APPENDIX C
GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION REPORT

Jefferson Union High School District Faculty and Staff Housing
699 Serramonte Boulevard
Daly City, California

Prepared for:

Jefferson Union High School District
699 Serramonte Boulevard, Suite 100
Daly City, California

Prepared by:

Slate Geotechnical Consultants Inc.
Oakland, California

June 13, 2019

Project No.

18-007.00

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June 13, 2019
Project No. 18-007.00

Slate Geotechnical Consultants Inc.
490 43rd Street
Oakland, California 94609

Ms. Tina Van Raaphorst
Associate Superintendent, Business Services
Jefferson Union High School District
699 Serramonte Boulevard, Suite 100
Daly City, California 94015-4132

Subject: GEOTECHNICAL INVESTIGATION REPORT
Jefferson Union High School District Faculty and Staff Housing
699 Serramonte Boulevard
Daly City, California

Dear Ms. Van Raaphorst:

Slate Geotechnical Consultants Inc. (Slate) is pleased to present this report summarizing the findings of our geotechnical investigation to support the design and construction of the proposed Jefferson Union High School District (JUHSD) new Faculty and Staff Housing at 699 Serramonte Boulevard in Daly City, California.

The proposed development includes demolition of an existing parking lot and construction of several at-grade 3- to 4-story wood-frame condominium buildings and common-use areas, as well as installation of a series of puzzle lift-type parking structures. The development can be constructed as planned provided the geotechnical recommendations herein are followed. The main geotechnical/geologic issues affecting the site are strong ground shaking from large earthquake events and long-term differential settlement of structures built within areas of existing undocumented fill.

We appreciate the opportunity to provide geotechnical services for this project. Please call us should you have any questions.

Sincerely yours,

Slate Geotechnical Consultants Inc.

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Jefferson Union High School District Faculty and Staff Housing
699 Serramonte Boulevard
Daly City, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Slate Geotechnical Consultants Inc. (Slate) for the proposed Jefferson Union High School District (JUHSD) new Faculty and Staff Housing development at 699 Serramonte Boulevard in Daly City, California. The project site is located adjacent to the southeast corner of Serramonte Boulevard and Sir Francis Boulevard and is currently used as surface parking for the nearby JUHSD facility as shown on the Site Location Map, Figure 1.

Based on our review of the conceptual designs prepared by Seidel Architects, dated Jan 31, 2019, the proposed development consists of a complex of three- to four-story wood-framed apartment buildings, communal/administration buildings, and a series of two-story scissor lift parking structures. All structures are planned to be constructed at-grade and supported on isolated spread footings and/or strip footings. It is our understanding that the existing parking lot will be demolished and regraded for the new development.

2.0 SCOPE OF WORK

The primary purpose of this study was to evaluate the subsurface conditions at the site to provide geotechnical recommendations to support the design and construction of the proposed new buildings and site improvements. Our scope of services consisted of:

- performing a subsurface investigation;
- performing geotechnical laboratory tests on selected soil samples;
- developing geotechnical soil parameters, code-based earthquake parameters, foundation design recommendations; and
- providing geotechnical considerations for construction.

3.0 FIELD EXPLRORATION AND LABORATORY TESTING

The geotechnical investigation at the site involved drilling and logging five soil borings and performing geotechnical laboratory testing on select soil samples retrieved from the borings. Prior to the investigation we obtained a drilling permit from the City of Daly City Department of Water and Wastewater Resources and contacted Underground Service Alert (USA) to notify them of our work, in accordance with California state law. We also retained a private utility locator, C. Cruz Sub-Surface Locators, to clear proposed boring locations of existing underground utilities. The field exploration program and laboratory testing program are described in the subsections below.

3.1 Geotechnical Borings

Five borings (B-1 through B-5) were drilled to depths ranging from 29.5 to 50.0 feet below ground surface (bgs) on January 17 and 18, 2019 at the approximate locations shown on Figure 2. The borings were performed by Gregg Drilling of Martinez, California using a Mobile B-61 truck-mounted rig equipped with hollow stem augers (4-inch I.D., 7-inch O.D.).

