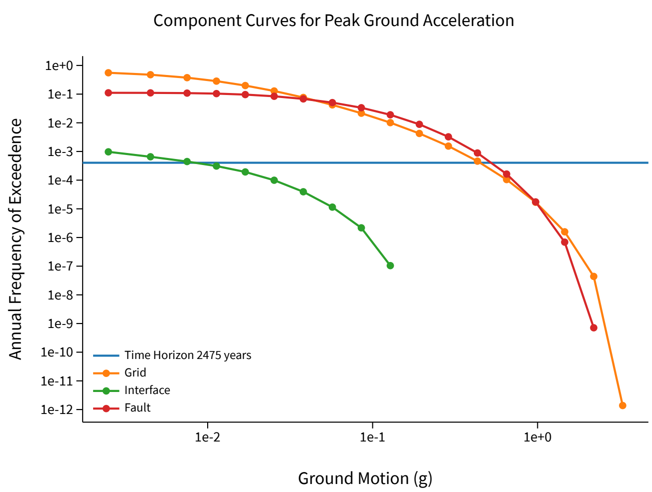
Site Class

259 m/s (Site class D)

Hazard Curve





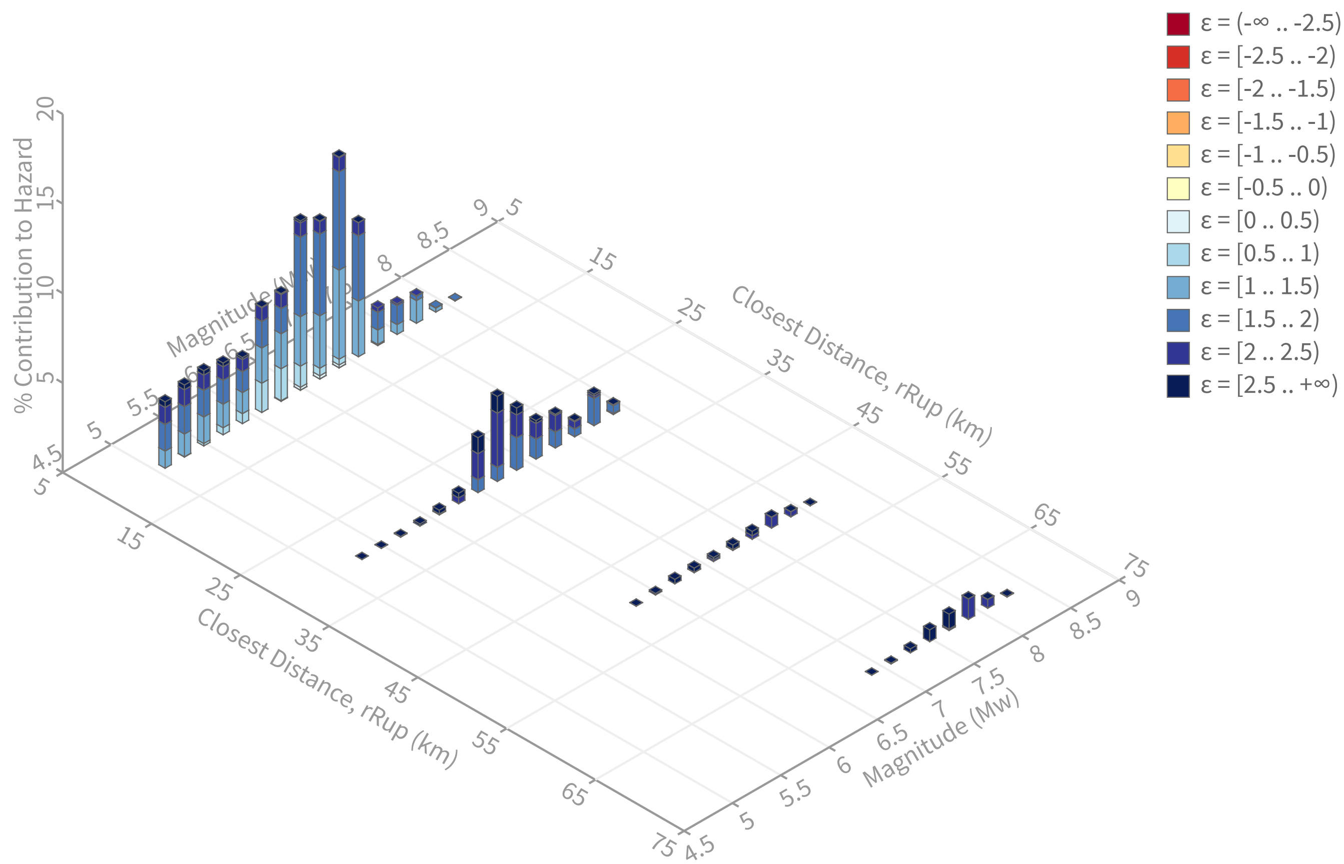


[View Raw Data](#)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.58520247 g

