GEOTECHNICAL EXPLORATION
PROPOSED FIRE STATION NO. 68
SOUTH OF SOQUEL CANYON PARKWAY AND PIPELINE AVENUE, CITY OF CHINO HILLS
SAN BERNARDINO COUNTY, CALIFORNIA

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Project No. 13353.001

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Attention: Mr. Kelley Needham

Subject: Geotechnical Exploration
Proposed Fire Station No. 68
South of Soquel Canyon Parkway and Pipeline Avenue
Chino Valley Fire Protection District
City of Chino Hills, San Bernardino County, California

In accordance with our November 8, 2021 proposal, authorized on November 11, 2021, along with authorization for additional exploration on April 4, 2022, Leighton Consulting, Inc. (Leighton) has completed geotechnical exploration in support of design of the new Fire Station No. 68 for the Chino Valley Fire Protection District, to be constructed south of the intersection of Soquel Canyon Parkway and Pipeline Avenue in the City of Chino Hills, California. The purpose of our exploration was to evaluate geologic hazards and geotechnical conditions of the site with respect to the proposed improvements and to provide geotechnical recommendations for design of the proposed Fire Station No. 68 development.

This site is not located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. The site is located about 0.6 mile west of the Chino fault zone and does not require a fault study. However, as is the case for most of southern California, strong ground shaking has and will occur at this site.

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

We appreciate this opportunity to be of additional service to WLC Architects, Inc. If you have any questions or if we can be of further service, please contact us at your convenience at ***866-LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 <u>Site Location and Description</u>

As depicted on Figure 1, *Site Location Map*, this proposed fire station site is located in the City of Chino Hills, San Bernardino County, California (latitude 33.9583° and longitude -117.7149°). The existing approximate 3.1-acre undeveloped site (Lot H) is mapped as Assessor Parcel Numbers (APN) 1030-341-68 and a portion of 117-241-28 by the County of San Bernardino. This site is in a mass-graded state consisting of one superpad. The proposed Fire Station No. 68 building is planned to be constructed towards the northwestern portion of this superpad. The site is bounded by Soquel Canyon Parkway to the north and single-family residential home developments to the east and west. The southern portion of APN 117-241-28 outside of the proposed fire station site is also vacant.

Based on our review of aerial imagery dating back to 1938, the site location has remained vacant since the construction of the eastern residential homes and Soquel Canyon Parkway between the years of 1994 and 1998, where it appears excess material was placed during adjacent grading of the site. Between the periods of 2006 and 2007, installation of the existing 60-inch-diameter storm drain line took place within the site, along with the site grading of the western residential developments and Soquel Canyon Road's extension. The site appeared to have been used as a temporary storage yard for equipment during construction of the lots south of Oakley Circle in 2009, and appears to have had minor grading conducted. Since 2014, the site appears to have been used as a designated mud pit dump area within the storm drain easement area; the material appears to be stockpiled and spread across the eastern area of APN 1030-341-68.

This site is gently sloping towards the north-northeast to Soquel Canyon Parkway from an approximate elevation of 796 feet at the southwestern most part of the site to 766 feet in the northeast corner. Slopes from the adjacent residential home developments are located to the east and west, the largest adjacent slope being located on the west and on the order of approximately 25 feet in height and appears to be at a slope of 3:1 (H:V).

1.2 Proposed Fire Station No. 68

Based on the July 13, 2017, Proposed Site Layouts (Options 1 and 2) prepared by WLC Architects Inc., the approximate 3.1-acre site will accommodate a fire station building with an approximately 9,800-square-foot (SF) footprint in plan area. Both options (Options 1 and 2) depict the fire stations footprints within the same general location and similar size, however with different building orientation. The proposed fire station building will feature an apparatus bay in the center and include associated parking, drives, emergency generator, above ground fuel tank, hose tower, trash enclosure, sliding security gates, and perimeter walls.

At this time, structural loading of the proposed foundations has not been provided, but we assume the proposed building will be relatively lightly loaded, and we assume that the proposed building will have a concrete slab-on-grade and will consist of reinforced masonry or wood and/or cold-formed steel stud construction.

1.3 Previous Geotechnical Reports

As part of our investigation, we reviewed available literature for the site and surrounding areas. Some of these reports indicated the potential of landslides within the site of the proposed fire station building. Below is a summary of these reviewed reports.

Leighton and Associates, Inc. performed a preliminary geotechnical investigation for Tentative Tract 15898, which is located immediately to the west of the proposed fire station property, and provided a report dated June 2, 1998. Leighton and Associates identified an ancient landslide, referred to as “Qls 4”, within the eastern boundary of Tentative Tract 15898, and that landslide extended into the proposed fire station site. Leighton and Associates provided recommendations and mitigation options to stabilize the slope where the mapped landslide was mapped within Tentative Tract 15898. The proposed mitigation of the landslide was located along the eastern property boundary of Tract 15898 and included a 60-foot-wide shear key to a depth of 5 feet below the landslide rupture surface. Based on observation and testing during rough grading for Tentative Tract 15898 (Leighton and Associates, 2006), the bottom of the shear key ranged in elevation from 762 feet on its northern end to 768 feet above mean sea level (msl) on its southern end. During rough grading of Tentative Tract

15898, all landslide material was removed within that property, and we are unaware of any remedial removals that were performed within the proposed fire station site.

Medall, Aragon, Worswick, and Associates, Inc. (MAW) conducted a Preliminary Geotechnical Investigation in 1985 for Tentative Tract No. 13295, which is located to the northwest across Soquel Canyon Parkway from the proposed fire station. As a part of their investigation, MAW drilled a bucket auger boring in the area of the proposed fire station. Although MAW mapped the majority of the fire station site to be underlain by colluvium (Qcol), their bucket auger indicated a relatively small landslide within the proposed fire station site near the existing wing wall outlet.

Schaefer Dixon Associates, Inc. (SDA) provided a Geotechnical Report of Rough Grading for Tract 13295 Lots 1 through 154 in 1987. Tract 13295 is located to the northeast across Soquel Canyon Parkway from the proposed fire station. SDA reported rough grading within the eastern portion of the fire station site (outside of the areas proposed for structures). SDA reported soils in Tract 132958 mapped as artificial compacted fill (afc) over bedrock formation (Tpy) with smaller areas mapped as compacted artificial fill over older alluvium (Qoal). Based on the density test location maps provided in their report, SDA mapped the native earth materials in the area proposed for fire station structures as being composed of older alluvium.

Eberhart and Stone, Inc.'s 1994, (E&S) provided a Supplemental Geotechnical Investigation and Grading Plan Review for Tract 13601 in 1994. Tract 13601 is located immediately east of the proposed fire station property. E&S included the existence of a landslide within the proposed fire station property. No evidence from subsurface exploration was provided by E&S, and presumably this landslide was mapped based on previous mapping by others or mapping from the surface by E&S.

1.4 Purpose and Scope of Exploration

Purpose of our exploration was to: (1) evaluate geotechnical conditions of the site of the proposed Fire Station No. 68 with respect to the proposed improvements, (2) identify significant geotechnical or geologic issues that would impact proposed structures, and (3) provide geotechnical recommendations for design and construction of proposed building and associated improvements as currently planned. The scope of our exploration included the following:

- **Research**: We reviewed readily available geotechnical literature, reports and aerial photographs relevant to this site. Pertinent geotechnical documents are referenced at the end of this report text.

- **Field Exploration**: On November 19, 2021, six (6) hollow-stem auger borings were drilled with a truck-mounted rig, logged and sampled to depths ranging from approximately 10 feet to 47 feet below the existing ground surface (bgs). Water infiltration testing was performed on boring LB-5. After sampling, logging, and testing, all borings were immediately backfilled. Additional exploration for the proposed Fire Station No. 68 was performed on April 25th and 26th, 2022, which included three (3) large-diameter borings drilled with a Lodril-mounted excavator, sampled and down-hole logged to depths ranging from approximately 43½ to 56 feet bgs. Approximate boring locations are depicted on Figure 2, *Geotechnical Map*. A description of encountered soil conditions are presented in our boring logs in Appendix A, *Field Exploration*.

- **Geotechnical Laboratory Testing**: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field exploration. Our laboratory testing program was designed to evaluate engineering characteristics of onsite soils. A description of test procedures and results are presented in Appendix B, *Geotechnical Laboratory Testing*.

- **Engineering and Geologic Analysis**: Data obtained from field exploration and geotechnical laboratory testing were evaluated and analyzed to develop geotechnical conclusions and provide recommendations in general accordance with the California Geological Survey (CGS) Note 48.

- **Report Preparation**: Results of our geologic hazards review and geotechnical exploration have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical design recommendations.

This report does not address the potential for encountering hazardous materials in site soils or within groundwater. Important information about limitations of geotechnical reports in general, is presented in Appendix D, *GBA's Important Information About This Geotechnical-Engineering Report*.

2.0 FINDINGS

2.1 Regional Geologic Setting

This site is located in the northwestern portion of the Peninsular Ranges Geomorphic Province of southern California in the eastern Puente Hills. This is an area where the lateral strain of the Elsinore Fault Zone to the south is accommodated by the faults and folds bounding and within the east-west trending Puente Hills.

The Puente Hills are a structural block, north of the Whittier fault and southwest of the Chino fault, that uplifted and emerged in the Pleistocene. This uplift is a result of north-south compression that has been accommodated by the Puente Hills blind thrust fault (Grant and Gath, 2007). The relief of the Puente Hills is a result of a history of uplift and erosion. During Quaternary uplift, erosion rates of the streams in the Puente Hills increased, and gullies were incised in existing broad canyons. These gullies decrease in depth upstream, and, in general, streams that flow towards the southwest are longer than those flowing to the north and northeast. This pattern of gully depth and the asymmetrical pattern of the older broad canyons indicates that the Puente Hills block tilted towards the northeast during Quaternary uplift (Durham and Yerkes, 1964).

The dominant structural features in the eastern Puente Hills region are the Whittier fault and the Chino fault. This area of Southern California has and is continuously experiencing major crustal disturbance as the site is located relatively near the boundary between the Pacific and North American Plates. The bulk of the generally right-lateral transform movement between the two major tectonic plates occurs along the San Andreas fault and associated faults such as the Elsinore and San Jacinto faults.

2.2 Subsurface Soil Conditions

Based on results of our research and subsurface exploration, the encountered site soils to the depths explored consisted of the following:

- **Undocumented Fill (Afu)**: We are unaware of any fill placement documentation for Lot H, so we have identified encountered fill as undocumented. Undocumented artificial fill was observed in all of our borings drilled during this exploration. The overall fill thickness of undocumented fill encountered within our borings ranged in depth from approximately 20 feet to

25 feet below the existing ground surface. Sampled fill was predominantly very stiff to hard, sandy clay and clay with moisture contents ranging from 11 to 31 percent moistures. Trace construction debris was visible in the fill encountered in the upper 2 to 4 feet of borings LB-1, LB-3, and LB-6. Where observed down-hole (large-diameter borings BA-1 through BA-3), undocumented fill appeared to be firm based on hammer blows throughout its entire thickness. Undocumented fill was relatively uniform in appearance and texture where observed down-hole, except in the upper 1 to 1½ feet where undocumented fill was observed to be dry and desiccated. In all three large-diameter borings, the bottom of the undocumented fill was observed to have a clean (no debris or organics), sharp contact with intact bedrock of the Puente Formation below.

In situ moisture and density laboratory testing was performed in recovered samples within undocumented fill. In situ dry densities on tested undocumented fill ranged from 92 pcf to 114 pcf. In situ moisture content of recovered samples ranged from 8 to 23 percent. Compaction was calculated relative to the modified Proctor maximum laboratory density determined in accordance with Standard Test Method ASTM D 1557 from representative soil samples collected during subsurface exploration during this study. Proctor compaction from the samples collected by Leighton for this study are summarized in the table below:

Table 1. Summary of Maximum Dry Density Test Results

Soil Type	**Maximum Dry Density (pcf)**	**Optimum Moisture (% H_2O)**
Dark Olive Lean Clay (CL)	115.5	12.5
Brown Clayey Sand (SC)	110.0	15.5
Dark Brown Clayey Sand (SC)	107.2	14.8

All collected samples in borings where structures are proposed onsite (LB-2, LB-3, LB-4, BA-1, BA-2, and BA-3) indicated a relative compaction of at least 90 percent at depths below approximately 11.5 feet below the current surface. Collected samples of onsite fill below a depth of 11.5 feet were tested to have moisture contents of at least 1.5 percent above optimum. Based on N-values (ranging from 16 to 39) interpreted from blow counts measured during sample collection in hollow-stem auger borings LB-1, LB-2, LB-3, and LB-4, the consistency of the samples of fill with relative compaction less than 90 percent was stiff to very stiff. Observations made during down-hole logging in undocumented fill indicated firm soils below depths of 11.5 feet based on hammer blows and visual appearance.

- **Puente Formation (Tsh)**: Sedimentary bedrock comprised of a dark gray claystone of the Puente Formation was encountered in our borings at approximate depths of 20 to 25 feet bgs. Unoxidized claystone was observed at a depth of approximately 35 feet bgs within our large-diameter borings (BA-1 through BA-3). This unoxidized claystone was logged to the total depth of each large-diameter boring. Puente Formation bedrock encountered was described as moderately hard to hard.

More detailed descriptions of subsurface soils encountered are presented on our boring logs in Appendix A.

2.3 Groundwater

Minor groundwater seepage was encountered at a depth of 41 feet bgs within one of our six borings (LB-3) drilled to a maximum depth of 47 feet below existing ground surface on November 9, 2021. Perched groundwater in the form of moderate seepage from small fractures within the bedrock formation was also encountered within two of our three drilled bucket auger borings (BA-1 and BA-3) at depths ranging from 33 to 40 feet.

The bedrock onsite is not generally considered water-bearing, and a review of the Geohydrology Maps of the Chino-Riverside Area (CDWR, 1933) dating back to 1933 indicated that the site is in an area of nonwater-bearing rocks.

Groundwater was not encountered within onsite fill during exploration and is therefore not expected to be encountered during construction activities for the proposed fire station.

2.4 Faulting and Seismicity

Southern California is a seismically active area. As such, the site will be subject to seismic hazards from numerous sources in the area. The severity of potential seismic hazards is related to site-specific geology, distances from seismic sources, and the magnitude of earthquake events. Principal seismic hazards evaluated on a site-specific basis included: potential for surface rupture along active or potentially active fault traces, magnitude of seismic shaking, and the susceptibility to ground failure (liquefaction, lurching, and seismically induced landslides). The potential for fault rupture and seismic shaking are discussed below.

2.4.1 **<u>Surface Faulting</u>** Fault classification criteria adopted by the California Geological Survey, formerly the California Division of Mines and Geology, defines Earthquake Fault Zones along active or potentially active faults. The California Alquist-Priolo Earthquake Fault Zoning Act of 1972 classification system is used in this report, as follows:

- **Active**: An active fault is one that has ruptured within the Holocene epoch (the last 11,700 years).
- **Potentially Active**: A fault that has ruptured during the last 1.8 million years (Quaternary period), but has not been proven by direct evidence to have not moved within the Holocene epoch is considered to be potentially active.
- **Inactive**: A fault that has not moved during both Pleistocene and Holocene epochs (that is no movement within the last 1.8 million years) is considered to be inactive.

Based on our review of available in-house literature, and as depicted on Figure 4, there are no currently known active surface faults that traverse or trend towards this site, and this site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (CGS, 1995), or a fault zone delineated by the County or City.

The closest know active or potentially active faults are the Chino fault located approximately 0.6 miles east of the site, and the Whittier fault located approximately 5.2 miles southwest of the project site. The known regional active or potentially active faults that could produce the most significant ground shaking at the site include the Chino, Whittier, and Yorba Linda faults. Nearby faults are depicted in Figure 4 – *Regional Fault and Historical Seismicity Map*.

2.4.2 **<u>Seismicity (Ground Shaking)</u>:** A principal seismic hazard that could impact this site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults throughout southern California. An evaluation of historical seismicity from significant past earthquakes related to the site was performed. Plotted on Figure 4, *Regional Fault and Historic Seismicity Map*, are epicenters of historic earthquakes (1769 through 2016) in and around Chino Hills, color coded as a function of magnitude. 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 that 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.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in this region could include soil liquefaction, earthquake-induced settlement, slope instability and landslides, earthquake-induced seiches and tsunamis flooding. Site-specific potential for secondary seismic hazards is discussed in the following subsections:

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

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

The State of California and the County of San Bernardino has not prepared a map delineating zones of liquefaction potential for the quadrangle that contains the site. Perched groundwater was encountered in one of our drilled borings at a depth of 41 feet bgs at the approximate bedrock contact depth, and collected data indicated that groundwater depths at and near this site have been historically 100 feet deep beneath the site or more. In addition, encountered fine-grained undocumented artificial fill soils onsite were generally very stiff to hard, and relatively shallow bedrock was encountered in our deeper borings. Based on the absence of shallow groundwater and the dense nature of the onsite soils and generally shallow bedrock, liquefaction is unlikely to occur at the site.

2.5.2 **Lateral Spreading:** Lateral spreading is unlikely to occur at the site due to the lack of liquefaction potential and lack of significant topographic relief at and around this site.

2.5.3 **Seismically Induced Settlement:** During a strong seismic event, non-liquefaction, seismically induced settlement can occur within loose and dry granular soils. Settlement caused by ground shaking is often unevenly distributed, which can result in differential settlement. Fill soils are typically highly susceptible to seismically induced settlement. Undocumented fill soils under the proposed building footprint are recommended (discussed later in this report) to be recompacted to mitigate dynamic settlement concerns.

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 suggested that the onsite soils are susceptible to less than 1 inche of seismic settlement based on the MCE. Differential settlement due to seismic loading is assumed to be 1/2 inch over a horizontal distance of 30 feet based on the MCE. A summary of seismic settlement analysis is included in Appendix C.

2.5.4 **Slope Instability and Landslides:** Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The State of California and the County of San Bernardino has not prepared a map delineating zones of landslide potential for the quadrangle that contains the site. However, the site and vicinity are gently sloping. The potential for seismically induced landslide activity is considered negligible for this site due to the lack of significant slopes.

This site is gently sloping towards the north-northeast to Soquel Canyon Parkway from an approximate elevation of 796 feet at the southwestern most part of the site to 766 feet in the northeast corner. Slopes from the adjacent residential home developments are located to the east and west, the largest adjacent slope being located on the west and on the order of approximately 25 feet in height and appears to be at a slope of 3:1 (H:V). The geologic structure observed in the two bucket auger borings showed bedrock bedding angles dipping into slope, which is a favorable condition. The descending slopes adjacent to the site are anticipated to be grossly stable.

2.5.5 **Earthquake-Induced Seiches and Tsunamis:** Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are predominately ocean waves generated by undersea large magnitude fault displacement or major ground movement.

Based on separation of the site from any enclosed body of water, there is no seiche impact at the site. Also, due to average site elevation of -feet above mean sea level and the inland location of this site relative to the Pacific Ocean tsunami risks at this site is nil.

2.5.6 **Earthquake-Induced Inundation:** This inundation hazard is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Figure 5, *Dam Inundation Map*, shows an area of dam inundation approximately 8,000 feet northeast of the site. The subject site is not mapped within a dam breach inundation zone.

2.6 Storm-Induced Flood Hazard

As depicted on Figure 6, *Flood Hazard Zone Map*, this site is not mapped within a "100-year" or "500-year" flood zone as defined by the Federal Emergency Management Agency's (FEMA's) Flood Insurance Rate Map (FIRM).

2.7 Infiltration Testing

Infiltration testing was conducted within one of our borings onsite (LB-5) to estimate the infiltration characteristics of the onsite soils at the depths tested. The infiltration testing was conducted at a bottom test zone depth of approximately 10 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. A test consists of excavating a boring to the depth of the test (or deeper as long as 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 then placed in the boring bottom to temporarily support a perforated well casing pipe and a float valve system. Once the well casing pipe has been installed, coarse sand or gravel is poured in the annular space outside of the well casing within the test zone to prevent the boring from caving/collapsing or spalling when water is added. The float valve is lowered into the boring inside the casing, which will control the water added into the boring as water within the boring infiltrates into the soil, maintaining a relatively constant water head. 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 test yielded negligible infiltration rates within the onsite clay soils. As the encountered onsite soils consisted of fined grained soils to the maximum explored depths of 47 feet bgs, infiltration at the site is not feasible. Results of infiltration testing are provided in Appendix B.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

This site is not located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. However, as is the case for most of southern California, strong ground shaking has and will occur at this site. Groundwater levels are on the order of 100 feet below the surface or deeper based on available well data. Encountered onsite soils were stiff to hard fine grained soils and shallow fine grained bedrock; therefore liquefaction potential is very low at this site. Near-surface onsite clay soils have medium to high expansion potential.