The borings were sampled at approximately 2½-foot intervals in the upper 10 to 15 feet of each boring, and at 5-foot intervals in the lower portions of the borings using a Modified-California (Mod-Cal) split-barrel sampler (3-inch O.D., 2.5-inch I.D., or 2.43-inch I.D lined with stainless steel tubes) or a Standard Penetration Test (SPT) split-barrel sampler (2-inch O.D., 1.5-inch I.D. with no room for liners). The samplers were advanced up to 18 inches using a 140-pound, automatic-trip safety hammer falling about 30-inches per drop. Blow counts were recorded as the number of hammer blows required to drive the sampler in 6-inch intervals; the field blow count for each drive was recorded as the sum of the final two 6-inch intervals. Soil samples extracted from the borings were classified in the field in general accordance with the Visual-Manual Procedure for Description and Identification of Soils (ASTM International Standard D2488). Mod-Cal sample tubes were capped, and SPT samples were bagged to preserve in situ moisture content.

Soil cuttings generated from the borings were contained in ten 55-gallon steel drums and temporarily stored at the western end of the parking lot. The drums were collected by Belshire Environmental Services, Inc. on January 29, 2019, transported to their staging yard in Foothill Ranch, California, tested for contaminants, and disposed of in a non-hazardous waste facility after confirmation of non-contamination. Upon completion, borings were backfilled with neat cement grout in accordance with City of Daly City requirements.

Final boring logs were developed based on the conditions observed by the field representative during drilling and the results of laboratory tests on selected soil samples. The boring logs, presented in Appendix A, include information specific to each boring, raw (field) blow counts for each drive sample, field and laboratory-supported classifications of the encountered soil, and summaries of laboratory tests performed on select samples. While encountered groundwater levels are typically summarized on these logs, no free groundwater was observed during the drilling of any of the five borings.

3.2 Laboratory Testing

Representative samples collected from the five borings were submitted to Cooper Testing Laboratory of Palo Alto, California for testing. Typical tests included: in-situ moisture content and density, plasticity (Atterberg limits), grain-size distribution (sieve) analyses, corrosion, and R-value. Table 1 summarizes the results of the laboratory testing on select samples. Data sheets provided by the geotechnical laboratory are included in Appendix B. Results of moisture-density, grain size analysis, and Atterberg Limits also are indicated at the corresponding sample locations and depths on the boring logs in Appendix A.

4.0 SITE AND SUBSURFACE CONDITIONS

The following subsections describe the JUHSD site regional geologic, tectonic, and seismic settings, and subsurface and groundwater conditions encountered during the field exploration program.

4.1 Regional Geology

Daly City, California is located along the west side of the San Francisco Peninsula amongst the hilly terrain of the northern Santa Cruz Mountains. The greater San Francisco Bay Area is within the Coast Ranges physiographic province, a series of northwest-trending mountain ranges and valleys that extend along much of the coastal region of California.

The Coast Ranges were formed by folding and faulting of the collisional plate boundary margins during the Plio-Pleistocene era, approximately 5 million years ago. The basement bedrock of the region generally consists of Franciscan Complex sandstone, shale, chert, conglomerate, serpentinite, and metamorphosed volcanic rocks.

The topographic low area of the San Francisco Bay was originally an inland basin that began to fill with water and sediment after the end of the last major glacial period (roughly 10,000 years ago). The basin-filling sediments include fine-grained and sandy sediments deposited in the bay lowlands, and thick packages of sandy alluvial soils deposited as outwash from the surrounding hills.

Geologic mapping indicates that the site vicinity is underlain by the late Pliocene- to early Pleistocene-age Merced Formation, deposited between 3.6 and 1.8 million years ago (Brabb et al., 1998). The Merced Formation generally consists of weak, friable sandstone, siltstone, and claystone, with local fossiliferous beds that are well-cemented. The unit has been identified along a northwest-trending strip up to about 1.4 miles wide on the east side of the San Andreas fault as it crosses the San Francisco Peninsula. Northeast of the site, the Pleistocene-age (deposited 2.6 million to 100,000 years ago) Colma Formation sand is mapped along a northwest-trending zone adjacent to the Merced Formation and parallel to the San Andreas fault. Southeast of the site and on the west side of the San Andreas fault, the terrain is generally mapped as Franciscan Complex greenstone and sandstone (Brabb et al., 1998).

4.2 Tectonic and Seismic Setting

The San Andreas fault system is the primary tectonic plate boundary zone between the North American and Pacific plates and serves as the dominant source of tectonic activity in the region. The fault system accommodates nearly 1 inch/year of total displacement and is composed of numerous faults that generally trend northwest-southeast through the region. Active faults of the San Andreas fault system in the region surrounding the site are shown on Figure 3.