Recovered targets

Return period: 3014.5434 yrs
Exceedance rate: 0.0003317252 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.59
r: 17.19 km
ε_0: 1.72 σ

Mode (largest m-r bin)

m: 6.9
r: 12.83 km
ε_0: 1.53 σ
Contribution: 11.76 %

Mode (largest m-r-ε_0 bin)

m: 6.91
r: 14.52 km
ε_0: 1.68 σ
Contribution: 5.48 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
bFault.ch	Fault							20.70
San Joaquin Hills		11.07	6.98	1.32	117.823°W	33.688°N	171.39	4.92
Puente Hills (Coyote Hills)		12.13	6.71	1.50	117.921°W	33.871°N	321.86	3.90
Puente Hills		16.48	7.06	1.56	117.867°W	33.927°N	351.14	1.89
Chino - alt 1		17.21	6.54	2.03	117.648°W	33.907°N	53.14	1.67
Chino - alt 2		20.12	6.70	1.97	117.629°W	33.886°N	60.81	1.43
bFault.gr	Fault							14.58
San Joaquin Hills		11.19	6.74	1.43	117.823°W	33.688°N	171.39	4.58
Puente Hills (Coyote Hills)		12.13	6.61	1.55	117.921°W	33.871°N	321.86	2.68
Puente Hills		18.62	6.81	1.84	117.867°W	33.927°N	351.14	1.11
Chino - alt 1		17.21	6.49	2.04	117.648°W	33.907°N	53.14	1.01
CAmap.21.ch.in (opt)	Grid							13.08
PointSourceFinite: -117.841, 33.828		6.75	5.76	1.28	117.841°W	33.828°N	0.00	1.08
PointSourceFinite: -117.841, 33.819		6.21	5.73	1.21	117.841°W	33.819°N	0.00	1.04
PointSourceFinite: -117.841, 33.837		7.32	5.79	1.35	117.841°W	33.837°N	0.00	1.03
CAmap.24.ch.in (opt)	Grid							13.04
PointSourceFinite: -117.841, 33.828		6.75	5.76	1.28	117.841°W	33.828°N	0.00	1.08
PointSourceFinite: -117.841, 33.819		6.21	5.73	1.21	117.841°W	33.819°N	0.00	1.04
PointSourceFinite: -117.841, 33.837		7.32	5.79	1.35	117.841°W	33.837°N	0.00	1.03
aFault_aPriori_D2.1	Fault							12.03
Elsinore : W		13.90	6.96	1.73	117.792°W	33.907°N	18.74	3.63
Elsinore : GI		23.65	6.82	2.13	117.590°W	33.829°N	78.66	2.62
Elsinore : GI+T		23.65	7.26	1.96	117.590°W	33.829°N	78.66	1.85
Elsinore : GI+T+J+CM		23.65	7.74	1.74	117.590°W	33.829°N	78.66	1.02
aFault_MoBal	Fault							11.61
Elsinore : W		13.90	6.94	1.74	117.792°W	33.907°N	18.74	5.31
Elsinore : GI		23.65	6.79	2.14	117.590°W	33.829°N	78.66	1.53
CAmap.21.gr.in (opt)	Grid							6.38
CAmap.24.gr.in (opt)	Grid							6.27
aFault_unseg	Fault							2.28
Elsinore		15.43	7.48	1.54	117.792°W	33.907°N	18.74	2.14

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 12482.001

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)		Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-4			1.3	
3	4	14		14	18	0.22
5	3.5	30		30	39	0.09
10	5	32		32	42	0.12
15	5	52		52	68	0.07
20	5	52		52	68	0.07
25	5	90		90	100	0.05
30	5	52		52	68	0.07
35	5	33		33	43	0.12
40	5	15		15	20	0.26
45	5	90		90	100	0.05
50	7.5	30		30	39	0.19
60	10	40	Assumed	40	52	0.19
70	10	40		40	52	0.19
80	15	40		40	52	0.29
100	10	40		40	52	0.19
Summatioı	100					2.18
					Navg = Sum(di) / Sum(di / Ni) =	**46**