3.1.1 Existing Fill and Subgrade Conditions

Based on conditions observed during drilling, down-hole logging, and laboratory test results, existing fill onsite is potentially compressible in the upper 11.5 feet from the current surface. Existing fills onsite below 11.5 deep were moist and firm based on hammer blows and visual appearance observed down-hole and an evaluation of relative compaction based on in situ dry densities relative to laboratory proctor compaction of representative soil samples. All relatively undisturbed samples of fill collected below a depth of 11.5 feet below the surface were evaluated to have a relative compaction of 90 percent or greater based on modified Proctor (ASTM D1557) test results.

The contact between fill and underlying subgrade earth materials was observed down-hole in borings BA-1 to BA-3 to be clean (no debris or organics) and sharp with intact bedrock of the Puente Formation below. The contact was observed to be approximately level, and undulated a few inches within each boring. Based on this observation, it appeared that debris, organics, compressible soil and weathered bedrock had been removed prior to placement of overlying fill materials.

Bedrock below fill was observed to be intact and had consistent structure throughout. Additionally, unoxidized claystone was observed at a depth of approximately 35 feet bgs within our large-diameter borings (BA-1 through BA-3). This unoxidized claystone was logged to the total depth of each large-diameter boring, which extended to elevations lower than the toe of landslides mapped during previous geotechnical reporting (MAW, 1985 and E&S, 1994).

Additionally, no evidence of basal landslide rupture surface was observed during down-hole logging. Based on these, the subgrade of existing fill materials onsite appears to be intact Puente Formation bedrock and not ancient landslide debris. In addition, the bedrock bedding is oriented favorably with respect to the adjacent minor slopes.

3.2 Recommendations Summary

We are unaware of any fill placement documentation for Lot H within Tracts 13601 where the proposed fire station building footprint is proposed to be located. Based upon our geotechnical exploration and analysis, existing undocumented fill soil within proposed structural footprints should be excavated and recompacted to provide more uniform shallow foundation support. Overexcavation should extend a minimum of 11.5 feet below existing grade, or a minimum of 3 feet below proposed footings, whichever is deeper, within building footprints. The onsite soils are anticipated to exhibit a medium to high expansion potential; as such the proposed fire station should be founded on stiffened foundations. This may include a post-tension foundation system designed in accordance with the California Building Code (CBC) bearing solely on a zone of newly excavated and recompacted fill soils derived from onsite soils, overlying solely undisturbed clays.

Geotechnical recommendations for the proposed Fire Station 68 site are presented in the following subsections.

3.3 Earthwork

Project earthwork is expected to include overexcavation and recompaction of undocumented fill soils and onsite alluvium soils below the proposed new building footprint as described in the following subsections:

3.3.1 Earthwork Observation and Testing: Leighton should observe and test all grading and earthwork to check that the site has been properly prepared, to assess that selected fill materials are satisfactory, and to evaluate that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications. Any imported soil or aggregate material to be evaluated for its suitability as onsite fill material should be submitted to a Leighton geotechnical laboratory at least two working days in advance of earth material placement and compaction. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, we should be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Conclusions and recommendations presented in this report are contingent upon construction geotechnical observation services.

3.3.2 **Surface Drainage:** Water should not be allowed to pond or accumulate anywhere except in approved drainage areas, which should be set back at least 15 feet from proposed structures. Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns and drainpipes approved at the time of fine grading should be maintained throughout the life of proposed structures. Percolation or stormwater infiltration should not be allowed within at least horizontal 15 feet of the proposed Fire Station 8 building.

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

Based on encountered site conditions, we recommend that existing fill should be excavated from proposed structural footprints, to a minimum of 3 feet below the bottoms of proposed footings or at least 11.5 feet below existing grade, whichever is deeper. Overexcavation bottoms should extend horizontally either the thickness of fill below finish grade or at least 5 feet horizontally beyond the outside edges of proposed building perimeter footings, whichever is greater, encompassing the whole new building footprint, including attached columns. Any underground obstructions encountered should be removed. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted where interfering with proposed construction.

Areas outside proposed building footprint limits planned for asphalt and/or concrete pavement should be overexcavated to a minimum depth of 24 inches below existing or finish grade, or 12 inches below proposed pavement sections; whichever is deeper.

Resulting removal excavation bottom surfaces should be observed by Leighton prior to placement of any backfill or new construction. It is essential that exposed existing fill soils to remain below the proposed building footprints be tested for proper compaction and moisture content. Deeper overexcavation may be required if unsatisfactory existing artificial fill soil is encountered below the minimum overexcavation depths noted above. After overexcavations are completed and tested and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to 4 percent above optimum moisture content, and recompacted to a minimum 90 percent relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve).

3.3.4 Fill Placement and Compaction: Onsite soils free of organics and debris are suitable for use as compacted structural fill provided they are free of oversized material greater than 8 inches in its largest dimension. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill should be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 3 percent above optimum moisture content, and compacted to a minimum 90% relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve) within the building footprint. Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction.

3.3.5 Shrinkage or Bulking: Volume change of excavated on-site fill soils, upon recompaction, is a function of current in-situ density; which is expected to vary significantly with material type (e.g. undocumented fill, alluvium, etc.). This means and methods choice will have a significant impact on estimated bulking or shrinkage.

Particularly in undocumented fill soils, in-place densities vary significantly and accurate overall determination of shrinkage and bulking cannot be made. Therefore, we recommend site grading include, if possible, a balance area or ability to adjust grades slightly to accommodate some variation. Based on our limited geotechnical laboratory testing, as a starting point, we expect shrinking when existing fill materials have been recompacted to minimum 90 percent of ASTM D1557 modified Proctor laboratory maximum density) of approximately 1 to 5 percent by volume within the upper 11.5 feet.

3.3.6 **<u>Pipeline Backfilling</u>:** Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition standards. Backfill in and above the pipe zone should be as follows:

- **Pipe Zone**: Pipe bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). Due to expansive clays, sand bedding for conduits should **<u>not</u>** be allowed on this site within the building footprint and at least 4 feet beyond the building footprint. CLSM bedding should be placed to 1 foot over the top of the conduit, and vibrated. CLSM should not be jetted. In areas outside of the building, sand with a minimum sand equivalent of 30 may be used as pipe bedding and shading; this material should be densified by mechanical means to a minimum of 90 percent relative density (ASTM D 1557); jetting and water densification should not be used; gravel should not be used as pipe bedding or shading, unless special provisions are made so that surrounding soils are not able to erode into the gravel, such as wrapping the gravel in geotextile filter fabric or using a mix that is filter compatible with the onsite clays; bedding sand should be observed and tested by Leighton.

- **Over Pipe Zone**: Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material greater than 3 inches in largest dimension. As an option, the whole trench can be backfilled with two-sack CLSM same as presented above for the pipe bedding zone. Oversized rock (cobbles and/or boulders) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone. Gravel larger than ¾ inch in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90% relative compaction (relative to the laboratory modified Proctor maximum dry density), relative to the ASTM D1557 laboratory maximum dry density within the building footprint and hardscape areas, or 85% under landscape areas. Backfill above the pipe zone should not be flooded or jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton.

3.4 Seismic Design Parameters

The site will experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

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

Table 2. 2019 CBC Site-Specific Seismic Parameters

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude: 33.9583, -117.7149	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	C
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.947 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.684 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.200
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.400
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	2.336 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	0.957 g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.557 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.638 g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.836 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), F_{PGA}	1.200
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	1.003 g

A Site Class analysis is included in Appendix C in accordance with ASCE 7-16 Chapter 20. Soil data below 50 feet was estimated using Standard Penetration Test (SPT) blowcounts from our deepest boring terminating in bedrock.

3.5 Foundations

Based on our preliminary exploration and our experience in the region, onsite soils exposed at pad grade will exhibit medium to high expansive potential. As such we recommend that the proposed structures be constructed using stiffened foundations, this may include a post-tension foundation system designed in accordance with the California Building Code (CBC). Anticipated foundation loads were not available during preparation of this report. We assumed maximum column dead loads up to (≤) 50 kips and wall loads of 3 kips per lineal foot for our preliminary foundation recommendations. Overexcavation and recompaction of footing subgrade soils should be performed as detailed in Section 3.3 of this report. Post-tension foundation recommendations are provided in the following section.

3.5.1 **Post Tension Foundation Design Parameters:** Post-tensioned foundations should be designed by a qualified structural engineer in accordance with the 2019 CBC using the minimum geotechnical parameters provided below for soils with a medium Expansion Index. Expansion index should be confirmed upon completion of grading. While we do not expect this value to change, expansion index (EI) should be confirmed upon completion of grading.

Post-tensioned Foundation Design Recommendations		
Edge Moisture Variation, e_m	Center Lift	8.4 feet
	Edge Lift	4.3 feet
Differential Swell Y_m	Center Lift	1.0 inch
	Edge Lift	1.5 inch
Modulus of Subgrade Reaction		100 pci

For post-tension slab foundations, exterior footings (thickened edges) should have a minimum depth of 24 inches below the lowest adjacent soil grade and a minimum width of 12 inches. These footings may be designed for a maximum allowable bearing pressure of 1,800 pounds per square foot. The allowable bearing pressure may be increased by one-third for short-term loading. A lateral sliding coefficient of 0.30 may be used in the design. The recommended slab design parameters are based on the Post-Tensioning Institute Design of Post-Tensioned Slabs-on Ground, 3rd Edition with 2008 supplement (PTI DC10.1-08). The structural engineer should also design the post-tensioned slabs with adequate stiffness to minimize potential cracking in the slabs.

To provide more uniform moisture in the subgrade, the top 18 inches of the prepared subgrade should be pre-saturated to 120 percent of the optimum moisture prior to placement of concrete.

The Post-Tensioning Institute (PTI) has recommended the following guidelines:

- Initial landscaping should be done on all sides adjacent to the foundation. Positive drainage away from the foundation should be implemented and maintained.
- Irrigation watering should be done in a uniform manner as equally as possible on all sides of the foundation to maintain constant soil moisture

content. Ponding of irrigation or rainfall water adjacent to the foundation slab can cause differential soil moisture levels potentially leading to differential movements.

- Planting trees closer to the structure than a distance equal to one-half the mature height of the tree could allow the root system to enter under the foundation. The root system could alter the soil moisture content within the soil and cause soil shrinkage, which may lead to differential movements of the foundation. A landscape architect should be consulted and made aware of these recommendations.

Based on the time of year and characteristics of fill material observed during our investigation, near surface soils to a depth of at least 2 feet will dry rapidly during hot windy weather and do not meet minimum optimal moisture conditions. Therefore, it is critical to long term performance of the foundations that the soil-moisture prior to construction and around the immediate perimeter of the slab after construction be maintained at 2 percent above optimum moisture content up through occupancy of the homes. All fill soils should be compacted to a minimum of 90 percent relative compaction.

3.6 Concrete Slab-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2019 CBC requirements. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Slab-on-grade subgrade soil should be moisture conditioned to or within 3% over optimum moisture content, to a minimum depth of 24 inches within building footprints, and compacted to 95% of the modified Proctor (ASTM D1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.

- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 5 inches should be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 30 inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 175 pounds per square inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾ inch.

- **Slab-On-Grade Control Joints**: Slab-on-grade crack control joint locations and spacing should be designed by the project Structural Engineer (SE). We suggest control joints of 12 feet on center. Control joints should form square panels.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

3.7 Sulfate Attack and Ferrous Corrosion Protection

3.7.1 Sulfate Exposure: Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2019 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 3. Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	ACI 318-14 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

3.7.2 **<u>Ferrous Corrosivity</u>:** Many factors can modify corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "*Effects of Soil Characteristics on Corrosion*" (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as follows:

Table 4. Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in modifying corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

3.7.3 Corrosivity Test Results: To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Table 5. Results of Corrosivity Testing

Locations	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
Boring LB-4	0 - 5	83	51	7.5	1,150

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- **Sulfate Exposure**: Based on our previous experience and Table 19.3.1.1 of ACI 318-14, in our opinion, sulfate exposure should be considered "negligible" with an Exposure Class S0 for native silty sands sampled at the site. Based on Table 19.3.2.1 of ACI 318-14, for this Exposure Category S0, there would be no restrictions on cement type ("cementitious material") nor water/cement ratio, and an f_c' (28-day compressive strength) of at least 2,500 pounds per square inch (psi) is required at a minimum for structural concrete.
- **Ferrous Corrosivity**: As shown above, minimum soil resistivity of 1,150 ohm-centimeters was measured in our laboratory test. In our opinion, it appears for site soils that corrosion potential to buried steel may be characterized as "severely corrosive" at the site Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials.

3.8 Lime Treatment Recommendations

Due to the expansive nature of the subgrade soil at the site, chemical alteration of the subgrade with lime can be used to stabilize subgrade soils within proposed building footprint and hardscapes. The addition of lime can reduce the swell potential of expansive clays. The treated soil can also act as a moisture barrier between the pavement sections and untreated subgrade soil.

We recommend the use of lime treatment to chemically alter the subgrade expansive soils. Subgrade soils should be mixed uniformly with a minimum of 4% dry weight lime to a minimum depth of 12 inches below subgrade elevation. In addition, the lime should be mixed and placed in accordance with Greenbook Section 301-5. The lime mixtures should then be placed and compacted at a minimum 95 percent relative compaction in accordance with ASTM Test Method D 558 with moisture content to at least 3 percentage points above optimum moisture content. Compacted layer thickness should not exceed 8 inches. The lime stabilized layer should be allowed to mellow after mixing for a minimum of 48 hours prior to final compaction. The surface of the compacted lime stabilized layer shall be kept moist until covered by the pavement section. Subgrade preparation and lime treatment should be performed by a contractor with the proper equipment and experience in this application. The use of lime to modify the soils may impact the construction of Portland cement concrete improvements that are in contact with the modified soils. The modified materials may necessitate the use of special cement (Type V) and concrete mix designs that will provide greater resistance to chemical attack from soluble sulfates as described in the building code.

Although lime subgrade stabilization is addressing near-surface expansive clays, water intrusion from surface drainage, rainfall and other sources may enter underlying expansive soils and cause potential swelling of the untreated soils. To reduce the potential for moisture intrusion into subgrade soils, the following considerations are provided:

- Broken/leaking irrigation sprinklers should quickly be shut off and repaired.
- Runoff generated from outlets from building down drains should be directed away from flatwork areas.
- Positive drainage away from structures.
- Vertical moisture barriers/cutoffs may be considered to be installed adjacent to landscape planters and open areas adjacent to pavement areas to prevent water from entering into the pavement and building subgrade.
- Moisture intrusion into underlying soils may occur throughout the life of the project, therefore periodic repairs and maintenance of flatwork/pavement cracks may be required.

It is also recommended that the subgrade soil be lime treated prior to the rainy season, as the treated soil acts as a moisture barrier to the underlying soils and provides a stable working surface; if left opened during the rainy season, the soil may become unstable and expand due to rainfall.

3.9 Pavement Section Design

Based on design procedures outlined in the 2017 Caltrans *Highway Design Manual* and an R-value of 6 for clay subgrade based on laboratory testing, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below:

Table 6. Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0 (automobile parking, driveways)	3.0	9.5
6.0 (truck traffic)	4.0	11.5
7.0 (roadways and heavy truck traffic)	5.5	12.5

For fire truck (60,000-pound "apparatus") lanes, asphalt pavements designed for a TI=6.0 are recommended. However, note that undistributed 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.

Onsite clays are medium to highly expansive, and R-value test on a near-surface sample was low (R<10). A subgrade sample can be collected during grading to perform additional R-Value tests to verify pavement design. Alternatively, in order to reduce the pavement section thicknesses provided in the table above, the subgrade soils can be cement-treated. We recommend that additional samples be collected during construction for R-Values testing, and also to determine the ideal cement treatment ratio (typically on the order of 3 to 5 percent). The following table provides pavement sections for cement-treated subgrade.

Table 7. Hot Mixed Asphalt (HMA) Pavement Sections Cement Treated Subgrade

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0 (automobile parking, driveways)	3.0	5.5
6.0 (truck traffic)	3.5	7.5
7.0 (roadways and heavy truck traffic)	4.0	9.5

*Subgrade treated with 3%-5% cement.

Portland cement concrete (PCC) 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 8. Portland Cement Concrete Pavement Sections

Assumed Traffic Index	PC Concrete (inches)	Base Course (inches)
5.0 (automobile parking, driveways)	6	4
6.0 (roadways and truck traffic)	7.5	

We have assumed that this Portland cement concrete will have a compressive strength of at least 3,000 psi. Prior to placement of aggregate base, subgrade soils should be scarified to a minimum depth of 8 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction, determined in accordance with ASTM D1557 modified Proctor laboratory maximum density. Aggregate base should be placed in thin lifts; moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction. Field observation and periodic testing, as needed during placement of base course materials, should be undertaken to ensure that requirements of Caltrans' *Standard Specifications* (2015) and Special Provisions are fulfilled. Consideration should be given to reinforce concrete pavements where large outrigger point loads are anticipated.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. All pavement construction should be performed in accordance with the current Caltrans *Standard Specifications* or *Standard Speciffications for Public Works Construction* ("Greenbook"). Recommended structural pavement materials should conform to the specified provisions in the Caltrans *Standard Specifications* (2015) including grading and quality requirements, shown below:

- **Asphalt Concrete (Hot Mixed Asphalt)** for pavement should be Type A and should conform to Section 39 of the *Standard Specifications*. Asphalt concrete specimens should be tested for surface abrasion in accordance with CT-360.
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.
- **Class II Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.

Traffic Indices (TIs) used in our pavement design are considered reasonable values for typical parking lot areas, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving, will result in premature pavement failure. Traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel-load analysis or a traffic study. The project Civil Engineer should confirm the TI assumptions.

3.10 Retaining Wall Recommendations

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 7, *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 9. Retaining Wall Design Parameters

Static Equivalent Fluid Pressure (pcf)	
Condition	Level Backfill
Active	40
At-Rest (drained, compacted-fill backfill)	60
Passive (allowable)	240 (Max. 3,000 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 0.30 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 51 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 18 inches below the lowest adjacent grade. An allowable bearing pressure of 1,800 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3,000 psf.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Excavations

Based on our field observations, caving of cohesionless and loose fill soils will likely be encountered in unshored trench excavations. To protect workers entering excavations, excavations should be performed in accordance with OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders, see:

http://www.dir.ca.gov/title8/sb4a6.html

Contractors should be advised that fill soils should initially be considered Type C soils as defined in the California Construction Safety Orders. As indicated in Table B-1 of Article 6, Section 1541.1, Appendix B, of the California Construction Safety Orders, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. This may be impractical near adjacent existing utilities and structures; so shoring may be required depending on trench locations. Stiff undisturbed native clays will stand steeper.

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

4.2 Temporary Shoring

Temporary cantilever shoring can be designed based on the active equivalent fluid pressure of 40 pounds-per-cubic-foot (pcf) in alluvium. If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure expressed in pounds-per-square-foot (psf), may be assumed to be 20 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 15 feet in depth at this site.

4.3 Geotechnical Services During Construction

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading, foundation and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- 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, and/or
- If and when any unusual geotechnical conditions are encountered.

5.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please also refer to Appendix C, *GBA's Important Information About This Geotechnical-Engineering Report*, presenting additional information and limitations regarding geotechnical engineering studies and reports.

Until reviewed and accepted by the reviewing government agency, this report may be subject to change. Changes may be required as part of the review process. Leighton Consulting, Inc. assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted.

This report was prepared for WLC Architects, Inc., based on their needs, directions and requirements at the time of our exploration, in accordance with generally accepted geotechnical engineering practices at this time in Chino Hills for public sites. This report is not authorized for use by, and is not to be relied upon by, any party except WLC Architects Inc., and their design and construction management team, with whom Leighton Consulting, Inc. has contracted for this work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc.