The closest major active faults to the site include the following:

- San Andreas fault, Peninsula segment (0.6 miles or 1.0 km southwest of the site);
- San Andreas fault, North Coast segment (18.7 miles or 30.1 km northwest of the site);
- San Gregorio fault, North segment (5.0 miles or 8.0 km southwest of the site);
- Point Reyes (connector) fault (6.3 miles or 10.1 km west of the site);
- Pilarcitos fault (4.5 miles or 7.2 km southwest of the site);
- Hayward fault, North segment (18.1 miles or 29.1 km northeast of the site); and
- Hayward fault, South segment (18.2 miles or 29.3 km northeast of the site).

Faults in the San Francisco Bay Area have hosted numerous moderate and large magnitude historic earthquakes. The most historically-significant and damaging events in the Bay Area were the 1906 moment magnitude (M_W) 7.8 California (aka, San Francisco) earthquake and the 1989 M_W 6.9 Loma Prieta earthquake. The 1906 earthquake ruptured 270 miles (430 km) of the San Andreas fault along the North Coast, Peninsula, and Santa Cruz Mountain segments. The Peninsula segment of the San Andreas fault was also ruptured by a M_W 7 event in 1838. The Loma Prieta earthquake epicenter was located just west of the Santa Cruz Mountains segment of the San Andreas fault, and the rupture extended to within about 40 miles (64 km) southeast of the site.

A significant level of shaking was likely experienced at the site during both the 1906 San Francisco and 1989 Loma Prieta earthquakes. A compilation of reported earthquake-caused ground failures by Youd and Hoose (1978) indicates that, following the 1906 earthquake, ground surface cracks due to landslides were observed in the hillslopes to the southeast of the cemetery adjacent to the site. Despite the level of shaking felt across the region, no ground failures were reported in the area immediately surrounding the site following the Loma Prieta earthquake (Tinsley et al., 1998). A compilation of reported ground failures resulting from these two earthquakes indicate that there were no reported ground failures in the area immediately surrounding the site nor within the nearby areas mapped within the same geologic units (Youd and Hoose, 1978; Tinsley et al., 1998).

4.3 Surface and Subsurface Conditions

According to a geotechnical evaluation report by Woodward Clyde Consultants (WCC, 1985), the broader JUHSD site straddles a ravine created by the former Chinese Creek drainage that ran southeasterly through the site. Mass grading for the original JUHSD development was performed between 1966 and 1969 to create the building pad for the school and other site improvements. The grading consisted of a significant cut and leveling of a high knoll on the eastern half of the site, mass excavation of the ravine walls up to 80 feet deep and hillside cuts along the western boundary of the site, and subsequent construction of the engineered fill pad with materials sourced from the excavation activities. The final grade of the pad is relatively level with local fill slopes up to 10 feet high and results in areas of exposed native soils at the ground surface, and fill thicknesses up to 140 feet in the area of and south of the existing JUHSD administrative buildings. Based on maps of historical topography (USGS, 1995), the thalweg of the former Chinese Creek was aligned from the northwestern corner of the proposed faculty and staff housing complex and ran south/southeast through the site (Figure 2). The fill is considerably thinner along the contact with the hillside and knoll cuts to the east (near Boring B-3, Figure 2) and increasing in thickness moving west across the site.

In general, the five borings performed for this study encountered 4 to 6 inches of asphalt/aggregate base, underlain by general site (undocumented) fill of variable thickness (about 45 feet in Boring B-1, to less than 5 feet in Boring B-3), underlain by undifferentiated native soil/weathered rock materials. The undocumented fill was generally characterized with variable compositions of sand, clayey sand, and sandy clay ranging in density and consistency from medium dense to very dense, and medium stiff to hard. Field blow counts in the fill ranged from 14 blows per foot (bpf) to 49 bpf. Individual samples of the fill occasionally contained trace shell fragments suggesting some fill zones were sourced from marine sediments excavated from the former Chinese Creek bed. Native soil/weathered rock was characterized as clayey sand and sandy clay ranging in density and consistency from dense to very dense and stiff to hard.

4.4 Site Class

Site Class is defined by ASCE 7-10, Chapter 20 (ASCE/SEI, 2010) as one of six classes (A through F) based on average shear wave velocity (V_{S30}), average SPT blow count (N_{avg30}), or average undrained shear strength (S_{u30}) in the upper 30 meters (100 feet) of a soil profile:

- Site Classes A and B define rock conditions;
- Site Class C defines very dense soil or soft rock conditions with $360 < V_{S30} \leq 760$ m/s, $N_{avg30} > 50$ blows per foot (bpf), or $S_{u30} > 95$ kPa;
- Site Class D defines stiff soil conditions with $180 < V_{S30} \leq 360$ m/s, $15 < N_{avg30} \leq 50$ bpf, or $45 < S_{u30} \leq 95$ kPa;
- Site Class E defines soft soil conditions; and
- Site Class F defines soils vulnerable to potential failure or collapse under seismic loading.