Extract of ASCE 7-10 Table 20.3-1 Site Classification (2016 CBC 1613A.3.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	50.001	100	1200	2500	366	762		
D	Stiff Soil	**15**	**50**	600	1200	183	366	46	1128
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

Site class, Table 20.3-1: D

Liquefaction Susceptibility Analysis: SPT Method

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

Project: 12482

Project No.: Proposed Fire Station 1

Leighton

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)	
LB-1	100	100	0	211	111
LB-3	100	100	0	211	111
LB-4	100	100	0	211	111
LB-5	100	100	0	211	111
					0
					0
					0
					0
					0
					0

General Parameters:		
a_{max} =	0.51g	MCE
M_W =	6.9	
MSF eq:	1	(Idriss, 2001)
MSF =	1.24	
Hammer Efficiency =	83	%
C_E =	1.38	
C_B =	1	
$C_{S(SPT)}$ =	1.2	
$C_{S(ring)}$ =	1	
Rod Stickup (feet) =	3	
Ring sample correction =	0.65	

Summary of Liquefaction Susceptibility Analysis: SPT Method

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

Project: 12482

Project No.: Proposed Fire Station 1

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 4	3	4		30	115	31	2	1	20.2	345	35.5	45.7	>Range	345	0.33	0.27	**NonLiq**	45.7	0.01		0.00	0.0
LB-1	4 to 8	5	4		30	125	43	2	1	28.0	585	49.3	61.6	>Range	585	0.33	0.26	**NonLiq**	61.6	0.01		0.00	0.0
LB-1	8 to 13	10	5		10	125	80	2	1	52.0	1210	80.3	82.9	>Range	1210	0.32	0.26	**NonLiq**	82.9	0.01		0.00	0.0
LB-1	13 to 18	15	5		10	125	80	2	1	52.0	1835	65.2	67.5	>Range	1835	0.32	0.26	**NonLiq**	67.5	0.01		0.01	0.0
LB-1	18 to 22	20	5		10	125	80	1	1.2	96.0	2460	116.2	119.6	>Range	2460	0.32	0.26	**NonLiq**	119.6	0.01		0.00	0.0
LB-3	0 to 4	3	4		28	115	12	2	1	7.8	345	13.8	20.2	0.218	345	0.33	0.27	**NonLiq**	20.2	0.05		0.02	0.1
LB-3	4 to 7	5	3		28	125	26	2	1	16.9	585	29.8	38.5	>Range	585	0.33	0.26	**NonLiq**	38.5	0.03		0.01	0.0
LB-3	7 to 9	8	3		5	125	33	2	1	21.5	960	35.0	35.0	>Range	960	0.33	0.26	**NonLiq**	35.0	0.04		0.01	0.0
LB-3	9 to 13	10	4		5	125	50	2	1	32.5	1210	50.2	50.2	>Range	1210	0.32	0.26	**NonLiq**	50.2	0.01		0.00	0.0
LB-3	13 to 18	15	5		10	120	80	2	1	52.0	1823	65.4	67.7	>Range	1822.5	0.32	0.26	**NonLiq**	67.7	0.01		0.01	0.0
LB-3	18 to 22	20	5		10	120	80	1	1.2	96.0	2423	117.1	120.5	>Range	2422.5	0.32	0.26	**NonLiq**	120.5	0.01		0.00	0.0
LB-4	0 to 4	3	4		30	120	21	2	1	13.7	360	24.1	32.5	>Range	360	0.33	0.27	**NonLiq**	32.5	0.02		0.01	0.2
LB-4	4 to 8	5	4		10	130	47	2	1	30.6	610	53.9	55.9	>Range	610	0.33	0.26	**NonLiq**	55.9	0.01		0.00	0.2
LB-4	8 to 13	10	5		31	125	50	2	1	32.5	1248	49.4	62.2	>Range	1247.5	0.32	0.26	**NonLiq**	62.2	0.01		0.01	0.2
LB-4	13 to 18	15	5		7	115	80	2	1	52.0	1848	65.0	65.7	>Range	1847.5	0.32	0.26	**NonLiq**	65.7	0.01		0.01	0.2
LB-4	18 to 23	20	5		7	115	80	2	1	52.0	2423	63.4	64.1	>Range	2422.5	0.32	0.26	**NonLiq**	64.1	0.01		0.01	0.2
LB-4	23 to 28	25	5		7	120	80	1	1.2	96.0	3010	105.1	106.1	>Range	3010	0.31	0.25	**NonLiq**	106.1	0.01		0.01	0.2
LB-4	28 to 33	30	5		7	120	80	2	1	52.0	3610	54.7	55.3	>Range	3610	0.31	0.25	**NonLiq**	55.3	0.01		0.01	0.1
LB-4	33 to 38	35	5		80	125	28	1	1.2	33.6	4223	32.7	44.2	>Range	4222.5	0.29	0.24	**NonLiq**	44.2	0.02		0.01	0.1
LB-4	38 to 43	40	5		80	125	24	2	1	15.6	4848	14.2	22.0	0.242	4847.5	0.28	0.23	**NonLiq**	22.0	0.15		0.09	0.1
LB-4	43 to 48	45	5		10	125	80	1	1.2	96.0	5473	82.0	84.7	>Range	5472.5	0.27	0.22	**NonLiq**	84.7	0.01		0.01	0.0
LB-4	48 to 52	50	5		80	130	47	2	1	30.6	6110	24.7	34.6	>Range	6110	0.25	0.21	**NonLiq**	34.6	0.06		0.03	0.0
LB-5	0 to 4	3	4		5	125	59	2	1	38.4	375	67.6	67.6	>Range	375	0.33	0.27	**NonLiq**	67.6	0.00		0.00	0.0
LB-5	4 to 8	5	4		5	120	80	2	1	52.0	620	91.7	91.7	>Range	620	0.33	0.26	**NonLiq**	91.7	0.00		0.00	0.0
LB-5	8 to 13	10	5		5	120	80	2	1	52.0	1220	80.0	80.0	>Range	1220	0.32	0.26	**NonLiq**	80.0	0.01		0.01	0.0
LB-5	13 to 18	15	5		8	125	80	2	1	52.0	1833	65.3	66.4	>Range	1832.5	0.32	0.26	**NonLiq**	66.4	0.01		0.01	0.0
LB-5	18 to 22	20	5		8	125	80	1	1.2	96.0	2458	116.3	118.1	>Range	2457.5	0.32	0.26	**NonLiq**	118.1	0.01		0.00	0.0