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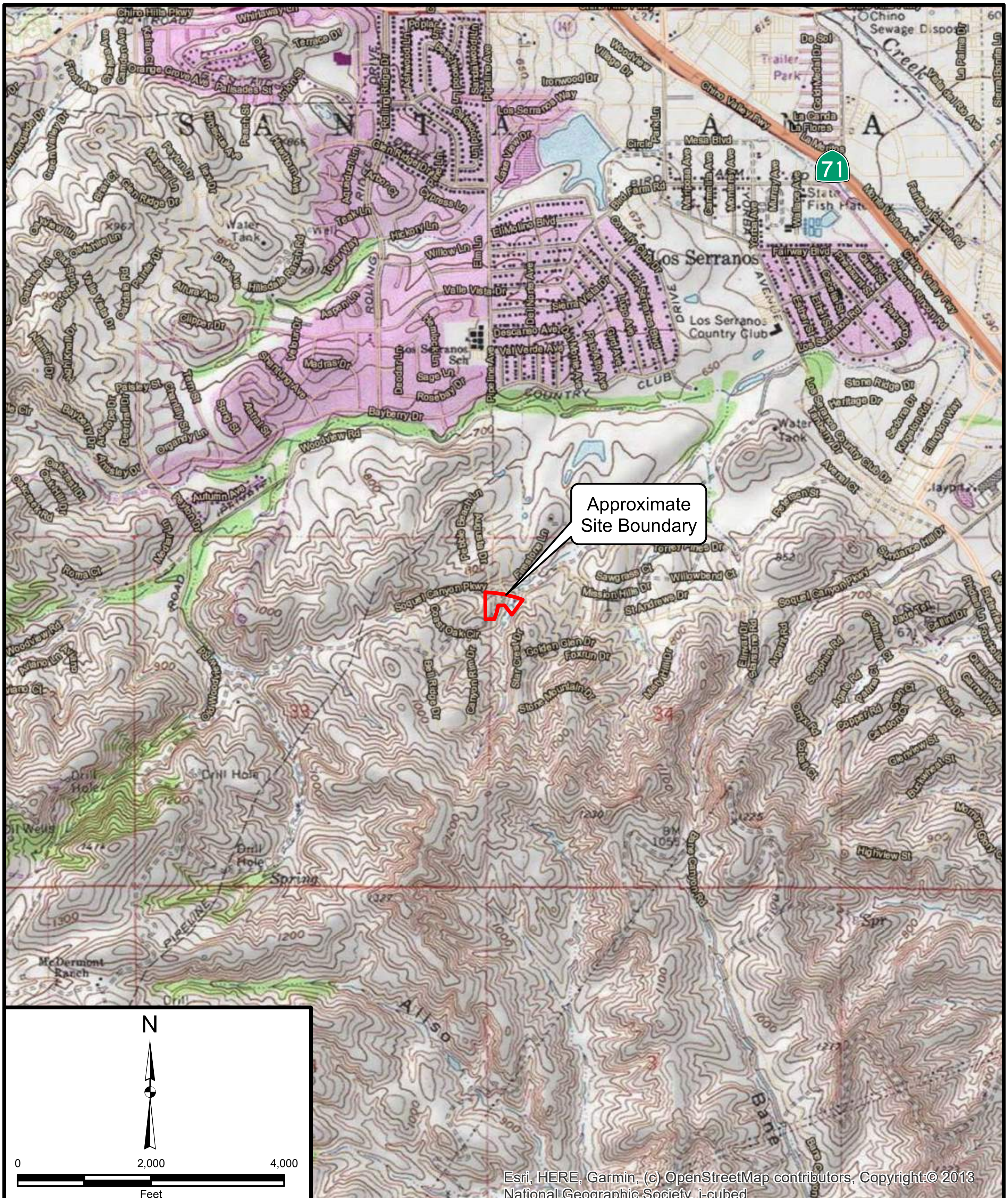


Project: 13353.001	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: June 2022

Base Map: ESRI ArcGIS Online 2021
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Proposed Fire Station No. 68
South of Soquel Canyon Parkway and
Pipeline Avenue
Chino Hills, California

FIGURE 1

Leighton

Project: 13353.001	Eng/Geol: JDH/SGO
Scale: 1 " = 60 '	Date: June 2022

Base Map: ESRI ArcGIS Online 2022

Author: Leighton Geomatics (btran)

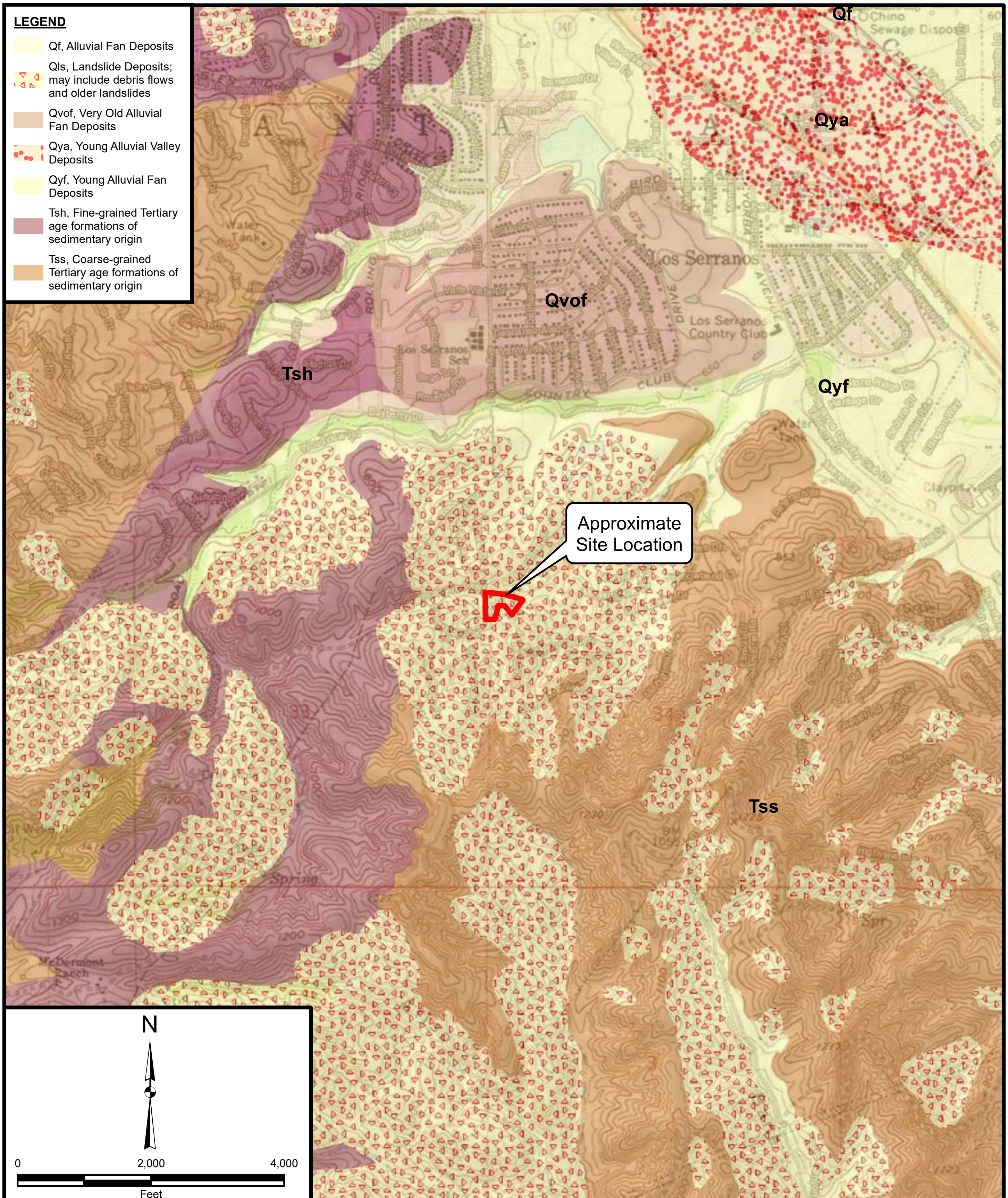
GEOTECHNICAL MAP

Proposed Fire Station No. 8
South of Soquel Canyon Parkway and Pipeline Avenue
Chino Hills, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\13353\001\Maps\13353-001_F02_ELM_2022-05-16.mxd on 5/16/2022 10:06:42 AM



Project: 13353.001	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: June 2022

Base Map: ESRI ArcGIS Online 2021
Socal Preliminary Geology, 2010

Author: Leighton Geomatics (btran)

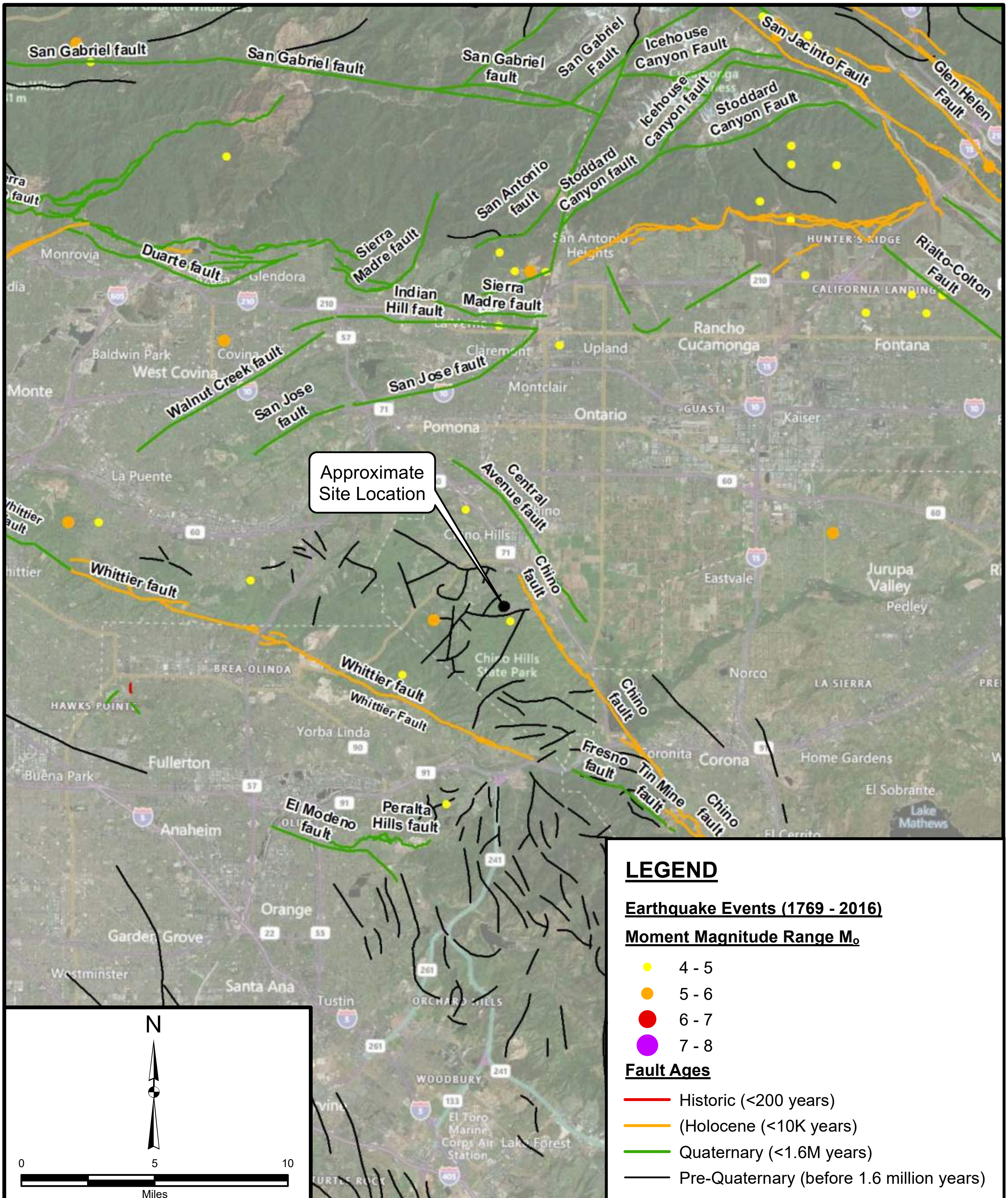
REGIONAL GEOLOGY MAP

Proposed Fire Station No. 68
South of Soquel Canyon Parkway
and Pipeline Avenue
Chino Hills, California

FIGURE 3

Leighton

Map Saved as V:\Drafting\13353\001\Maps\13353-001_F03_RGM_2021-11-24.mxd on 11/24/2021 1:52:36 PM



Project: 13353.001	Eng/Geol: JDH/SGO	REGIONAL FAULT AND HISTORICAL SEISMICITY MAP	FIGURE 4
Scale:1 " = 5 miles	Date: June 2022	Proposed Fire Station No. 68 South of Soquel Canyon Parkway and Pipeline Avenue Chino Hills, California	Leighton
Reference: ESRI ArcGIS Online 2021 Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC. Author: Leighton Geomatics (btran)			

Map Saved as V:\Drafting\13353\001\Maps\13353-001_F04_RF&HSM_2021-11-24.mxd on 11/24/2021 1:55:04 PM

Project: 13353.001	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: June 2022
Base Map: ESRI ArcGIS Online 2021	
Author: Leighton Geomatics (btran)	

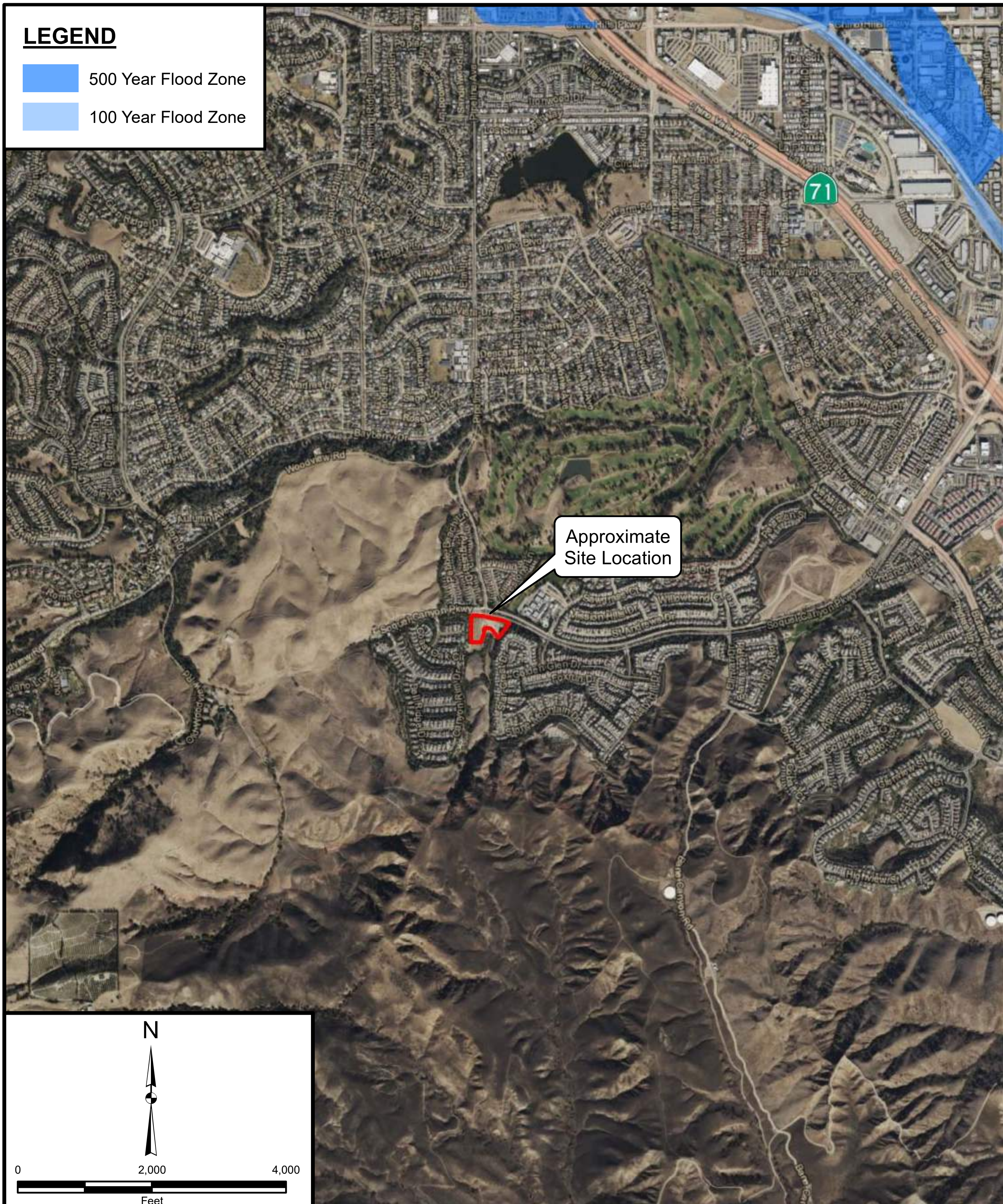
DAM INUNDATION MAP

Proposed Fire Station No. 68
South of Soquel Canyon Parkway
and Pipeline Avenue
Chino Hills, California

FIGURE 5

Leighton

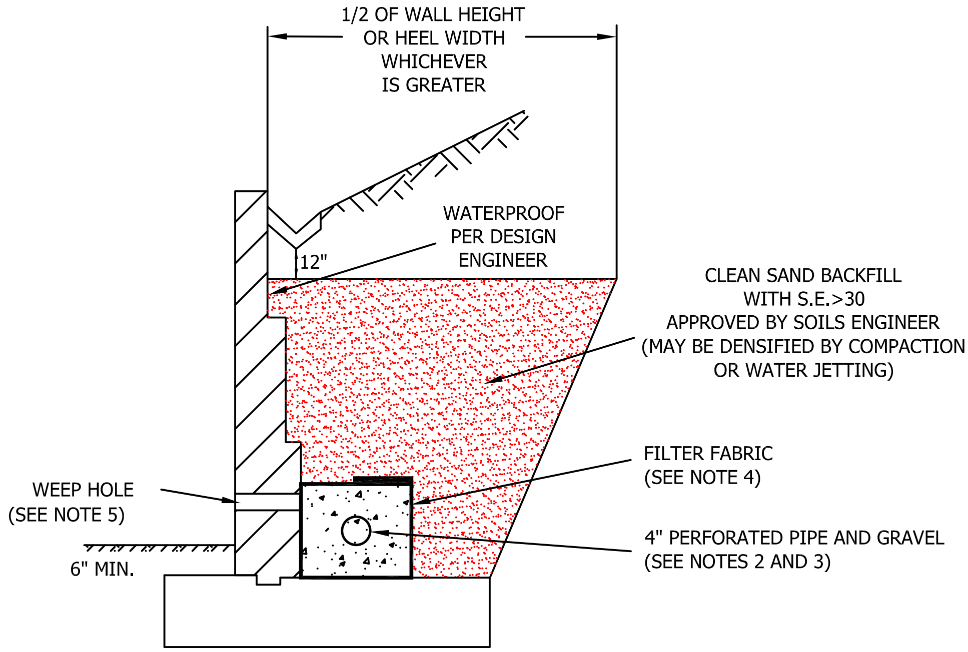
Map Saved as V:\Drafting\13353\001\Maps\13353-001_F05_DIM_2021-11-24.mxd on 11/24/2021 2:16:19 PM



Project: 13353.001	Eng/Geol: JDH/SGO	FLOOD HAZARD ZONE MAP Proposed Fire Station No. 68 South of Soquel Canyon Parkway and Pipeline Avenue Chino Hills, California	FIGURE 6
Scale:1 " = 2,000 '	Date: June 2022		Leighton
Base Map: ESRI ArcGIS Online 2021 Reference: CA DWR, FEMA Author: Leighton Geomatics (btran)			

Map Saved as V:\Drafting\13353\001\Maps\13353-001_F06_FHM_2021-11-24.mxd on 11/24/2021 2:02:47 PM

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



NOTE: AS AN ALTERNATE TO CLEAN SAND BACKFILL, CLEAN GRAVEL MAY BE UTILIZED WITH APPROVED FILTER FABRIC. A SECOND ALTERNATE IS TO UTILIZE AN AGGREGATE BASE MATERIAL COMPACTED TO 90% RELATIVE COMPACTION. A SAMPLE OF THE PROPOSED BASE MUST BE APPROVED BY THE GEOTECHNICAL CONSULTANT PRIOR TO BACKFILL FOR SUITABILITY. COMPACTION SHOULD BE ACHIEVED WITHOUT DAMAGING THE WALL.

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

Leighton

Figure 7

APPENDIX A

FIELD EXPLORATION

Our field exploration consisted of geologic reconnaissance and a subsurface exploration program consisting of six (6) borings, three (3) bucket auger borings, and one (1) infiltration test. These subsurface exploration locations are plotted on Figure 2, *Geotechnical Map*, and describe in more detail below:

Hollow Stem Auger Borings: On November 19, 2021, six borings were drilled with a truck rig, logged and sampled to depths ranging from approximately 10 feet to 47 feet. After sampling and logging, all borings were immediately backfilled, except for LB-5 where an infiltration test was performed in accordance with the guidelines of San Bernardino County. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Near surface bulk soil samples were collected from these borings. Boring logs and infiltration test results are included as part of this appendix.

Bucket Auger Borings: Between April 25, 2022 and April 26, 2022, three bucket auger borings were drilled utilizing a Lodril mounted excavator, logged and sampled to depths ranging from approximately 43.5 feet to 56.5 feet below the existing ground surface. The drilled borings were downhole logged by a Certified Engineering Geologist in accordance with the Unified Soil Classification System (ASTM D2488). Bucket auger boring logs are included as part of this appendix.