As discussed in Section 4.3, above, the five borings performed for this study generally encountered undocumented fill of variable thickness with blow counts greater than 15 bpf over dense/stiff native soil/weathered rock with field blow counts in excess of 50 blows per foot. Therefore, we judge that Site Class D is the appropriate classification for the development of ground motions for design (see Section 5.0, below) at the JUHSD site.

4.5 Groundwater Conditions

Groundwater was not encountered in any of the borings drilled at the site. Prior geotechnical assessments of the project site (WWC, 1985) make no mention of groundwater concerns or provide considerations for managing groundwater during construction. For the purposes of design and anticipating construction conditions, groundwater may be considered to be deeper than 50 feet below ground surface.

5.0 GROUND MOTIONS FOR DESIGN

The acceleration response spectra for the JUHSD site were developed in accordance with the 2016 California Building Code (CBSC, 2016) and ASCE/SEI 7-10 (ASCE/SEI, 2010). The process for developing the design and MCE_R-level earthquake scenario response spectra is described below.

Response spectra parameters were established using mapped values from the 2008 USGS National Seismic Hazard Mapping Project (NSHMP) design maps (Working Group on California Earthquake Probabilities [WGCEP], 2008). These values were obtained from the USGS/FEMA-NEHRP U.S. Seismic Design Maps web service tool (https://earthquake.usgs.gov/ws/designmaps/asce7-10.html). The values were then modified to develop code-based map-based design and Maximum Considered Earthquake (MCE_R) response spectra following ASCE/SEI 7-10. The USGS web service provides risk-targeted, maximum response orientation, mapped spectral accelerations for the MCE hazard level for a reference site condition, Site Class D, and the mapped long period transition period (T_L) based on the latitude and longitude of the site. The MCE short-period (S_S) and 1-second (S_1) spectral accelerations and T_L for the project site (37.6690° N and 122.4784° W) are shown below, along with the site response adjustment factors, F_a and F_v, which are used to calculate S_{MS} and S_{M1}.

$$S_S = 2.610g \text{ ; } F_a = 1.0 \text{ ; } S_{MS} = 2.610g$$

$$S_1 = 1.253g \text{ ; } F_v = 1.5 \text{ ; } S_{M1} = 1.880g$$

$$T_L = 12 \text{ seconds}$$

Design spectral accelerations S_{DS} and S_{D1} are taken as two-thirds of the S_{MS} and S_{M1}. The design response spectrum (DRS) was developed in accordance with Section 11.4.6 of ASCE/SEI 7-10 as defined by the following equations:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \text{ for } T < T_0$$

$$S_a = S_{DS} \text{ for } T_0 < T < T_S$$

$$S_a = \frac{S_{D1}}{T} \text{ for } T_S < T < T_L$$

$$S_a = \frac{S_{D1} * T_L}{T^2} \text{ for } T > T_L$$

The MCE_R is taken as 1.5 times the DRS at all periods. The map-based design and maximum considered earthquake response spectra for the project site are shown on Figure 4 and tabulated in Table 2.

6.0 EARTHQUAKE-INDUCED GEOLOGIC HAZARDS

Given the location of the JUHSD site in relation to the many active faults within and surrounding the San Francisco Bay Area, the proposed structures will be subject to very strong to violent ground shaking from a future large earthquake. The ground motion parameters provided in the Section 5.0 reflect those high levels of shaking. Therefore, there exists the potential for earthquake-related geologic hazards to impact the proposed JUHSD development. We evaluated the potential for ground surface fault rupture, earthquake-induced landsliding, liquefaction triggering and associated ground failures, and cyclic densification to impact the proposed structures.

6.1 Surface Fault Rupture

Earthquake-related ground surface fault rupture is generally associated with moderate magnitude (roughly M_W 6) and larger earthquakes and typically occurs along faults that have been recently active, at least within the geologic timeframe. The JUHSD site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act of 1972 State of California maps (California Geological Survey [CGS], 2000). No mapped faults cross the JUHSD site at the ground surface, or in the immediate vicinity of the JUHSD site (USGS and CGS, 2018; Field et al., 2013). Therefore, the potential for ground surface fault rupture and offset from a known active fault at the JUHSD site is considered to be very low.