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*    E Q S E A R C H    *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 12482.001

DATE: 08-21-2019

JOB NAME: WLC Orange FS1

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE: 5.00
   MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
   SITE LATITUDE: 33.7873
   SITE LONGITUDE: 117.8411

SEARCH DATES:
   START DATE: 1800
   END DATE: 1999

SEARCH RADIUS:
   60.0 mi
   96.6 km

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND: 0 Depth Source: A
   Basement Depth: 5.00 km   Campbell SSR: Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.160	VIII	13.8(22.2)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.057	VI	13.9(22.3)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.084	VII	14.0(22.5)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.055	VI	14.1(22.7)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.060	VI	14.1(22.7)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.060	VI	14.1(22.7)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.071	VI	14.1(22.7)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.055	VI	14.1(22.7)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.059	VI	14.3(23.0)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.059	VI	14.3(23.0)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.054	VI	15.5(24.9)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.063	VI	16.7(27.0)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.054	VI	16.8(27.0)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.044	VI	17.3(27.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.056	VI	19.9(32.0)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.065	VI	23.3(37.5)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.042	VI	23.5(37.8)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.141	VIII	24.4(39.3)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.036	V	24.6(39.6)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.027	V	24.8(39.9)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.031	V	25.7(41.3)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.026	V	26.0(41.9)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.062	VI	26.0(41.9)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.026	V	26.0(41.9)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.079	VII	26.2(42.1)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.024	IV	27.7(44.5)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.024	IV	27.7(44.5)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.024	IV	27.7(44.5)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.035	V	28.7(46.2)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.028	V	30.1(48.5)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.020	IV	31.4(50.5)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.025	V	34.0(54.8)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.049	VI	36.9(59.4)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.022	IV	37.5(60.4)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.038	V	37.6(60.5)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.020	IV	37.8(60.8)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.038	V	38.0(61.1)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.024	IV	38.1(61.3)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.054	VI	40.4(65.0)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.014	IV	40.5(65.2)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.014	IV	40.5(65.2)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.076	VII	41.7(67.1)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.011	III	46.0(74.0)

DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.013	III	46.7(75.2)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.014	IV	47.5(76.4)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.039	V	48.3(77.7)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.054	VI	48.3(77.8)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.011	III	48.3(77.8)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.010	III	49.3(79.4)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.048	VI	49.5(79.7)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.012	III	50.2(80.8)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.025	V	51.1(82.2)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.009	III	52.9(85.1)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.012	III	53.7(86.3)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.019	IV	53.7(86.3)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.019	IV	53.7(86.3)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.033	V	53.7(86.3)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.009	III	54.5(87.7)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.011	III	54.5(87.7)
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.010	III	55.3(89.0)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.022	IV	56.0(90.1)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.011	III	56.4(90.7)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.008	III	57.9(93.2)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.009	III	59.3(95.4)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.009	III	59.3(95.4)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.012	III	60.0(96.5)

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-END OF SEARCH- 66 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 1999

LENGTH OF SEARCH TIME: 200 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 13.8 MILES (22.2 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.160 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.043
b-value= 0.349
beta-value= 0.803

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year

4.0	66	0.33166
4.5	66	0.33166
5.0	66	0.33166
5.5	23	0.11558
6.0	15	0.07538
6.5	5	0.02513
7.0	2	0.01005

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*    Version 3.00    *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 12482.001

DATE: 08-20-2019

JOB NAME: WLC Orange FS1

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.7873
SITE LONGITUDE: 117.8411

SEARCH RADIUS: 60 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELYSIAN PARK THRUST	8.6(13.9)	6.7	0.303	IX
WHITTIER	8.8(14.1)	6.8	0.243	IX
COMPTON THRUST	9.9(15.9)	6.8	0.287	IX
NEWPORT-INGLEWOOD (L.A.Basin)	12.2(19.6)	6.9	0.199	VIII
ELSINORE-GLEN IVY	12.6(20.3)	6.8	0.184	VIII
CHINO-CENTRAL AVE. (Elsinore)	13.0(20.9)	6.7	0.220	IX
NEWPORT-INGLEWOOD (Offshore)	14.2(22.9)	6.9	0.175	VIII
SAN JOSE	17.5(28.2)	6.5	0.147	VIII
PALOS VERDES	22.2(35.8)	7.1	0.129	VIII
SIERRA MADRE	23.9(38.4)	7.0	0.145	VIII
CUCAMONGA	24.2(39.0)	7.0	0.142	VIII
RAYMOND	27.7(44.5)	6.5	0.088	VII
CLAMSHELL-SAWPIT	28.5(45.8)	6.5	0.084	VII
VERDUGO	29.8(47.9)	6.7	0.092	VII
ELSINORE-TEMECULA	30.0(48.3)	6.8	0.076	VII
HOLLYWOOD	32.0(51.5)	6.4	0.067	VI
SAN JACINTO-SAN BERNARDINO	35.6(57.3)	6.7	0.058	VI
CORONADO BANK	36.2(58.2)	7.4	0.093	VII
SAN JACINTO-SAN JACINTO VALLEY	38.2(61.4)	6.9	0.061	VI
SANTA MONICA	38.4(61.8)	6.6	0.062	VI
SAN ANDREAS - San Bernardino	40.1(64.5)	7.3	0.077	VII
SAN ANDREAS - Southern	40.1(64.5)	7.4	0.082	VII
SAN ANDREAS - 1857 Rupture	40.3(64.9)	7.8	0.106	VII
SAN ANDREAS - Mojave	40.3(64.9)	7.1	0.066	VI
CLEGHORN	42.2(67.9)	6.5	0.040	V
SIERRA MADRE (San Fernando)	42.8(68.8)	6.7	0.058	VI
MALIBU COAST	43.1(69.4)	6.7	0.058	VI
SAN GABRIEL	44.5(71.6)	7.0	0.054	VI
NORTHRIDGE (E. Oak Ridge)	46.2(74.4)	6.9	0.061	VI
NORTH FRONTAL FAULT ZONE (West)	48.2(77.6)	7.0	0.063	VI
ANACAPA-DUME	50.9(81.9)	7.3	0.074	VII
ROSE CANYON	51.3(82.6)	6.9	0.042	VI
SANTA SUSANA	52.8(84.9)	6.6	0.040	V
SAN JACINTO-ANZA	53.2(85.6)	7.2	0.050	VI
ELSINORE-JULIAN	55.4(89.1)	7.1	0.044	VI
HOLSER	58.6(94.3)	6.5	0.032	V

-END OF SEARCH-   36 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELYSIAN PARK THRUST   FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 8.6 MILES (13.9 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3030 g

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

LEIGHTON CONSULTING, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 <u>General</u>

1.1 <u>Intent</u>: 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 <u>The Geotechnical Consultant of Record</u>: 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.