Subsurface Variations and Limitations: These attached subsurface exploration logs and related information depict subsurface conditions only at the approximate locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. Passage of time may result in altered subsurface conditions due to possible environmental changes. In addition, any stratification lines depicted on these logs represent an approximate boundary between soil types, but these transitions can be gradual.

GEOTECHNICAL BORING LOG LB-1

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	796'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Vegetation over CLAY (CL), brown, dry to slightly mosit, white plastic material present approximately 1/10-inch wide, rootlets	RV
795				R-1	13 29 49				@2.5': CLAY (CL), hard, brown to dark brown, dry to slightly moist, low to medium plasticity, manganese oxide lenses, iron oxide specs, micaceous	
790	5			R-2	15 23 33	113	16		@5': As above, very stiff	
				R-3	6 17 27			CL	@7.5': As above, very stiff	
785	10			R-4	8 13 17	105	22		@10': SANDY CLAY (CL), very stiff, variegated brown, dark brown, orangish brown, and gray, moist, fine sand, low to medium plasticity	
780	15			R-5	6 14 20			CL	@15': CLAY (CL), hard, brown to dark brown, dry to slightly moist, low to medium plasticity, manganese oxide lenses, iron oxide specs, micaceous, trace coarse gravel	
775	20			S-1	3 6 7				@20': As above, stiff, 2-inch rock stuck in sampler, poor recovery	
770	25			R-6	7 14 12				**PUENTE FORMATION (Tsh) [CLAYSTONE]** @25': CLAYSTONE, variegated brown, dark brown, orangish brown, and gray, slightly to moderately indurated, moist, fine sand, low plasticity **TOTAL DEPTH = 26.5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS**	
	30									

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

TYPE OF TESTS:

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

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GEOTECHNICAL BORING LOG LB-2

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	780'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
780	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Minor vegetation over SANDY CLAY (CL), brown, slightly moist, fine to medium sand, low to medium plasticity, micaceous	
775	5			R-1	11 26 40	111	17		@5': CLAY (CL), very stiff, variegated brown, dark brown, olive brown, and gray, slightly moist, trace fine sand, low to medium plasticity, iron oxide specs, manganese lenses, micaceous	
770	10			R-2	10 25 40	92	21		@10': SANDY CLAY (CL), very stiff, brown to dark brown, slightly moist to moist, fine sand, low to medium plasticity, iron oxide specs, micaceous, 66% fines (lab)	-200
765	15			R-3	8 20 27	106	20	CL	@15': CLAY (CL), very stiff, variegated brown, dark brown, olive brown, and gray, slightly moist, trace fine sand, low to medium plasticity, iron oxide specs, manganese lenses, micaceous	
760	20			S-1	2 4 8				**PUENTE FORMATION (Tsh)** @20': CLAYSTONE, variegated brown, dark brown, olive brown, and gray, moderately indurated, slightly moist, trace fine sand, low to medium plasticity, iron oxide specs, manganese lenses, micaceous	AL
755	25			R-4	4 11 19	97	27	CL	@25': CLAYSTONE, variegated brown, dark brown, olive brown, and gray, moderately indurated, slightly moist, trace fine sand, low to medium plasticity, iron oxide specs, manganese lenses, micaceous	
750	30									

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

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

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*** * * 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.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	780'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
750	30			S-2	10 17 24				@30': CLAYSTONE, variegated brown, gray, and orangish brown, moderately indurated, slightly moist, trace fine sand, low plasticity, iron oxide specs, manganese lenses, micaceous	
									TOTAL DEPTH = 31.5 FEET **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS**	
745	35									
740	40									
735	45									
730	50									
725	55									
720	60									

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

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

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GEOTECHNICAL BORING LOG LB-3

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	776'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
775	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Vegetation over CLAY (CL), dry, tan, few pieces of white plastic material present	
				R-1	9 21 34	114	14		@2.5': CLAY (CL), hard, brown to dark brown, dry to slightly moist, low to medium plasticity, manganese oxide lenses, iron oxide specs, micaceous, 79% fines (lab)	-200, AL
770	5			R-2	17 50/6"				@5': As above, poor recovery	
				R-3	16 50/6"			CL	@7.5': As above, poor recovery	
765	10			R-4	6 13 14	97	18		@10': Very stiff	CN
760	15			S-1	5 22 21			CH	@15': CLAY (CH), hard, variegated brown, dark brown, gray, and orangish brown, fine sand, medium to high plasticity, iron oxide staining, micaceous	AL
755	20			R-5	14 50/4"				**PUENTE FORMATION (Tsh)** @20': SANDY CLAYSTONE, dark gray, well indurated, fine sand, low plasticity, iron oxide staining	
750	25			S-2	7 19 50/5.5"				@25': SANDY CLAYSTONE, dark gray, well indurated, fine sand, low plasticity, iron oxide staining	
	30								**TOTAL DEPTH = 26.5 FEET NO GROUNDWATER ENCOUNTERED DURING DRILLING BACKFILLED WITH SOIL CUTTINGS**	

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

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

Leighton

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GEOTECHNICAL BORING LOG LB-4

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	786'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
785	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Vegetation over CLAY (CL), dark olive brown, moist, low plasticity	MD, EI, CR
780	5			R-1	11 23 27	109	20		@5': SANDY CLAY (CL), very stiff, variegated brown to dark brown, olive , gray, and orangish brown, slightly moist to moist, trace fine sand, trace coarse gravel, iron oxide specs, manganese oxide specs, low to medium plasticity, 72% fines (lab)	-200, AL
775	10			R-2	8 18 25	112	16		@10': As above, very stiff	
770	15			S-1	6 8 5			CL	@15': As above, stiff @17': Auger grinding on gravels and cobbles	
765	20			R-3	7 18 22	100	22		@20': As above, very stiff	
760	25			S-2	4 7 8			CL	**PUENTE FORMATION (Tsh)** @25': SANDY CLAYSTONE, variegated browm, dark brown, olive brown, gray, and orangish brown, moderately indurated, fine sand, iron oxide specs, manganese oxide lenses, low to medium plasticity, 76% fines (lab)	-200
	30									

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

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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	786'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
755	30			R-4	4 10 15	100	27	CL	@30': CLAYSTONE, variegated brown to dark brown, olive , gray, and orangish brown, moderately indurated, slightly moist to moist, trace fine sand, trace coarse gravel, iron oxide specs, manganese oxide specs, low to medium plasticity	
750	35			S-3	3 7 8				@35': CLAYSTONE, variegated brown to dark brown, olive , gray, and orangish brown, moderately indurated, slightly moist to moist, trace fine sand, trace coarse gravel, iron oxide specs, manganese oxide specs, low to medium plasticity	
745	40			R-5	50/5.5"				@40': CLAYSTONE, dark gray, well indurated, fresh, fine grained	
740	45			S-4	22 50/1"				@45': CLAYSTONE, dark gray, well indurated, fresh, fine grained	
									Refusal	
735	50								**TOTAL DEPTH = 47 FEET** **GROUNDWATER ENCOUNTERED AT 41.7 FEET DURING DRILLING** **GROUNDWATER ENCOUNTERED AT 38.2 FEET AFTER DRILLING** **BACKFILLED WITH SOIL CUTTINGS**	
730	55									
	60									

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

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

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

Leighton

GEOTECHNICAL BORING LOG LB-5/IT-1

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	788'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Grass over CLAY (CL), dark brown, moist, low to medium plasticity, rootlets	
785										
	5			R-1 B-2	3 6 9				@5': CLAY (CL), stiff, variegated brown, dark brown, gray, orangish brown, moist, low to medium plasticity, micaceous, 1% gravel, 30% sand, 69% fines (lab)	SA
780										
				R-2	3 4 6			CL	@8.5': Same as above, dark brown,	
	10								**TOTAL DEPTH = 10 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **CONVERTED TO WELL PERMEAMETER TEST BORING ON 11/19/2021**	
775										
	15									
770										
	20									
765										
	25									
760										
	30									

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

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

Leighton

GEOTECHNICAL BORING LOG LB-6

Project No.	13353.001	**Date Drilled**	11-19-21
Project	Proposed Fire Station No. 8	**Logged By**	JP
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	769'
Location	See Figure 2 - Geotechnical Map	**Sampled By**	JP

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SC	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: Minor vegetation over CLAYEY SAND (SC), tan, dry, low plasticity, rootlets	
				R-1	23 24 27	89	8		@2.5': SANDY CLAY (SC), dense, tan, dry, fine sand, fine gravel, trace rootlets, low plasticity, 48% fines (lab)	-200, AL
765	5			S-1	6 13 11			CL	@5': CLAY (CL), very stiff, slightly moist, vareigated dark brown, olive brown, orangish brown, and gray, fine sand, mangenese oxide specs, iron oxide specs, low to medium plasticity	
				R-2	10 16 25	100	23	CL	@7.5': As above	
760	10			R-3	6 17 22				@10': As above	
755	15			S-2	3 2 2			CL	@15': As above, stiff	
750	20			R-4	5 14 22				**PUENTE FORMATION (Tsh)** @20': CLAYSTONE, slightly moist, vareigated dark brown, olive brown, orangish brown, and gray, moderately to well indurated, fine sand, mangenese oxide specs, iron oxide specs, low to medium plasticity **TOTAL DEPTH = 21.5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS**	
745	25									
740	30									

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

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

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GEOTECHNICAL BORING LOG BA-1

Project No.	13353.002	**Date Drilled**	4-25-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	777'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: dried vegetation, SANDY CLAY (CL)	
775										
	5			R-1	10	98	25	CL	@5':SANDY CLAY (CL), stiff, slightly moist to moist, brown, fine sand, trace fine gravel, trace oxidation, low to medium plasticity, firm (based on hammer blows)	
770										
	10			R-2	12	91	31	CL	@10':SANDY CLAY (CL), stiff, slightly moist to moist, brown, fine sand, trace fine gravel, trace oxidation, low to medium plasticity, firm (based on hammer blows)	
765										
	15			R-3	14	117	11	CL	@15':SANDY CLAY (CL), stiff, slightly moist to moist, brown, fine sand, trace fine gravel, trace oxidation, low to medium plasticity, firm (based on hammer blows)	
760										
	20			R-4	15	97	26	CL	@20':SANDY CLAY (CL), stiff, slightly moist to moist, brown, fine sand, trace fine gravel, trace oxidation, with claystone fragments, low to medium plasticity, firm (based on hammer blows)	
755										
									PUENTE FORMATION (Tsh) 24' to T.D.: CLAYSTONE, yellowish brown, no visible fractures, clean/sharp contact with fill above	
	25		B:N40E 125E	R-5	17					
750										
	30									

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

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

Leighton

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GEOTECHNICAL BORING LOG BA-1

Project No.	13353.002	**Date Drilled**	4-25-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	777'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30								@30': minor seepage, bedding appears to gradually flatten	
745			B:N45E 75E							
									@34': hard, gray to dark grayish brown, unoxidized	
	35			R-6	23	94	21			
									@36': moderate seepage from small fractures	
740										
	40		B:N78E 75E						@39.5': moderate seepage	
735										
	45			R-7	10				@45': hard, gray to dark grayish brown, non-fractured, unoxidized	
730										
	50									
725										
	55			R-8	28	95	20		@55': standing water	
720									**TOTAL DEPTH = 56.5 FEET** **GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	60									

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

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

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

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GEOTECHNICAL BORING LOG BA-2

Project No.	13353.002	**Date Drilled**	4-25-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	787'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: dried vegetation, over CLAY (CL)	
785									0-1': SANDY CLAY (CL), dry and desiccated	
									@3': SANDY CLAY (CL), moist, firm (based on hammer blows)	
	5			R-1	14	101	23	CL	@5': SANDY CLAY (CL), med stiff, brown to dark brown, slightly moist to moist, trace fine sand, trace oxidation, FeO2 lenses, low plasticity, firm (based on hammer blows)	
780										
	10			R-2	15	102	23	CL	@10': SANDY CLAY (CL), med stiff, brown to dark brown, slightly moist to moist, trace fine sand, trace FeO2 lenses, low to medium plasticity, firm (based on hammer blows)	
775										
	15			R-3	23	102	22	CL	@15': SANDY CLAY (CL), med stiff, variegated, brown to dark brown, dry to slightly moist, trace fine sand, trace oxidation, FeO2 lenses, low to medium plasticity, firm (based on hammer blows)	
770										
	20			R-4	16	96	26		@20': SANDY CLAY (CL), med stiff, variegated, brown to dark brown, dry to slightly moist, trace fine sand, trace oxidation, FeO2 lenses, low to medium plasticity, firm (based on hammer blows)	
765										
	25			R-5	19				**PUENTE FORMATION (Tsh)]** 25' to T.D.: CLAYSTONE, yellowish brown, clean/sharp contact, with fill above, no organics, slightly to moderately indurated	
760			B:N60E 45E							
	30									

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

TYPE OF TESTS:

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

Leighton

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GEOTECHNICAL BORING LOG BA-2

Project No.	13353.002	**Date Drilled**	4-25-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	787'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30									
755										
			B:N45E 70E						@34': gray to dark grayish brown, hard, non-fractured, unoxidized, moderately indurated	
	35			R-6	27				@35': gray to dark gray, fresh, unoxidized, hard, fine grained, non-fractured	
750										
									Standing Water	
	40									
745										
	45								**TOTAL DEPTH = 43.5 FEET** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
740										
	50									
735										
	55									
730										
	60									

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

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

Leighton

GEOTECHNICAL BORING LOG BA-3

Project No.	13353.002	**Date Drilled**	4-26-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	781'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**UNDOCUMENTED ARTIFICIAL FILL (Afu)** @Surface: dried vegetation, CLAY (CL)	
780									0-1.5': SANDY CLAY, dry and desiccated	
	5			R-1	13	104	20	CL	@5': SANDY CLAY (CL), medium stiff, dry to slightly moist, brown to gray, oxidized, FeO2 specs, carbonate specs, low plasticity, rootlets, firm (based on hammer blows)	
775										
	10			R-2	16	104	22	CL	@10': SANDY CLAY (CL), medium stiff, dry to slightly moist, brown to gray, trace oxidation, FeO2 specs, carbonate specs, low plasticity, rootlets, firm (based on hammer blows)	
770										
	15			R-3	16	100	25	CL	@15': SANDY CLAY (CL), medium stiff, dry to slightly moist, brown to gray, trace oxidation, FeO2 specs, carbonate specs, low plasticity, firm (based on hammer blows)	
765										
	20			R-4	17				**PUENTE FORMATION (Tsh)** 20' to T.D.: CLAYSTONE, yellowish brown, clean/sharp contact with fill above, no organics, slightly to moderate indurated @21': yellowish brown, clean/sharp contact, slightly to moderatly indurated, no organics @22'-29': structure is not apperant, claystone chips from hammer blows in fragments with no visible bedding	
760			B:N44E 125E							
	25		B:N50E 30E							
755										
			B:N73E 26E						@29': tectonically slickensided bedding	
	30									

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

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

Leighton

GEOTECHNICAL BORING LOG BA-3

Project No.	13353.002	**Date Drilled**	4-26-22
Project	Proposed Fire Station No. 8	**Logged By**	JP/SGO
Drilling Co.	Roy Brothers Drilling, Inc	**Hole Diameter**	24"
Drilling Method	Bucket Auger - Down Hole	**Ground Elevation**	781'
Location	See Figure 2- Geotechnical Map	**Sampled By**	JP

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-5	20	99	23		@30': hard, variegated, yellowish brown to brown to gray, no fractures, micaceous	
750										
									@33': minor seepage	
			B:N83E 165E						@34': color change (gray to dark gray), no clay seam	
	35								@35': gray to dark gray, hard, non-fractured, unoxidized, moderatly indurated	
745										
			B:N65E 35E							
	40			R-6	24				@40': gray to dark gray, hard, non-fractured, fine grained, micaceous	
740										
	45									
735										
	50			R-7	26	81	21		@50': gray to dark gray, hard, non-fractured, fine grained, micaceous Standing Water	
730										
	55									
725									**TOTAL DEPTH = 56 FEET** **GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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Results of Well Permeameter, from USBR 7300-89 Method

Leighton

Project: 13353.001

Exploration #/Location:	*LB-5*	Initial estimated Depth to Water Surface (in.):	71
Depth Boring drilled to (ft):	10	Average depth of water in well, "h" (in.):	42
Tested by:	JAT	approx. h/r:	10.6
USCS Soil Type in test zone:	CL	Tu (Fig. 8) (ft):	194.1
Weather (start to finish):	Sunny	Tu>3h?:	yes, OK
Water Source/pH:	H20		
Measured boring diameter:	8 in. 4 in. Well Radius	Cross-sectional area for vol calcs (in.^2):	20.1
Approx Depth to GW BGS:	200 ft (GW or aquatarde)		
Well Prep:	Set well at 10'BGS #3 sand around 3/4" and 4" diameter pipe up to 2.3'bgs	Use of Barrels:	Yes
		Use of Flow Meter:	No
		Use of DH Valve:	Yes
		Test Type:	Constant Head

	ft	in.	Total (in.)	
Depth to Bot of well measured from top of pilot tube	9.46 ft		114	
Pilot Tube stickup (+ is above ground)	0. ft		0	
Depth to top of sand outside of casing from top of pilot tube	2.3 ft			
Depth to top of DH valve/float assembly from top of pilot tube	5.52 ft		66.2	66.24 Depth below GS (in.)
Float Assembly ID		DHVB		
Float assembly Extension length (in.)		34		

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

Flow Meter:

Meter ID	
Meter Color	
Meter Units	
DL ID	

Field Data — **Calculations** hide hide

Date	Time	Water Level in Supply Barrel (in.)	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)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min.)	Bot of Well (in)	pilot tube stickup (in)	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)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 11/22/2021	Start time: 8:42		0																				
11/22/21	8:45	28.875			5.87		67			3	113.52	0	70.4	43.1									
11/22/21	9:00	28.875			5.9				15	18	113.52	0	70.8	42.7	-0.36	43	0	7	7	0	29	1.0	0.03
11/22/21	9:15	28.75			5.94				15	33	113.52	0	71.3	42.2	-0.48	42	50	10	59	4	237	1.0	0.21
11/22/21	9:30	28.75			5.96				15	48	113.52	0	71.5	42.0	-0.24	42	0	5	5	0	19	1.0	0.02
11/22/21	9:45	28.75			5.95				15	63	113.52	0	71.4	42.1	0.12	42	0	-2	-2	0	-10	1.0	-0.01
11/22/21	10:00	28.75			5.94		68		15	78	113.52	0	71.3	42.2	0.12	42	0	-2	-2	0	-10	1.0	-0.01
11/22/21	10:30	28.75			5.97				30	108	113.52	0	71.6	41.9	-0.36	42	0	7	7	0	14	1.0	0.01
11/22/21	11:00	28.625			5.96				30	138	113.52	0	71.5	42.0	0.12	42	50	-2	47	2	95	1.0	0.08
								Switched to FH															
																						Minimum Rate:	-0.01
																						Raw Rate for design, prior to application of adjustment factors	**0.01**

Results of Falling Head Infiltration Test

Leighton

Project: 13353.001

Exploration #/Location:	*LB-5*	Initial estimated Depth to Water Surface (in.):	80
Depth Boring drilled to (ft):	10	Average depth of water in well, "h" (in.):	34
Tested by:	JAT	approx. h/r:	8.4
USCS Soil Type in test zone:	CL	Tu (Fig. 8) (ft):	193.3
Weather (start to finish):	Sunny	Tu>3h?:	yes, OK
Water Source/pH:	H20		
Measured boring diameter:	8 in. 4 in. Well Radius	Cross-sectional area for vol calcs (in.^2):	20.1
Approx Depth to GW BGS:	200 ft (GW or aquatarde)		

Well Prep: Set well at 10'BGS #3 sand around 3/4" and 4" diameter pipe up to 2.3'bgs

	ft	in.	Total (in.)	
Depth to Bot of well measured from top of pilot tube	9.46 ft		114	
Pilot Tube stickup (+ is above ground)	0. ft		0	
Depth to top of sand outside of casing from top of pilot tube	2.3 ft			
Depth to top of DH valve/float assembly from top of pilot tube	5.52 ft		66.2	66.24 Depth below GS (in.)
Float Assembly ID				
Float assembly Extension length (in.)				