6.2 Earthquake-Induced Landsliding

The potential for earthquake-induced landsliding is highest in areas with moderate to steep terrain that is underlain by unfavorably oriented geologic layering or discontinuities. As described previously, the ground surface at JUHSD facility is relatively level and developed, and the surrounding terrain is slightly southeast-sloping, with no substantial or steep natural slopes immediately adjacent to the proposed buildings. To the west of the of the site is a moderately-sloped hillside cut, about 60 to 70 feet high. The slope is supported by a retaining wall that is offset roughly 50 feet from the planned scissor-lift structures. Earthquake-Induced Landslide Hazard Zones are not included in the current regulatory map for the vicinity CGS (2000). No landslide areas or landslide deposits have been mapped in the vicinity of the JUHSD site (Brabb and Pampeyan, 1972; Brabb et al., 1998). The site and surrounding area are mapped in an area identified with low to moderate susceptibility to landsliding, characterized by several scattered small landslides generally associated with very steep slopes and unstable bedrock units (Brabb et al., 1978). Additionally, a map of debris-flow probability by (Mark, 1992) for San Mateo County indicates that the site is located in an area with a very low (less than 5%) probability for debris-flow failures. Based on the available information on bedrock type, landslide susceptibility, and lack of observed landslides in the site vicinity, the potential for seismically-induced landsliding to affect the proposed JUHSD buildings is considered to be low.

6.3 Liquefaction and Associated Effects

Liquefaction is a soil behavior phenomenon in which saturated cohesionless soil loses a substantial amount of strength due to excess pore-water pressures generated by strong earthquake ground shaking. The types of soils most susceptible to liquefaction include relatively clean (fines content less than 15 percent), loose, uniformly graded sands and gravels, silty sands and gravels, and non-plastic silt deposits. Recently-deposited (i.e., within about the past 11,000 years) soils, such as alluvial, fluvial, and aeolian deposits, and relatively unconsolidated soils and artificial fills located below the groundwater surface are considered susceptible to liquefaction (Youd and Perkins, 1978; Idriss and Boulanger, 2008).

Susceptibility of soils to liquefaction is generally evaluated based on in-situ conditions, soil index testing, and depth to groundwater table (indicating whether or not soils are saturated). The soils encountered in the upper 5 to 45 feet during the current investigation may be characterized as medium dense to dense and medium stiff to hard fills comprised of various compositions of sands, silts, and clays. In the presence of groundwater, a liquefaction susceptibility and triggering assessment is typically warranted for zones of medium dense, predominantly sandy undocumented fills; however, we judge that there are sufficient fines and interbedding of stiff clay zones in the fill profile at the JUHSD site and groundwater is sufficiently deep (greater than 50 feet) to conclude that the near-surface materials are generally not susceptible to significant strength loss resulting from earthquake-induced liquefaction, and the potential for liquefaction is considered to be very low. Consequently, ground settlement resulting from post-liquefaction reconsolidation following the design earthquake scenarios is judged to be very low.

6.4 Earthquake-Induced Compaction/Densification

Cyclic compaction/densification may occur when unsaturated soils contract in volume from strong ground shaking, resulting in vertical settlement of the ground surface or overlying improvements. Materials subject to cyclic compaction/densification typically include loosely deposited or placed, clean (low fines content), granular soils above the groundwater table (Tokimatsu and Seed, 1987). Based on the characterization of the near-surface soils (undocumented fills) as medium dense to dense and medium stiff to hard fills comprised of various compositions of sands, silts, and clays, we judge the potential for significant seismically-induced compaction/densification affecting the existing structures is judged to be very low.

7.0 GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

The results of the subsurface investigation for the JUHSD Faculty and Staff Housing site described previously were used to develop geotechnical recommendations for the design and construction of the proposed development. The recommendations presented in this report should be incorporated into the project plans and specifications and be implemented during construction. The main geotechnical/geologic issues affecting the site are strong ground shaking from large earthquake events and long-term differential settlement of structures built within areas of existing undocumented fill.