Use of Barrels:	No
Use of Flow Meter:	No
Use of DH Valve:	No
Test Type:	Falling Head

Diameter of barrels (in.):

No. of Supply barrels:

Total Area of barrels (in.^2): 0

Flow Meter:

Meter ID

Meter Color

Meter Units

DL ID

Field Data **Calculations** hide hide

Date	Time	Water Level in Supply Barrel (in.)	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)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min.)	Bot of Well (in)	pilot tube stickup (in)	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)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 11/22/2021	Start time: 11:00		0																				
11/22/21	11:02				6.63		69			2	113.52	0	79.6	34.0									
11/22/21	11:17				6.635				15	17	113.52	0	79.6	33.9	-0.06	34	0	1	1	0	5	1.0	0.01
11/22/21	11:32				6.64				15	32	113.52	0	79.7	33.8	-0.06	34	0	1	1	0	5	1.0	0.01
11/22/21	11:47				6.65				15	47	113.52	0	79.8	33.7	-0.12	34	0	2	2	0	10	1.0	0.01
11/22/21	12:02				6.66				15	62	113.52	0	79.9	33.6	-0.12	34	0	2	2	0	10	1.0	0.01
11/22/21	12:32				6.68		70		30	92	113.52	0	80.2	33.4	-0.24	33	0	5	5	0	10	0.9	0.01
11/22/21	13:02				6.71				30	122	113.52	0	80.5	33.0	-0.36	33	0	7	7	0	14	0.9	0.02

Minimum Rate: 0.01

Raw Rate for design, prior to application of adjustment factors **0.01**

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying proposed improvements, and to aid in verifying soil classification.

In-Situ Moisture and Density: As-sampled soil moisture content was measured (ASTM D 2216) on selected samples recovered from our borings. In addition, in place dry density was measured (ASTM D 2937) on selected relatively undisturbed soil samples. Results of these tests are shown on our logs at the appropriate sample depths in Appendix A.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D 1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of this grain size analysis, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our test pit logs.

Particle Size (Sieve) Analysis: Particle size analysis of bulk soil samples by passing sieves was evaluated using the ASTM D 6913 Standard Test Method. Results of these analysis are presented on the *Particle-Size Distribution ASTM D 6913* sheets in this appendix.

Modified Proctor Compaction Curve: A laboratory modified Proctor compaction curve (ASTM D1557) was established for bulk soil-sample to evaluate the modified Proctor laboratory maximum dry density and optimum moisture content. Results of this test are presented on the following *Modified Proctor Compaction Test* sheet in this appendix.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk soil sample collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Chino Valley Fire Station No. 8	Tested By:	L. Manka	Date:	11/30/21
Project No.:	13353.001	Checked By:	A. Santos	Date:	12/07/21
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark olive lean clay (CL)				

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

Mold Volume (ft³) 0.03320 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3657	3789	3833	3818		
Weight of Mold (g)	1862	1862	1862	1862		
Net Weight of Soil (g)	1795	1927	1971	1956		
Wet Weight of Soil + Cont. (g)	522.7	471.0	475.6	470.7		
Dry Weight of Soil + Cont. (g)	486.8	431.8	427.5	415.3		
Weight of Container (g)	87.8	88.7	87.9	87.9		
Moisture Content (%)	9.00	11.43	14.16	16.92		
Wet Density (pcf)	119.2	128.0	130.9	129.9		
Dry Density (pcf)	109.4	114.8	114.6	111.1		

Maximum Dry Density (pcf) **115.5** **Optimum Moisture Content (%)** **12.5**

PROCEDURE USED

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

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

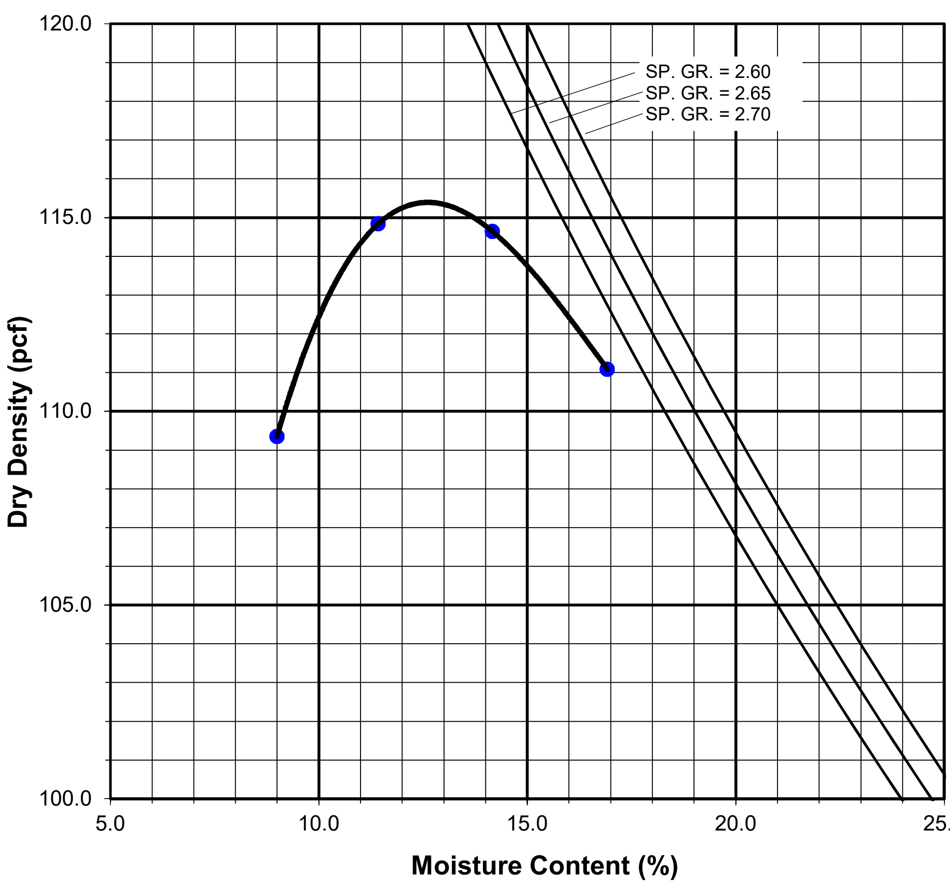
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-2	LB-3	LB-4	LB-4	LB-6			
Sample No.	R-2	R-1	R-1	S-2	R-1			
Depth (ft.)	10.0	2.5	5.0	25	2.5			
Sample Type	Ring	Ring	Ring	SPT	Ring			
Soil Identification	Brown sandy lean clay s(CL)	Brown lean clay with sand (CL)s	Brown lean clay with sand (CL)s	Brown lean clay with sand (CL)s	Dark grayish 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)	439.60	708.10	688.30	598.06	508.00			
Weight of Container (g)	106.60	107.40	106.50	109.60	109.30			
Weight of Dry Sample (g)	333.00	600.70	581.80	488.46	398.70			
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A			
Dry Weight of Sample + Cont. (g)	220.30	233.30	269.10	229.00	316.70			
Weight of Container (g)	106.60	107.40	106.50	109.60	109.30			
Dry Weight of Sample (g)	113.70	125.90	162.60	119.40	207.40			
% Passing No. 200 Sieve	**65.9**	**79.0**	**72.1**	**75.6**	**48.0**			
% Retained No. 200 Sieve	34.1	21.0	27.9	24.4	52.0			

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Chino Valley Fire Station No. 8
Project No.: 13353.001
Tested By: ACS/JD Date: 11/24/21

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

ASTM D 6913

Project Name:	Chino Valley Fire Station No. 8	Tested By:	J. Domingo	Date:	11/24/21
Project No.:	13353.001	Checked By:	A. Santos	Date:	12/06/21
Boring No.:	LB-5	Depth (feet):	5-10		
Sample No.:	B-2				
Soil Identification:	Dark gray sandy lean clay s(CL)				

		Moisture Content of Total Air - Dry Soil	
Container No.:	929	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	765.1	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	107.5	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	657.6	Moisture Content (%)	0.0

After Wet Sieve	Container No.	929
	Wt. of Dry Soil + Container (g)	329.2
	Wt. of Container (g)	107.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	221.7

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5	0.0	100.0
#4	4.75	5.7	99.1
#8	2.36	13.2	98.0
#16	1.18	24.1	96.3
#30	0.600	44.0	93.3
#50	0.300	76.3	88.4
#100	0.150	119.1	81.9
#200	0.075	206.7	68.6
PAN			

GRAVEL: **1 %**

SAND: **30 %**

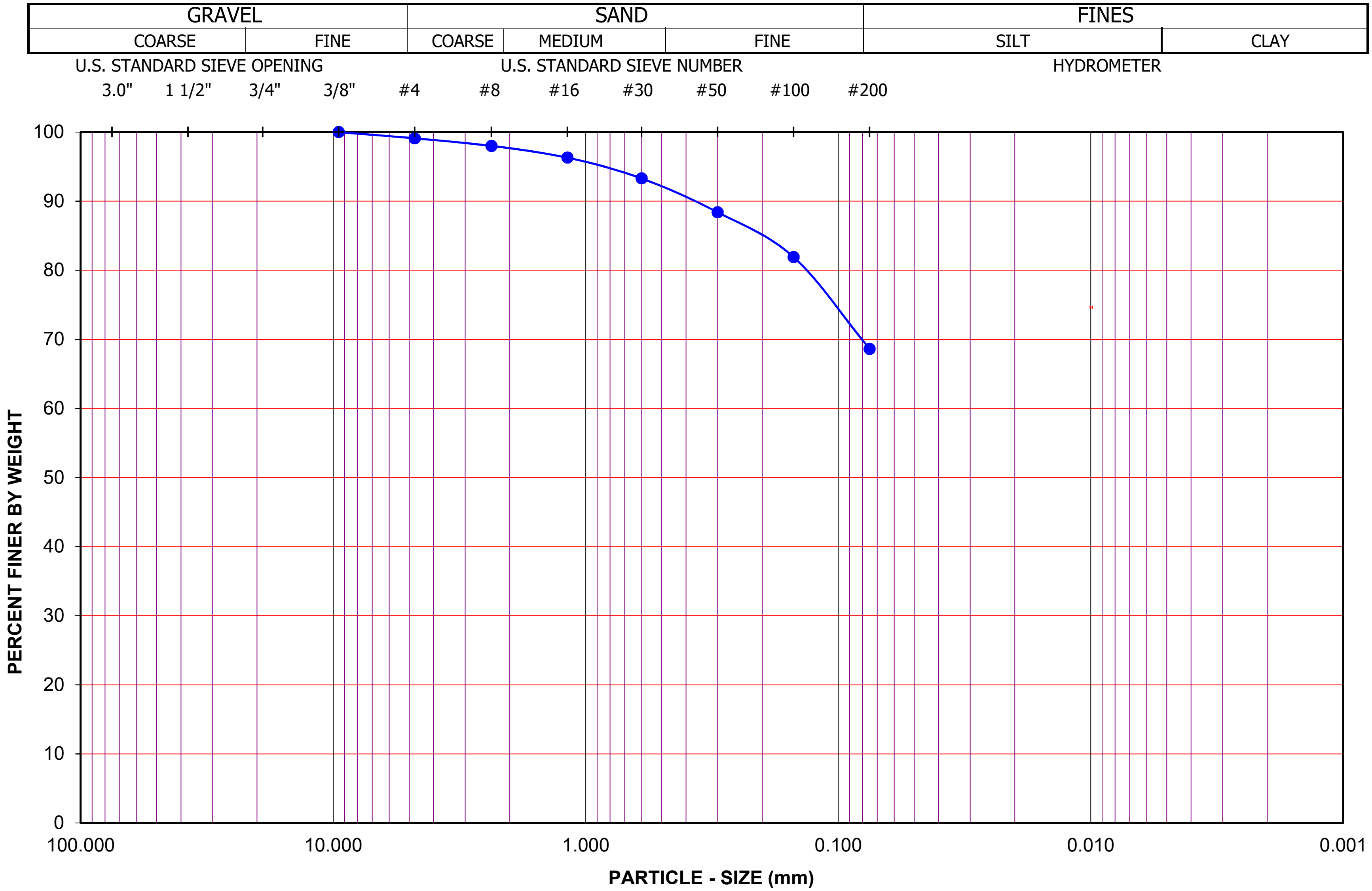
FINES: **69 %**

GROUP SYMBOL: **s(CL)**

$Cu = D60/D10 =$ ______

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

Remarks:



Project Name: Chino Valley Fire Station No. 8

Project No.: 13353.001

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-5 Sample No.: B-2

Depth (feet): 5-10 Soil Type : s(CL)

Soil Identification: Dark gray sandy lean clay s(CL)

GR:SA:FI : (%) 1 : 30 : 69

Dec-21

Leighton

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Chino Valley Fire Station No. 8	Tested By:	ACS/OHF	Date:	11/29/21
Project No.:	13353.001	Checked By:	A. Santos	Date:	12/07/21
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark olive lean clay (CL)				

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	1.0830
Wt. Comp. Soil + Mold (g)	573.10	437.20
Wt. of Mold (g)	184.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	787.90	621.60
Dry Wt. of Soil + Cont. (g)	713.10	536.16
Wt. of Container (g)	0.00	184.40
Moisture Content (%)	10.49	24.29
Wet Density (pcf)	117.2	121.8
Dry Density (pcf)	106.1	98.0
Void Ratio	0.589	0.721
Total Porosity	0.371	0.419
Pore Volume (cc)	76.7	93.9
Degree of Saturation (%) [S_{meas}]	**48.1**	91.0

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.)
11/29/21	10:00	1.0	0	0.6020
11/29/21	10:10	1.0	10	0.6020
Add Distilled Water to the Specimen				
11/29/21	22:15	1.0	725	0.6120
11/30/21	7:00	1.0	1250	0.6850
11/30/21	9:00	1.0	1370	0.6850

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**83**

ATTERBERG LIMITS

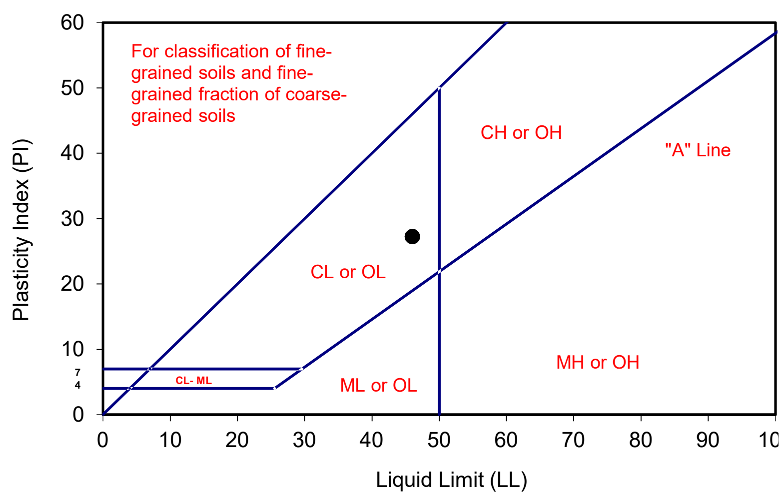
ASTM D 4318

Leighton

Project Name:	Chino Valley Fire Station No. 8	Tested By:	Y. Nguyen	Date:	11/29/21
Project No. :	13353.001	Input By:	G. Bathala	Date:	12/03/21
Boring No.:	LB-2	Checked By:	A. Santos		
Sample No.:	S-1	Depth (ft.)	20.0		
Soil Identification:	Brown lean clay (CL)				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2		
Number of Blows [N]			26	25	Trial 1 =	46
Wet Wt. of Soil + Cont. (g)	9.68	9.65	20.29	21.26	Trial 2 =	46
Dry Wt. of Soil + Cont. (g)	8.32	8.29	14.22	14.85	Ave. LL =	**46**
Wt. of Container (g)	1.06	1.04	1.10	1.06	(see equation below)	
Moisture Content (%) [Wn]	18.73	18.76	46.27	46.48		

Liquid Limit	**46**
Plastic Limit	**19**
Plasticity Index	**27**
Classification	**CL**



PI at "A" - Line = 0.73(LL-20) | 18.98

One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$

(Wn = water content, N = number of blows)

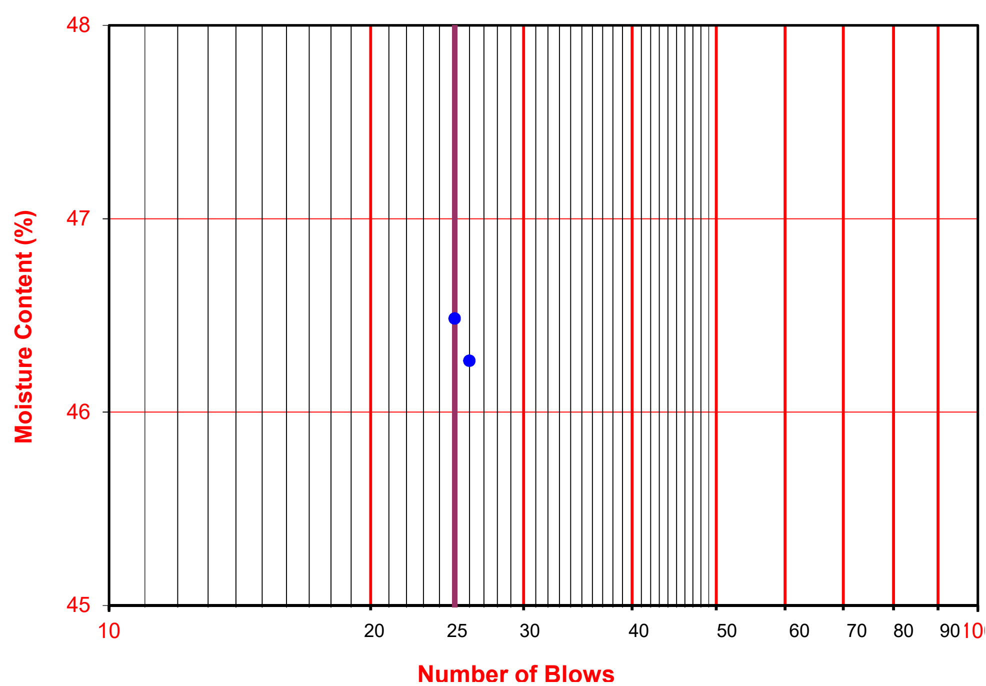
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
One-point

☐ Procedure A
Multipoint Test

☒ Procedure B
One-point Test



ATTERBERG LIMITS
ASTM D 4318

Project Name:	Chino Valley Fire Station No.8	Tested By:	Y. Nguyen	Date:	11/30/21
Project No. :	13353.001	Input By:	A. Santos	Date:	12/06/21
Boring No.:	LB-3	Checked By:	A. Santos		
Sample No.:	R-1	Depth (ft.)	2.5		
Soil Identification:	Brown lean clay with sand (CL)s				

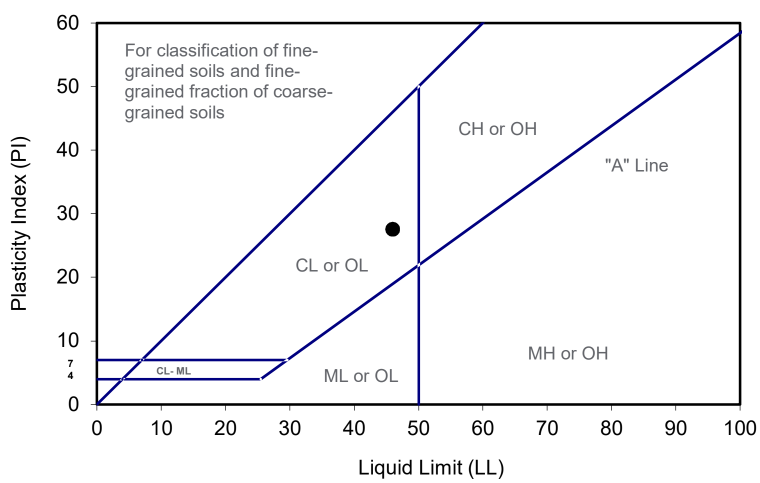
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	23	16	
Wet Wt. of Soil + Cont. (g)	9.77	9.62	20.98	22.67	20.78	
Dry Wt. of Soil + Cont. (g)	8.40	8.30	14.79	15.80	14.37	
Wt. of Container (g)	1.05	1.09	1.06	1.01	1.00	
Moisture Content (%) [Wn]	18.64	18.31	45.08	46.45	47.94	

Liquid Limit	**46**
Plastic Limit	**18**
Plasticity Index	**28**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 18.98

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



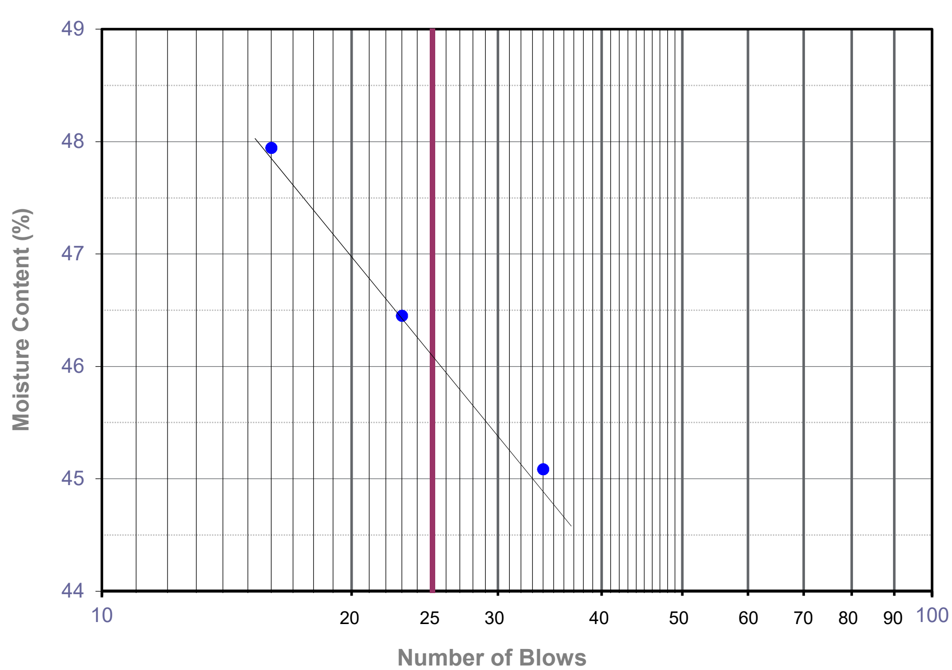
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

X Dry Preparation
Multipoint - Dry

X Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	Chino Valley Fire Station No. 8	Tested By:	Y. Nguyen	Date:	11/29/21
Project No. :	13353.001	Input By:	G. Bathala	Date:	12/03/21
Boring No.:	LB-3	Checked By:	A. Santos		
Sample No.:	S-1	Depth (ft.)	15.0		
Soil Identification:	Brown fat clay (CH)				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2		
Number of Blows [N]			26	25	Trial 1 =	53
Wet Wt. of Soil + Cont. (g)	9.70	9.85	20.25	19.32	Trial 2 =	53
Dry Wt. of Soil + Cont. (g)	7.98	8.12	13.63	13.01	Ave. LL =	**53**
Wt. of Container (g)	1.07	1.11	1.01	1.07	(see equation below)	
Moisture Content (%) [Wn]	24.89	24.68	52.46	52.85		

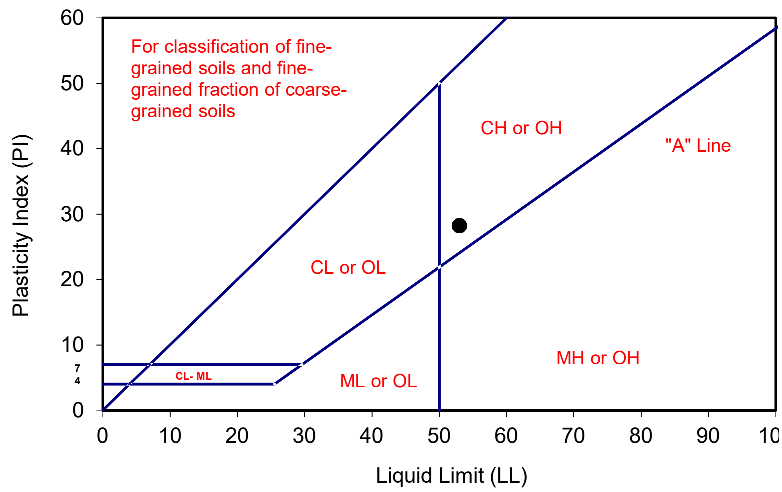
Liquid Limit	**53**
Plastic Limit	**25**
Plasticity Index	**28**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 24.09

One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$

(Wn = water content, N = number of blows)



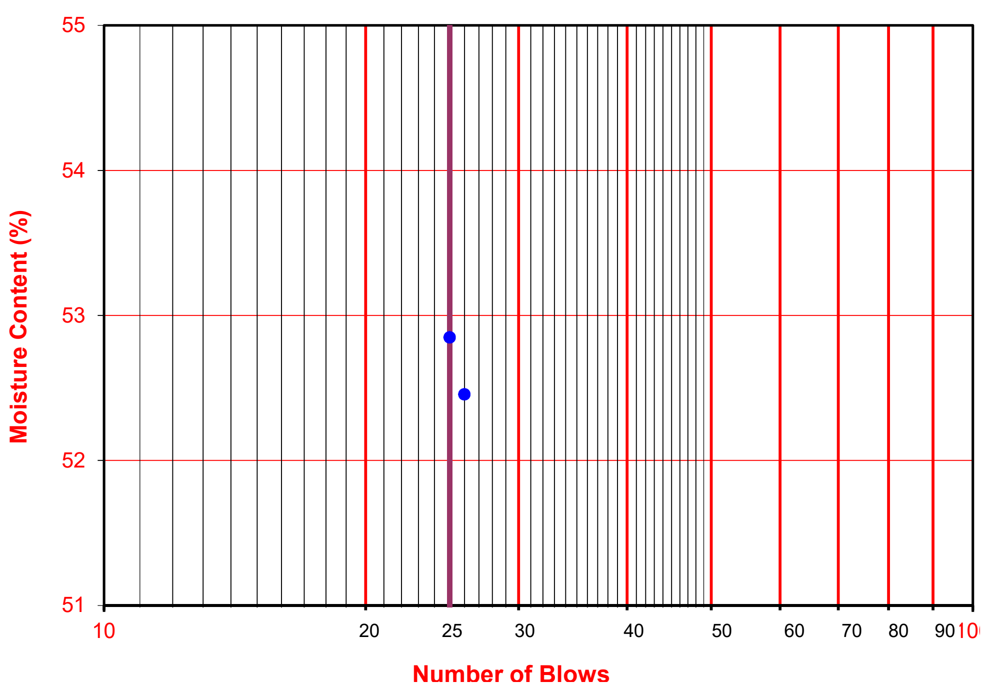
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
One-point

☐ Procedure A
Multipoint Test

☒ Procedure B
One-point Test



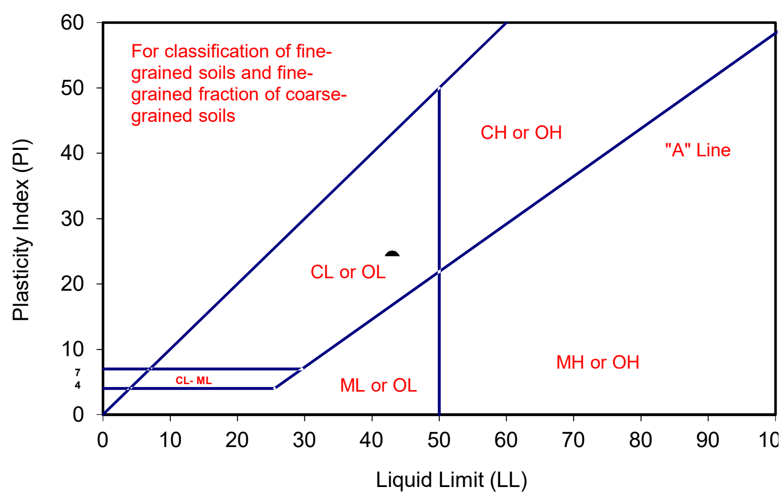
ATTERBERG LIMITS

ASTM D 4318

Project Name:	Chino Valley Fire Station No. 8	Tested By:	Y. Nguyen	Date:	11/30/21
Project No. :	13353.001	Input By:	G. Bathala	Date:	12/07/21
Boring No.:	LB-4	Checked By:	A. Santos		
Sample No.:	R-1	Depth (ft.)	5.0		
Soil Identification:	Brown lean clay with sand (CL)s				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2		
Number of Blows [N]			24	26	Trial 1 =	43
Wet Wt. of Soil + Cont. (g)	9.57	9.48	21.97	21.62	Trial 2 =	43
Dry Wt. of Soil + Cont. (g)	8.22	8.12	15.67	15.45	Ave. LL =	**43**
Wt. of Container (g)	1.10	1.02	1.02	1.01	(see equation below)	
Moisture Content (%) [Wn]	18.96	19.15	43.00	42.73		

Liquid Limit	**43**
Plastic Limit	**19**
Plasticity Index	**24**
Classification	**CL**



PI at "A" - Line = 0.73(LL-20) | 16.79

One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$

(Wn = water content, N = number of blows)

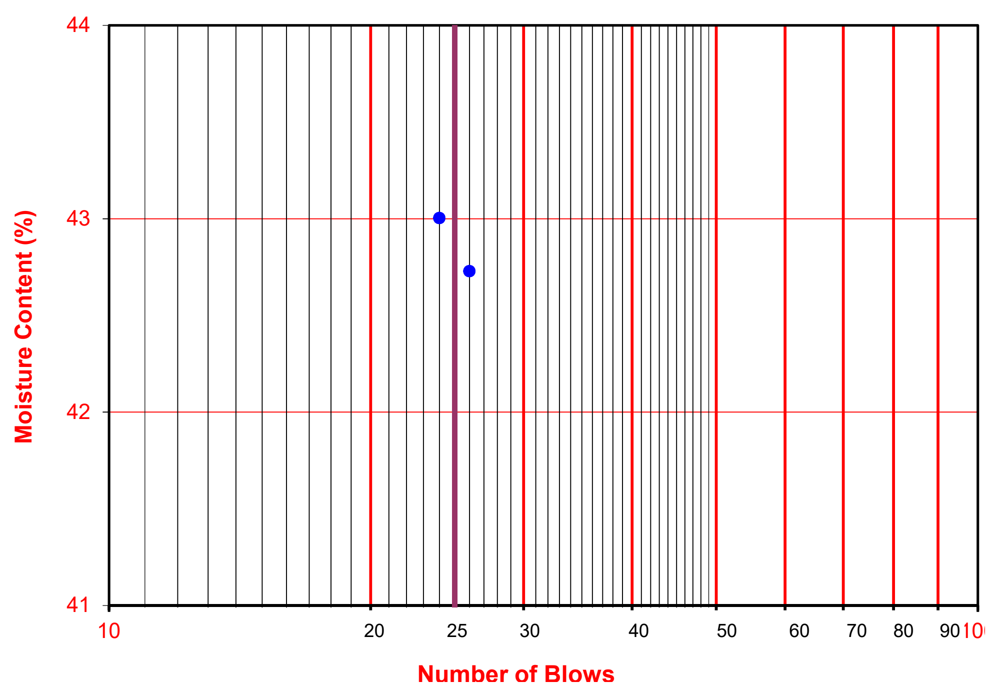
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
One-point

☐ Procedure A
Multipoint Test

☒ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Chino Valley Fire Station No. 8	Tested By:	Y. Nguyen	Date:	11/30/21
Project No. :	13353.001	Input By:	G. Bathala	Date:	12/03/21
Boring No.:	LB-6	Checked By:	A. Santos		
Sample No.:	B-1	Depth (ft.)	2.5		
Soil Identification:	Dark grayish brown clayey sand (SC)				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2		
Number of Blows [N]			25	27	Trial 1 =	33
Wet Wt. of Soil + Cont. (g)	9.95	10.00	20.80	20.96	Trial 2 =	33
Dry Wt. of Soil + Cont. (g)	8.73	8.78	15.86	16.02	Ave. LL =	**33**
Wt. of Container (g)	1.11	1.11	1.00	1.06	(see equation below)	
Moisture Content (%) [Wn]	16.01	15.91	33.24	33.02		

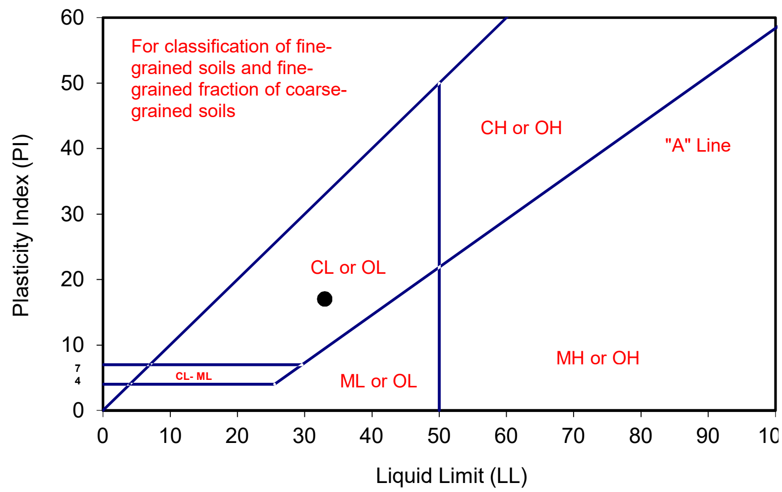
Liquid Limit	**33**
Plastic Limit	**16**
Plasticity Index	**17**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 9.49

One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$

(Wn = water content, N = number of blows)



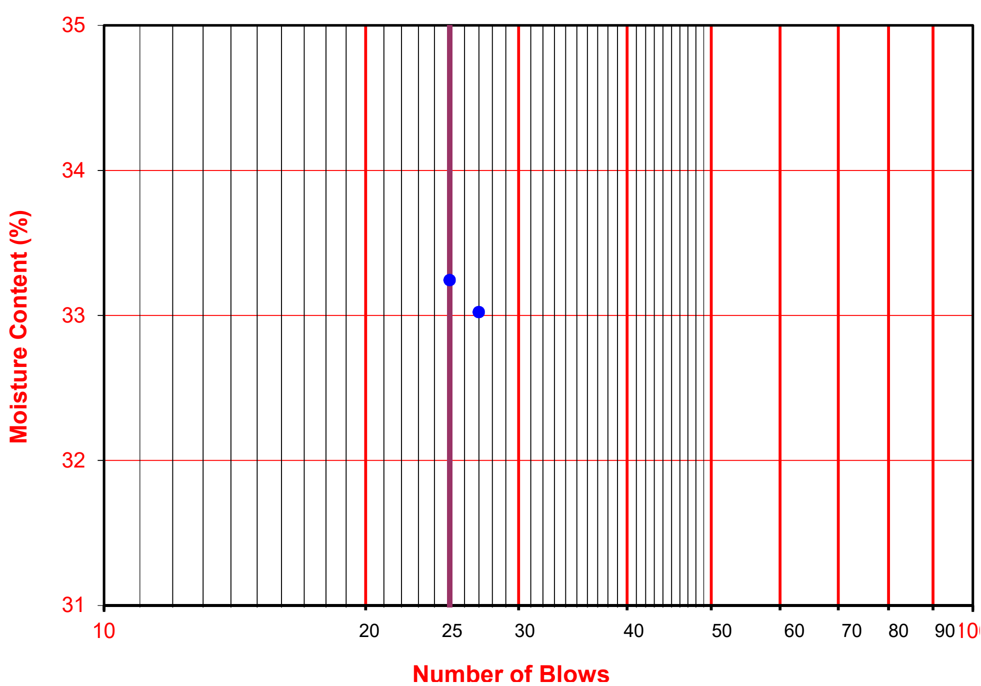
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
One-point

☐ Procedure A
Multipoint Test

☒ Procedure B
One-point Test



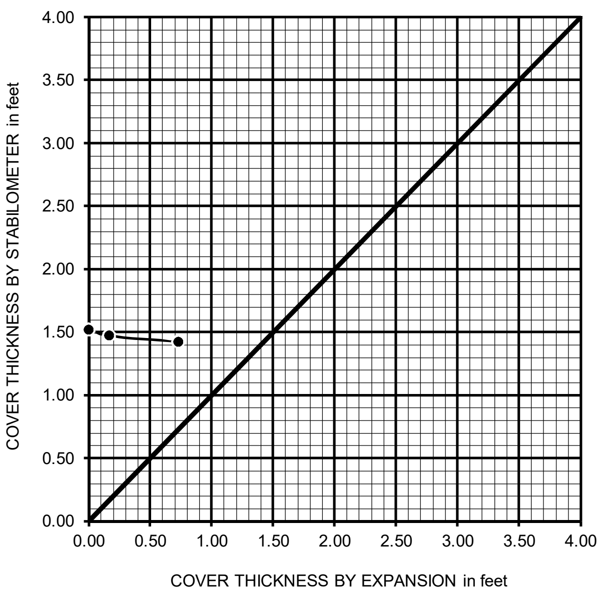
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Chino Valley Fire Station No. 8	PROJECT NUMBER:	13353.001
BORING NUMBER:	LB-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark brown lean clay (CL)	DATE COMPLETED:	11/29/2021

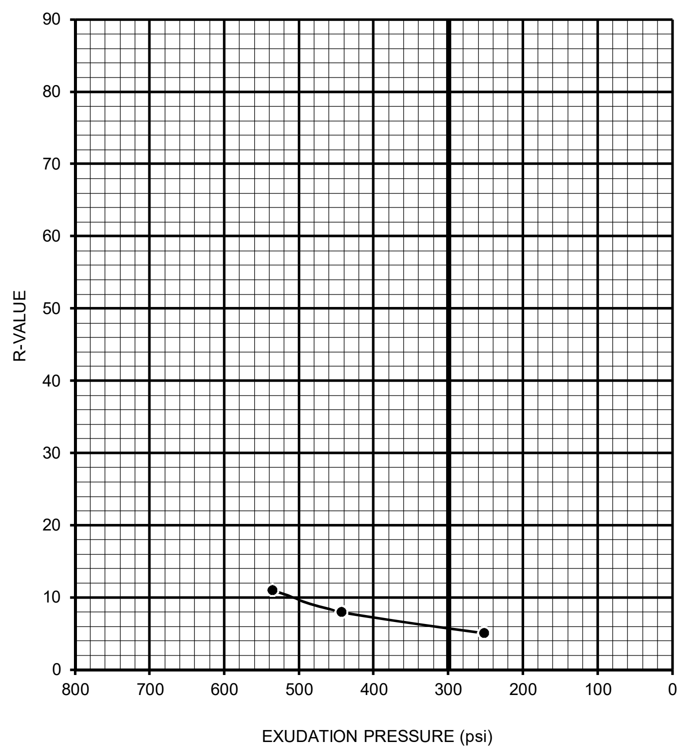
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	21.5	22.9	24.3
HEIGHT OF SAMPLE, Inches	2.51	2.51	2.49
DRY DENSITY, pcf	105.3	104.3	104.9
COMPACTOR PRESSURE, psi	80	70	50
EXUDATION PRESSURE, psi	535	443	251
EXPANSION, Inches x 10exp-4	22	5	0
STABILITY Ph 2,000 lbs (160 psi)	134	140	145
TURNS DISPLACEMENT	3.75	4.40	4.60
R-VALUE UNCORRECTED	11	8	5
R-VALUE CORRECTED	11	8	5

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.42	1.47	1.52
EXPANSION PRESSURE THICKNESS, ft.	0.73	0.17	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



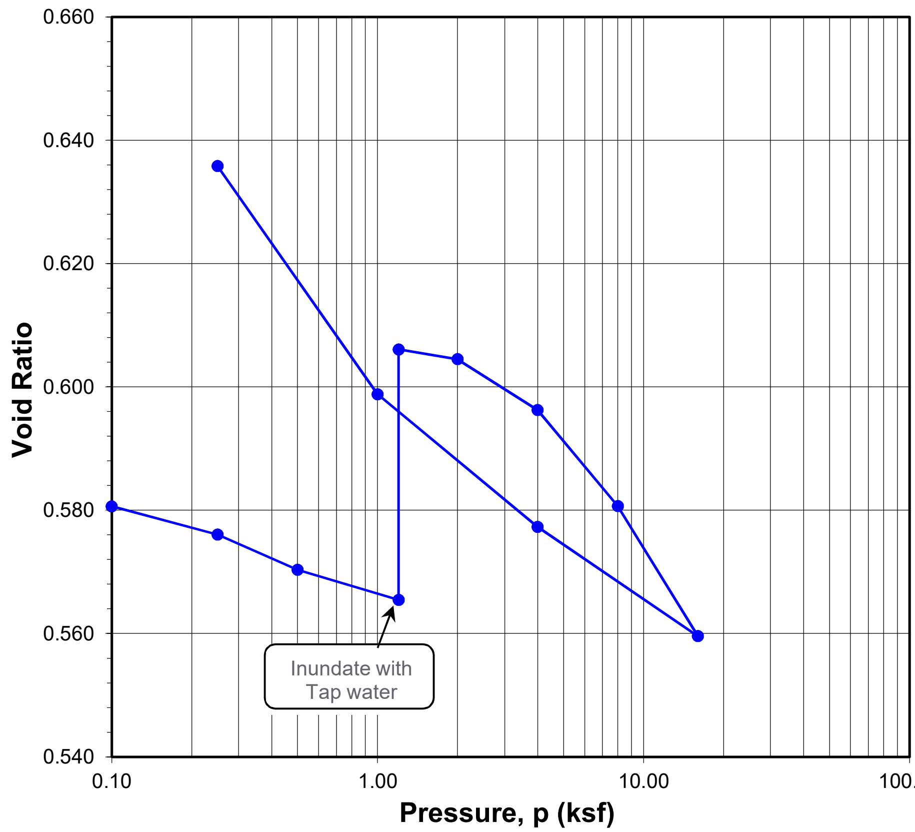
R-VALUE BY EXPANSION:	9
R-VALUE BY EXUDATION:	6
EQUILIBRIUM R-VALUE:	6

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Chino Valley Fire Station No. 8	Tested By: GB/YN	Date:	11/29/21
Project No.:	13353.001	Checked By: A. Santos	Date:	12/06/21
Boring No.:	LB-3	Depth (ft.): 10.0		
Sample No.:	R-4	Sample Type:	Ring	
Soil Identification:	Light olive brown lean clay (CL)			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	195.89
Weight of Ring (g)	44.85
Height after consol. (in.)	1.0346
Before Test	
Wt.Wet Sample+Cont. (g)	183.36
Wt.of Dry Sample+Cont. (g)	166.75
Weight of Container (g)	73.57
Initial Moisture Content (%)	17.8
Initial Dry Density (pcf)	106.6
Initial Saturation (%)	83
Initial Vertical Reading (in.)	0.2781
After Test	
Wt.of Wet Sample+Cont. (g)	256.32
Wt. of Dry Sample+Cont. (g)	226.02
Weight of Container (g)	55.14
Final Moisture Content (%)	24.04
Final Dry Density (pcf)	101.3
Final Saturation (%)	98
Final Vertical Reading (in.)	0.3092
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

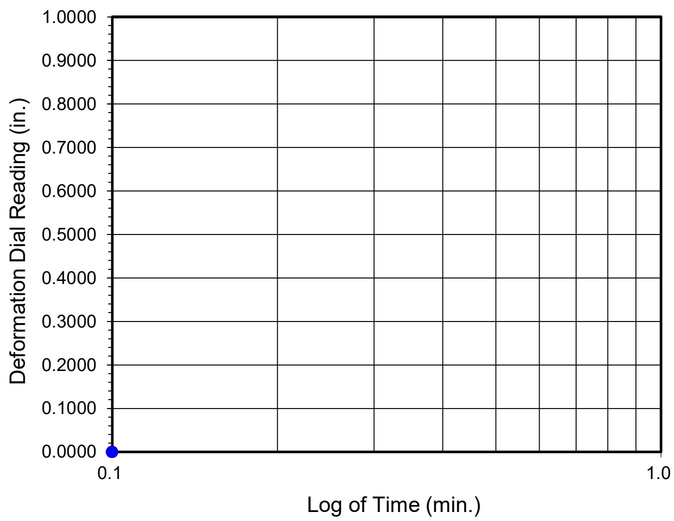


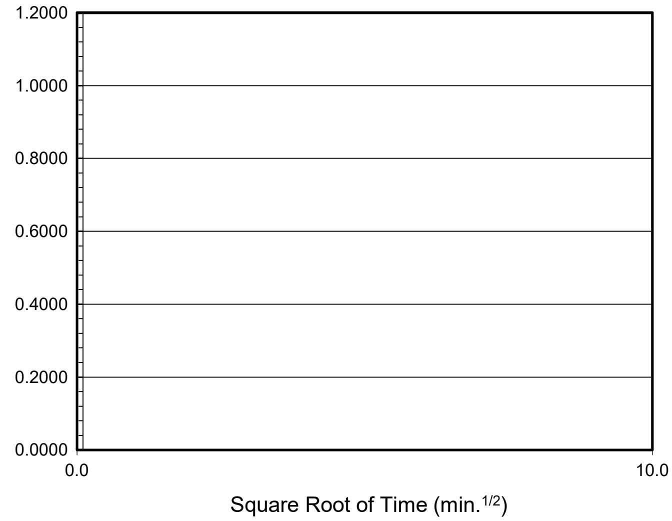
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2778	0.9997	0.00	0.03	0.581	0.03
0.25	0.2744	0.9963	0.05	0.37	0.576	0.32
0.50	0.2702	0.9921	0.11	0.79	0.570	0.68
1.20	0.2660	0.9879	0.22	1.21	0.565	0.99
1.20	0.2917	1.0136	0.22	-1.36	0.606	-1.58
2.00	0.2898	1.0117	0.31	-1.17	0.605	-1.48
4.00	0.2832	1.0051	0.45	-0.51	0.596	-0.96
8.00	0.2718	0.9937	0.61	0.64	0.581	0.03
16.00	0.2564	0.9783	0.81	2.17	0.560	1.36
4.00	0.2690	0.9909	0.67	0.91	0.577	0.24
1.00	0.2844	1.0063	0.49	-0.63	0.599	-1.12
0.25	0.3092	1.0311	0.35	-3.11	0.636	-3.46

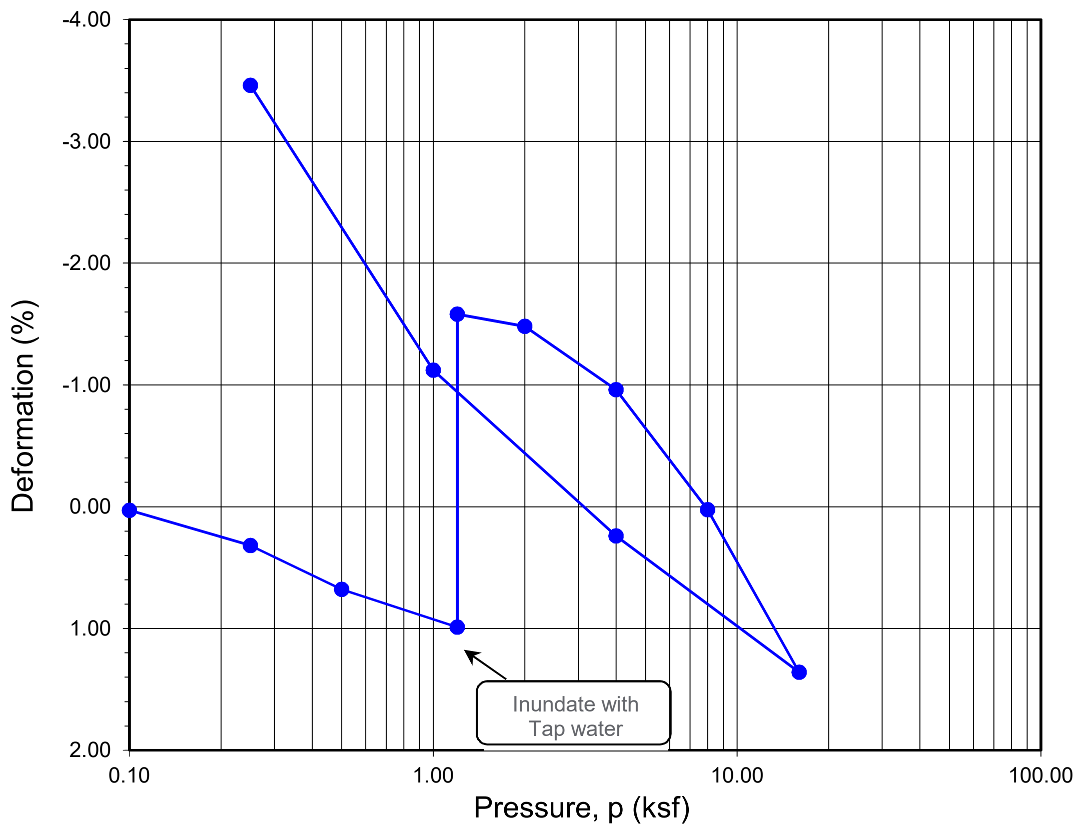
Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-3	**R-4**	**10.0**	**17.8**	**24.0**	**106.6**	**101.3**	**0.581**	**0.636**	**83**	**98**

Soil Identification: Light olive brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13353.001

Chino Valley Fire Station No. 8

12-21

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Chino Valley Fire Station No. 8 | Tested By : JD/OHF | Date: 11/29/21

Project No. : 13351.001 | Checked By: A. Santos | Date: 12/06/21

Boring No.	LB-4			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Dark olive (CL)			
Wet Weight of Soil + Container (g)	113.05			
Dry Weight of Soil + Container (g)	112.48			
Weight of Container (g)	65.68			
Moisture Content (%)	1.22			
Weight of Soaked Soil (g)	100.44			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	17			
Crucible No.	14			
Furnace Temperature (°C)	860			
Time In / Time Out	8:00/8:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	19.6865			
Wt. of Crucible (g)	19.6845			
Wt. of Residue (g) (A)	0.0020			
PPM of Sulfate (A) x 41150	82.30			
PPM of Sulfate, Dry Weight Basis	**83**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	50			
PPM of Chloride, Dry Wt. Basis	**51**			

pH TEST, DOT California Test 643

pH Value	**7.49**			
Temperature °C	21.5			

SOIL RESISTIVITY TEST

DOT CA TEST 643

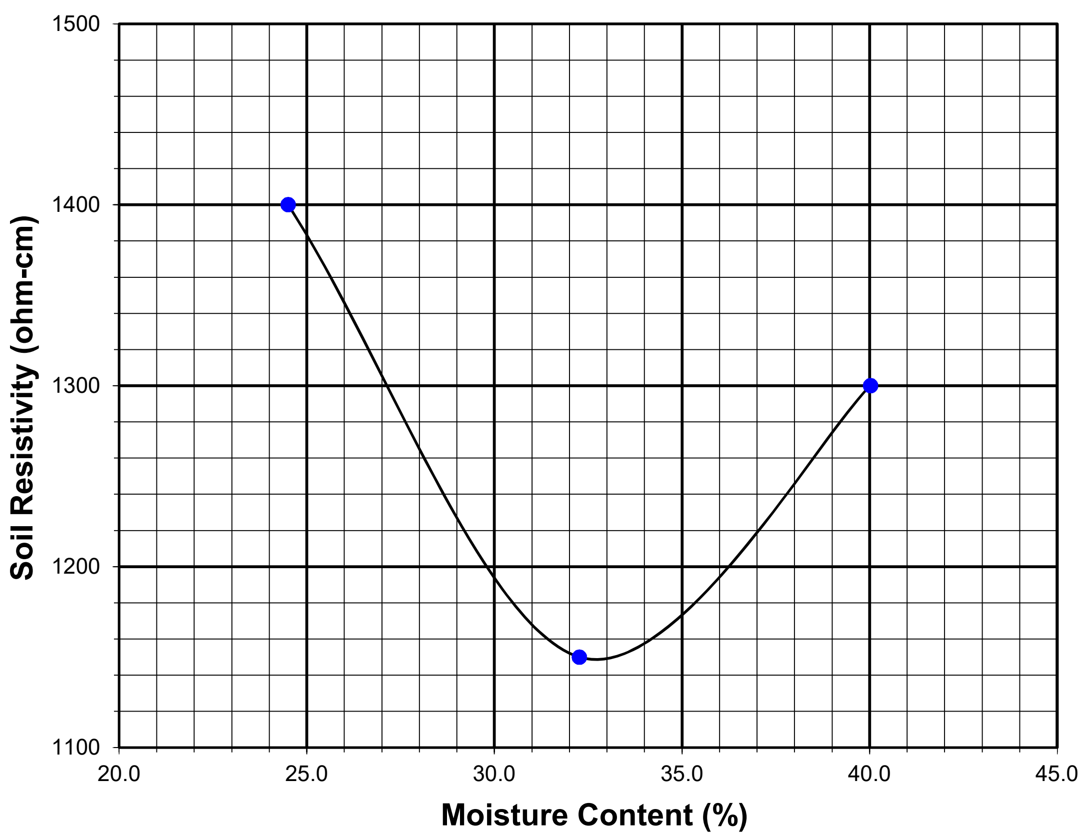
Project Name:	Chino Valley Fire Station No. 8	Tested By :	G. Berdy	Date:	07/06/21
Project No. :	13351.001	Checked By:	A. Santos	Date:	12/06/21
Boring No.:	LB-4	Depth (ft.) :	0-5		
Sample No. :	B-1				
Soil Identification:*	Dark olive (CL)				

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	24.50	1400	1400
2	40	32.27	1150	1150
3	50	40.03	1300	1300
4				
5				

Moisture Content (%) (MCi)	1.22
Wet Wt. of Soil + Cont. (g)	113.05
Dry Wt. of Soil + Cont. (g)	112.48
Wt. of Container (g)	65.68
Container No.	
Initial Soil Wt. (g) (Wt)	130.40
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	
1150	**32.5**	**83**	**51**	**7.49**	**21.5**



APPENDIX C

SEISMIC

OSHPD

Proposed Fire Station No. 8

Latitude, Longitude: 33.9583, -117.7149

Date	11/29/2021, 9:28:37 AM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.947	MCE_R ground motion. (for 0.2 second period)
S_1	0.684	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.336	Site-modified spectral acceleration value
S_{M1}	0.957	Site-modified spectral acceleration value
S_{DS}	1.557	Numeric seismic design value at 0.2 second SA
S_{D1}	0.638	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.836	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.003	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.947	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.152	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.362	Factored deterministic acceleration value. (0.2 second)
S1RT	0.684	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.755	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.773	Factored deterministic acceleration value. (1.0 second)
PGAd	0.972	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.905	Mapped value of the risk coefficient at short periods
C_{R1}	0.905	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. 2014 (u…

Latitude

Decimal degrees

33.9583

Longitude

Decimal degrees, negative values for western longitudes

-117.7149

Site Class

537 m/s (Site class C)

Spectral Period

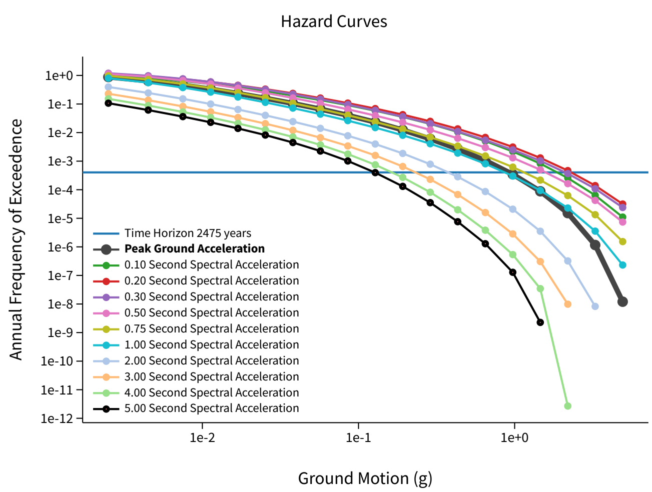
Peak Ground Acceleration

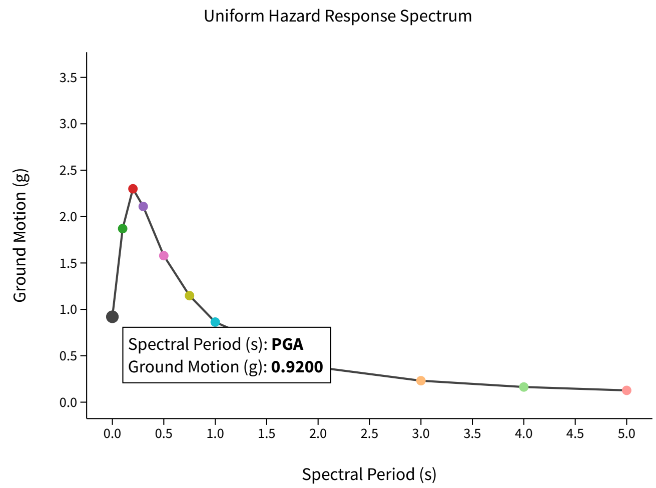
Time Horizon

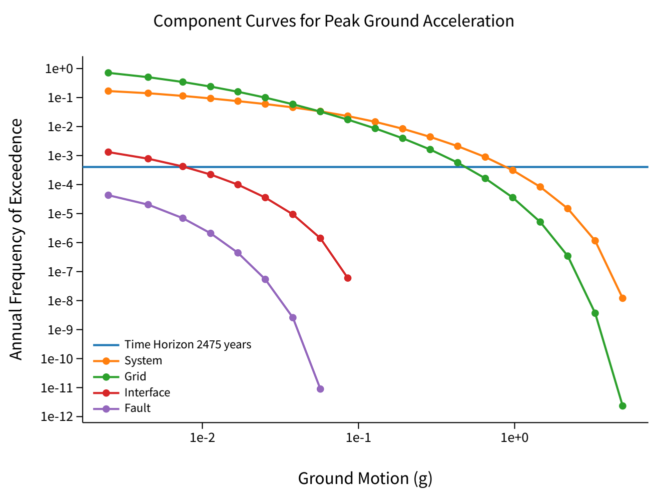
Return period in years

2475

Hazard Curve





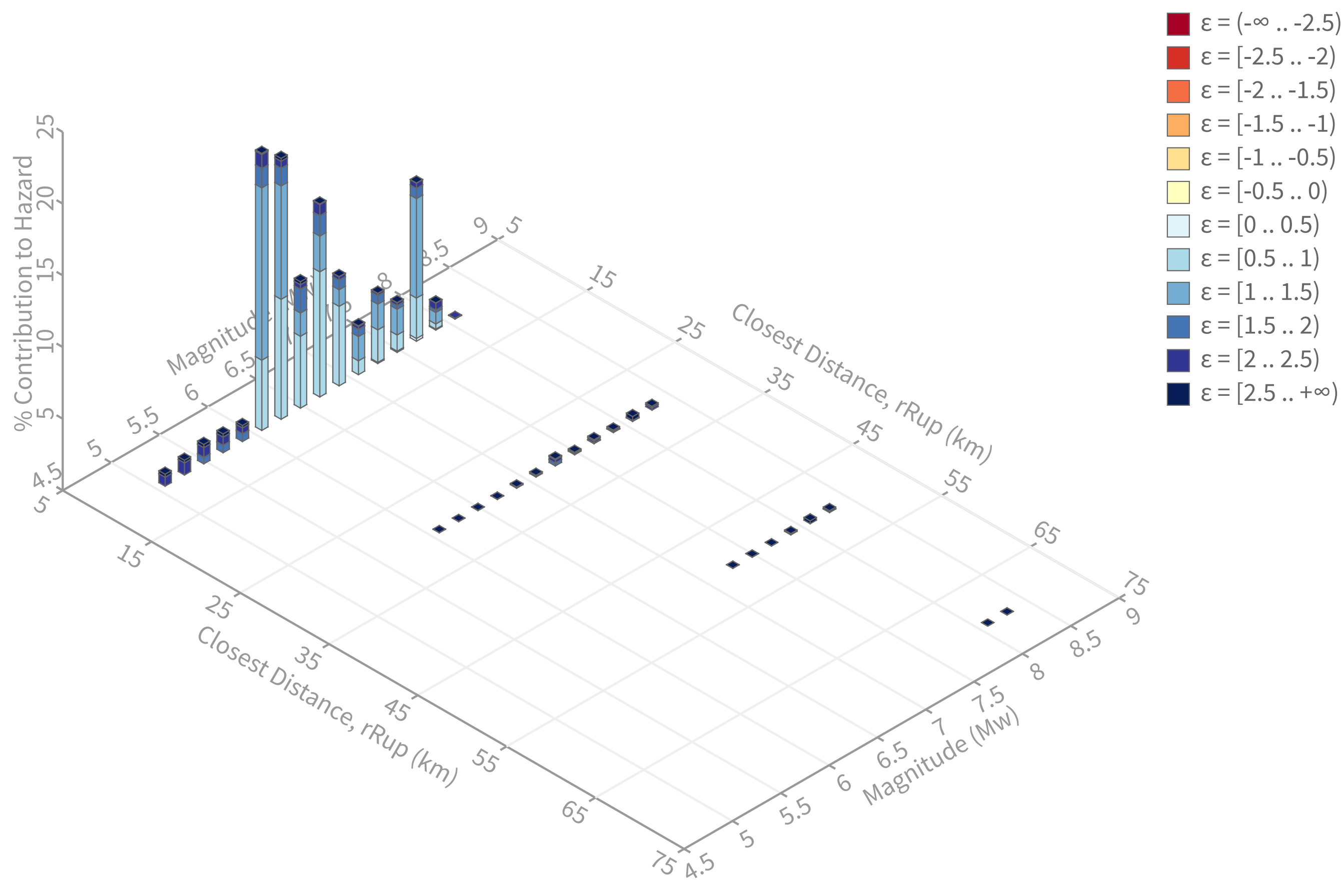


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.91999713 g

Recovered targets

Return period: 2776.0519 yrs
Exceedance rate: 0.00036022382 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 6.67
r: 4.92 km
ε_0: 1.21 σ

Mode (largest m-r bin)

m: 6.09
r: 2.18 km
ε_0: 1.18 σ
Contribution: 19.32 %

Mode (largest m-r-ε_0 bin)

m: 6.08
r: 1.25 km
ε_0: 1.12 σ
Contribution: 12.02 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							55.86
Chino alt 1 [1]		1.02	6.32	0.96	117.710°W	33.957°N	107.67	30.57
Chino alt 1 [2]		1.67	6.58	0.98	117.703°W	33.950°N	129.06	8.01
Whittier alt 1 [3]		7.90	7.40	1.18	117.758°W	33.897°N	210.36	7.99
Chino alt 1 [3]		6.32	6.82	1.51	117.669°W	33.917°N	137.12	1.73
Whittier alt 1 [2]		7.90	7.15	1.31	117.758°W	33.897°N	210.36	1.48
Yorba Linda [2]		3.35	6.77	1.08	117.719°W	33.931°N	187.45	1.25
UC33brAvg_FM32	System							34.74
Chino alt 2 [1]		1.56	6.68	0.87	117.712°W	33.965°N	19.69	18.57
Whittier alt 2 [2]		8.10	7.51	1.20	117.755°W	33.895°N	207.76	7.80
Chino alt 2 [2]		6.04	6.75	1.52	117.670°W	33.921°N	135.38	1.81
Yorba Linda [2]		3.35	7.44	0.81	117.719°W	33.931°N	187.45	1.26
Richfield [0]		10.97	6.38	1.96	117.803°W	33.889°N	226.51	1.19
UC33brAvg_FM31 (opt)	Grid							4.70
PointSourceFinite: -117.715, 33.999		6.49	5.82	1.78	117.715°W	33.999°N	0.00	1.17
PointSourceFinite: -117.715, 33.999		6.49	5.82	1.78	117.715°W	33.999°N	0.00	1.17
UC33brAvg_FM32 (opt)	Grid							4.70
PointSourceFinite: -117.715, 33.999		6.67	5.73	1.85	117.715°W	33.999°N	0.00	1.09
PointSourceFinite: -117.715, 33.999		6.67	5.73	1.85	117.715°W	33.999°N	0.00	1.09

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 13353.001 Fire Station No. 8

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)					Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2	LB-3	LB-4	LB-6		1.3	
5	7.5	34	40	60	30	14	36	46	0.16
10	5	18	39	16	26	23	24	32	0.16
15	5	20	28	43	13	4	22	28	0.18
20	5	13	12	60	24	22	26	34	0.15
25	5	16	19	74	15		31	40	0.12
30	5		41		15		28	36	0.14
35	5				15		15	20	0.26
40	5				60		60	78	0.06
45	5				100		100	100	0.05
50	7.5				100		100	100	0.08
60	10				100	* Based on blows from 40 feet	100	100	0.10
70	10				100		100	100	0.10
80	10				100		100	100	0.10
90	10				100		100	100	0.10
100	5				100		100	100	0.05
Summation	100								1.80

Navg = Sum(di) / Sum(di / Ni) = 56

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

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	**50.001**	**100**	1200	2500	366	762	56	1343
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

SITE CLASS, Table 20.3-1: C

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

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	1343	409
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1235	377
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1035	316
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	1010	308

Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Description: Chino Hills Fire Station No. 68; Case 1; PGAm 1.003; design GW 41; No overex 0
Project No.: 13353.001
Dec 2021

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve	Boring Location Coordinates X (ft)	Boring Location Coordinates Y (ft)
LB-1	200	41		0	794	753	-527.4	-265.8
LB-2	200	41		0	779	738	-396.4	-98.1
LB-3	200	41		0	775	734	-400.1	27.553
LB-4	200	41		0	784	743	-499	-21.18
LB-5	200	41		0	779	738	-503.5	77.946
LB-6	200	41		0	767	726	-107.6	-46.22
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		

General Parameters:	
a_{max} =	1.00g
M_W =	6.7
MSF eq:	1
MSF =	1.33
Hammer Efficiency =	84
C_E =	1.40
C_B =	1
C_S for SPT?	TRUE
Unlined, but room for liner	
Rod Stickup (feet) =	3
Ring sample correction =	0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Project: Chino Hills Fire Station No. 68; Case 1; PGAm 1.003; design GW 41; No overex 0

Project No.: 13353.001

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8		65	120	78	2	1	50.7	300	90.5	113.6	>Range	300	0.65	0.49	**NonLiq**	113.6	0.01		0.00	0.3
LB-1	3.8 to 6.3	5	2.5		65	120	56	2	1	36.4	600	65.0	83.0	>Range	600	0.64	0.48	**NonLiq**	83.0	0.07		0.02	0.3
LB-1	6.3 to 8.8	7.5	2.5		65	120	44	2	1	28.6	900	48.8	63.5	>Range	900	0.64	0.48	**NonLiq**	63.5	0.05		0.02	0.2
LB-1	8.8 to 12.5	10	3.8		55	120	30	2	1	19.5	1200	30.6	41.7	>Range	1200	0.64	0.48	**NonLiq**	41.7	0.12		0.05	0.2
LB-1	12.5 to 17.5	15	5.0		65	120	34	2	1	22.1	1800	28.3	39.0	>Range	1800	0.63	0.47	**NonLiq**	39.0	0.30		0.18	0.2
LB-1	17.5 to 22.5	20	5.0	n	65	120	15	1	1.23	18.4	2400	22.8	32.4	>Range	2400	0.62	0.47	**NonLiq**	32.4	0.00		0.00	0.0
LB-1	22.5 to 27.0	25	4.5	n	55	120	26	2	1	16.9	3000	18.8	27.5	>Range	3000	0.61	0.46	**NonLiq**	27.5	0.00		0.00	0.0
LB-2	0 to 7.5	5	7.5		55	120	66	2	1	42.9	600	76.6	96.9	>Range	600	0.64	0.48	**NonLiq**	96.9	0.06		0.05	0.1
LB-2	7.5 to 12.5	10	5.0		**66**	120	65	2	1	42.3	1200	66.3	84.6	>Range	1200	0.64	0.48	**NonLiq**	84.6	0.06		0.04	0.1
LB-2	12.5 to 17.5	15	5.0		55	120	47	2	1	30.6	1800	39.2	52.0	>Range	1800	0.63	0.47	**NonLiq**	52.0	0.06		0.04	0.0
LB-2	17.5 to 22.5	20	5.0	n	66	120	12	1	1.17	14.1	2400	17.5	26.0	>Range	2400	0.62	0.47	**NonLiq**	26.0	0.00		0.00	0.0
LB-2	22.5 to 27.5	25	5.0	n	66	120	30	2	1	19.5	3000	21.6	31.0	>Range	3000	0.61	0.46	**NonLiq**	31.0	0.00		0.00	0.0
LB-2	27.5 to 32.0	30	4.5	n	66	120	41	1	1.3	53.3	3600	56.8	73.2	>Range	3600	0.61	0.45	**NonLiq**	73.2	0.00		0.00	0.0
LB-3	0 to 3.8	2.5	3.8		**79**	120	55	2	1	35.8	300	63.8	81.6	>Range	300	0.65	0.49	**NonLiq**	81.6	0.01		0.01	0.2
LB-3	3.8 to 6.3	5	2.5		79	120	100	2	1	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.03		0.01	0.2
LB-3	6.3 to 8.8	7.5	2.5		79	120	100	2	1	65.0	900	110.9	138.1	>Range	900	0.64	0.48	**NonLiq**	138.1	0.02		0.01	0.2
LB-3	8.8 to 12.5	10	3.8		79	120	27	2	1	17.6	1200	27.5	38.1	>Range	1200	0.64	0.48	**NonLiq**	38.1	0.39		0.18	0.2
LB-3	12.5 to 17.5	15	5.0	n	55	120	43	1	1.3	55.9	1800	71.6	91.0	>Range	1800	0.63	0.47	**NonLiq**	91.0	0.00		0.00	0.0
LB-3	17.5 to 22.5	20	5.0	n	55	120	100	2	1	65.0	2400	80.6	101.8	>Range	2400	0.62	0.47	**NonLiq**	101.8	0.00		0.00	0.0
LB-3	22.5 to 27.0	25	4.5	n	55	120	74	1	1.3	96.2	3000	106.7	133.1	>Range	3000	0.61	0.46	**NonLiq**	133.1	0.00		0.00	0.0
LB-4	0 to 7.5	5	7.5		**72**	120	50	2	1	32.5	600	58.0	74.6	>Range	600	0.64	0.48	**NonLiq**	74.6	0.08		0.07	0.6
LB-4	7.5 to 12.5	10	5.0		72	120	43	2	1	28.0	1200	43.9	57.6	>Range	1200	0.64	0.48	**NonLiq**	57.6	0.09		0.05	0.5
LB-4	12.5 to 17.5	15	5.0		72	120	13	1	1.2	15.6	1800	20.0	29.0	0.409	1800	0.63	0.47	**NonLiq**	29.0	0.68		0.41	0.5
LB-4	17.5 to 22.5	20	5.0		72	120	40	2	1	26.0	2400	32.3	43.7	>Range	2400	0.62	0.47	**NonLiq**	43.7	0.13		0.08	0.1
LB-4	22.5 to 27.5	25	5.0	n	**76**	120	15	1	1.2	18.0	3000	20.0	29.0	>Range	3000	0.61	0.46	**NonLiq**	29.0	0.00		0.00	0.0
LB-4	27.5 to 32.5	30	5.0	n	76	120	25	2	1	16.3	3600	17.3	25.8	>Range	3600	0.61	0.45	**NonLiq**	25.8	0.00		0.00	0.0
LB-4	32.5 to 37.5	35	5.0	n	76	120	15	1	1.17	17.6	4200	17.4	25.9	>Range	4200	0.58	0.43	**NonLiq**	25.9	0.00		0.00	0.0
LB-4	37.5 to 41.0	40	3.5	n	76	120	100	2	1	65.0	4800	60.0	77.0	>Range	4800	0.55	0.41	**NonLiq**	77.0	0.00		0.00	0.0
LB-4	41.0 to 42.5	40	1.5	n	76	120	100	2	1	65.0	4800	60.0	77.0	>Range	4800	0.55	0.41	**NonLiq**	77.0			0.00	0.0
LB-4	42.5 to 47.0	45	4.5	n	76	120	100	1	1.3	130.0	5400	113.2	140.8	>Range	5150.4	0.55	0.41	**NonLiq**	140.8			0.00	0.0
LB-6	0 to 3.8	2.5	3.8		**48**	120	51	2	1	33.2	300	59.2	76.0	>Range	300	0.65	0.49	**NonLiq**	76.0	0.02		0.01	0.1
LB-6	3.8 to 6.3	5	2.5		65	120	24	1	1.3	31.2	600	55.7	71.8	>Range	600	0.64	0.48	**NonLiq**	71.8	0.08		0.02	0.1
LB-6	6.3 to 8.8	7.5	2.5		65	120	41	2	1	26.7	900	45.5	59.6	>Range	900	0.64	0.48	**NonLiq**	59.6	0.06		0.02	0.1

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-6	8.8 to 12.5	10	3.8		65	120	39	2	1	25.4	1200	39.8	52.8	>Range	1200	0.64	0.48	**NonLiq**	52.8	0.09		0.04	0.0
LB-6	12.5 to 17.5	15	5.0	n	65	120	4	1	1.1	4.4	1800	5.6	**11.8**	>Range	1800	0.63	0.47	**NonLiq**	11.8	0.00		0.00	0.0
LB-6	17.5 to 22.0	20	4.5	n	65	120	36	2	1	23.4	2400	29.0	39.8	>Range	2400	0.62	0.47	**NonLiq**	39.8	0.00		0.00	0.0

Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Description: Chino Hills Fire Station No. 68; Case 3; PGAm 1.003; design GW 41; Overex./scarify 11.5
Project No.: 13353.001
Dec 2021

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve	Boring Location Coordinates X (ft)	Boring Location Coordinates Y (ft)
LB-1	200	41		11.5	794	753	-527.4	-265.8
LB-2	200	41		11.5	779	738	-396.4	-98.1
LB-3	200	41		11.5	775	734	-400.1	27.553
LB-4	200	41		11.5	784	743	-499	-21.18
LB-5	200	41		11.5	779	738	-503.5	77.946
LB-6	200	41		11.5	767	726	-107.6	-46.22
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		
						0		

General Parameters:	
a_{max} =	1.00g
M_W =	6.7
MSF eq:	1
MSF =	1.33
Hammer Efficiency =	84
C_E =	1.40
C_B =	1
C_S for SPT?	TRUE
Unlined, but room for liner	
Rod Stickup (feet) =	3
Ring sample correction =	0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Project: Chino Hills Fire Station No. 68; Case 3; PGAm 1.003; design GW 41; Overex./scarify 11.5

Project No.: 13353.001

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8	OX	65	120	50	1	1.3	65.0	300	116.0	144.2	>Range	300	0.65	0.49	**NonLiq**	144.2	0.00		0.00	0.2
LB-1	3.8 to 6.3	5	2.5	OX	65	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.00		0.00	0.2
LB-1	6.3 to 8.8	7.5	2.5	OX	65	120	50	1	1.3	65.0	900	110.9	138.1	>Range	900	0.64	0.48	**NonLiq**	138.1	0.00		0.00	0.2
LB-1	8.8 to 11.5	10	2.8	OX	55	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1200	0.64	0.48	**NonLiq**	127.4	0.00		0.00	0.2
LB-1	11.5 to 12.5	10	1.0		55	120	30	2	1	19.5	1200	30.6	41.7	>Range	1200	0.64	0.48	**NonLiq**	41.7	0.12		0.01	0.2
LB-1	12.5 to 17.5	15	5.0		65	120	34	2	1	22.1	1800	28.3	39.0	>Range	1800	0.63	0.47	**NonLiq**	39.0	0.30		0.18	0.2
LB-1	17.5 to 22.5	20	5.0	n	65	120	15	1	1.23	18.4	2400	22.8	32.4	>Range	2400	0.62	0.47	**NonLiq**	32.4	0.00		0.00	0.0
LB-1	22.5 to 27.0	25	4.5	n	55	120	26	2	1	16.9	3000	18.8	27.5	>Range	3000	0.61	0.46	**NonLiq**	27.5	0.00		0.00	0.0
LB-2	0 to 7.5	5	7.5	OX	55	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.00		0.00	0.0
LB-2	7.5 to 11.5	10	4.0	OX	**66**	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1200	0.64	0.48	**NonLiq**	127.4	0.00		0.00	0.0
LB-2	11.5 to 12.5	10	1.0		**66**	120	65	2	1	42.3	1200	66.3	84.6	>Range	1200	0.64	0.48	**NonLiq**	84.6	0.06		0.01	0.0
LB-2	12.5 to 17.5	15	5.0		55	120	47	2	1	30.6	1800	39.2	52.0	>Range	1800	0.63	0.47	**NonLiq**	52.0	0.06		0.04	0.0
LB-2	17.5 to 22.5	20	5.0	n	66	120	12	1	1.17	14.1	2400	17.5	26.0	>Range	2400	0.62	0.47	**NonLiq**	26.0	0.00		0.00	0.0
LB-2	22.5 to 27.5	25	5.0	n	66	120	30	2	1	19.5	3000	21.6	31.0	>Range	3000	0.61	0.46	**NonLiq**	31.0	0.00		0.00	0.0
LB-2	27.5 to 32.0	30	4.5	n	66	120	41	1	1.3	53.3	3600	56.8	73.2	>Range	3600	0.61	0.45	**NonLiq**	73.2	0.00		0.00	0.0
LB-3	0 to 3.8	2.5	3.8	OX	**79**	120	50	1	1.3	65.0	300	116.0	144.2	>Range	300	0.65	0.49	**NonLiq**	144.2	0.00		0.00	0.0
LB-3	3.8 to 6.3	5	2.5	OX	79	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.00		0.00	0.0
LB-3	6.3 to 8.8	7.5	2.5	OX	79	120	50	1	1.3	65.0	900	110.9	138.1	>Range	900	0.64	0.48	**NonLiq**	138.1	0.00		0.00	0.0
LB-3	8.8 to 11.5	10	2.8	OX	79	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1200	0.64	0.48	**NonLiq**	127.4	0.00		0.00	0.0
LB-3	11.5 to 12.5	10	1.0		79	120	27	2	1	17.6	1200	27.5	38.1	>Range	1200	0.64	0.48	**NonLiq**	38.1	0.39		0.05	0.0
LB-3	12.5 to 17.5	15	5.0	n	55	120	43	1	1.3	55.9	1800	71.6	91.0	>Range	1800	0.63	0.47	**NonLiq**	91.0	0.00		0.00	0.0
LB-3	17.5 to 22.5	20	5.0	n	55	120	100	2	1	65.0	2400	80.6	101.8	>Range	2400	0.62	0.47	**NonLiq**	101.8	0.00		0.00	0.0
LB-3	22.5 to 27.0	25	4.5	n	55	120	74	1	1.3	96.2	3000	106.7	133.1	>Range	3000	0.61	0.46	**NonLiq**	133.1	0.00		0.00	0.0
LB-4	0 to 7.5	5	7.5	OX	**72**	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.00		0.00	0.5
LB-4	7.5 to 11.5	10	4.0	OX	72	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1200	0.64	0.48	**NonLiq**	127.4	0.00		0.00	0.5
LB-4	11.5 to 12.5	10	1.0		72	120	43	2	1	28.0	1200	43.9	57.6	>Range	1200	0.64	0.48	**NonLiq**	57.6	0.09		0.01	0.5
LB-4	12.5 to 17.5	15	5.0		72	120	13	1	1.2	15.6	1800	20.0	29.0	0.409	1800	0.63	0.47	**NonLiq**	29.0	0.68		0.41	0.5
LB-4	17.5 to 22.5	20	5.0		72	120	40	2	1	26.0	2400	32.3	43.7	>Range	2400	0.62	0.47	**NonLiq**	43.7	0.13		0.08	0.1
LB-4	22.5 to 27.5	25	5.0	n	**76**	120	15	1	1.2	18.0	3000	20.0	29.0	>Range	3000	0.61	0.46	**NonLiq**	29.0	0.00		0.00	0.0
LB-4	27.5 to 32.5	30	5.0	n	76	120	25	2	1	16.3	3600	17.3	25.8	>Range	3600	0.61	0.45	**NonLiq**	25.8	0.00		0.00	0.0
LB-4	32.5 to 37.5	35	5.0	n	76	120	15	1	1.17	17.6	4200	17.4	25.9	>Range	4200	0.58	0.43	**NonLiq**	25.9	0.00		0.00	0.0
LB-4	37.5 to 41.0	40	3.5	n	76	120	100	2	1	65.0	4800	60.0	77.0	>Range	4800	0.55	0.41	**NonLiq**	77.0	0.00		0.00	0.0
LB-4	41.0 to 42.5	40	1.5	n	76	120	100	2	1	65.0	4800	60.0	77.0	>Range	4800	0.55	0.41	**NonLiq**	77.0			0.00	0.0
LB-4	42.5 to 47.0	45	4.5	n	76	120	100	1	1.3	130.0	5400	113.2	140.8	>Range	5150.4	0.55	0.41	**NonLiq**	140.8			0.00	0.0

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-6	0 to 3.8	2.5	3.8	OX	**48**	120	50	1	1.3	65.0	300	116.0	144.2	>Range	300	0.65	0.49	**NonLiq**	144.2	0.00		0.00	0.0
LB-6	3.8 to 6.3	5	2.5	OX	65	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.64	0.48	**NonLiq**	144.2	0.00		0.00	0.0
LB-6	6.3 to 8.8	7.5	2.5	OX	65	120	50	1	1.3	65.0	900	110.9	138.1	>Range	900	0.64	0.48	**NonLiq**	138.1	0.00		0.00	0.0
LB-6	8.8 to 11.5	10	2.8		65	120	50	1	1.3	65.0	1200	102.0	127.4	>Range	1200	0.64	0.48	**NonLiq**	127.4	0.04		0.01	0.0
LB-6	11.5 to 12.5	10	1.0		65	120	39	2	1	25.4	1200	39.8	52.8	>Range	1200	0.64	0.48	**NonLiq**	52.8	0.09		0.01	0.0
LB-6	12.5 to 17.5	15	5.0	n	65	120	4	1	1.1	4.4	1800	5.6	**11.8**	>Range	1800	0.63	0.47	**NonLiq**	11.8	0.00		0.00	0.0
LB-6	17.5 to 22.0	20	4.5	n	65	120	36	2	1	23.4	2400	29.0	39.8	>Range	2400	0.62	0.47	**NonLiq**	39.8	0.00		0.00	0.0

APPENDIX D

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

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