7.1 Foundation Recommendations

The subsurface investigation performed for this study suggests that the composition and thickness of undocumented fill varies considerably across the site (e.g. less than 5 feet in boring B-3 to 45 feet in boring B-1). In general, the undocumented fill may be considered to have adequate capacity to support the anticipated building loads. However, depending on foundation type and location, proposed buildings are expected to experience permanent settlements from the introduction of static loads. Settlements will be most pronounced at the transitions between foundations that bear directly on native soils at the eastern end of the proposed development to those supported on thicker fill zones to the west. Total and differential static settlements may be mitigated with ground improvement methods and/or consideration of foundation type. Should ground improvement methods be desired, we should be consulted to provide additional recommendations in support of those improvements.

7.1.1 Foundation Types

It is our understanding that all structures planned for the JUHSD development are to be constructed at-grade (no basements). As such, structural loads may be supported on shallow-type foundations including isolated spread footings, strip footings, slabs-on-grade, or a combination of all three. Based on our communications with the structural engineer, dead and live loads are not expected to exceed 2,500 psf on foundation elements. Considerations for each foundation type are discussed below.

7.1.1.1 Isolated Spread Footings

Individual column loads may be supported on reinforced concrete isolated spread footings bearing on undisturbed native soil or undocumented fill either unmodified or modified with ground improvement methods. Footings with a minimum width of 2 feet and bearing at 1 foot below ground surface on stiff native soils may be considered to have an ultimate bearing capacity, q_{ULT}, of 25 ksf. An increase of 5 ksf may be assumed for each additional foot of embedment. Footings with a minimum width of 2 feet and bearing at 1 foot below ground surface on undocumented fill that is unimproved may be considered to have a q_{ULT}, of 5 ksf. An increase of 3 ksf may be assumed for each additional foot of embedment. Bearing capacities of footings in fill may also be increased with certain ground improvement methods. Bearing capacities may be increased by one-third when considering transient loads (i.e., wind or seismic loads).

Anticipated total (static) settlements of spread footings will be a function of footing size, embedment depth, and bearing material. Footings bearing at 1 foot below the ground surface on stiff native soils are anticipated to experience total vertical settlements of less than ¼-inch with loads up to 2,500 psf. Footings with a width of 2 feet and bearing at 1 foot below ground surface on undocumented fill that is unimproved are anticipated to experience total vertical settlements in accordance with the following load schedule:

Applied Load (DL+LL) [psf]	Expected Vertical Settlement [inches]
500	< ¼
1000	¼ to ¾
1500	½ to 1
2000	¾ to 1½
2500	1 to 2

For similarly-sized and similarly-loaded footings bearing on unimproved fill, expected differential settlement is anticipated to be up to about ¾-inch over 50 feet. Differential settlements may be up to 1½-inch over the transition from footings bearing on stiff native soil to footings bearing on unimproved fill. These conditions are most likely to occur at the eastern end of the development particularly between structures founded at a lower grade elevation than the adjacent buildings to the west.

7.1.1.2 Strip Footings

Wall loads may be supported on reinforced concrete strip footings bearing on undisturbed native soil or undocumented fill either unmodified or modified with ground improvement methods. Strip footings with a minimum width of 1½ feet and bearing at 1 foot below ground surface on stiff native soils may be considered to have an ultimate bearing capacity, q_{ULT}, of 20 ksf. An increase of 5 ksf may be assumed for each additional foot of embedment. Strip footings with a minimum width of 1½ feet and bearing at 1 foot below ground surface on undocumented fill that is unimproved may be considered to have a q_{ULT}, of 4.5 ksf. An increase of 3 ksf may be assumed for each additional foot of embedment. Bearing capacities of strip footings in fill may also be increased with certain ground improvement methods. Bearing capacities may be increased by one-third when considering transient loads (i.e., wind or seismic loads).

Anticipated total (static) settlements of strip footings will be a function of footing width, embedment depth, and bearing material. Strip footings bearing at 1 foot below the ground surface on stiff native soils are anticipated to experience total vertical settlements of less than ¼-inch. Strip footings with a width of 1.5 feet and bearing at 1 foot below ground surface on undocumented fill that is unimproved are anticipated to experience total vertical settlements in accordance with the following load schedule:

Applied Load (DL+LL) [psf]	**Expected Vertical Settlement [inches]**
500	< ¼
1000	<¼ to ½
1500	¼ to ¾
2000	½ to 1
2500	¾ to 1½

For similarly-sized and similarly-loaded strip footings bearing on unimproved fill, expected differential settlement is anticipated to be up to about ½-inch over 50 feet. Differential settlements may be up to 1-inch over the transition from footings bearing on stiff native soil to footings bearing on unimproved fill.

7.1.1.3 Slabs-On-Grade

Slab-on-grade foundations may be designed using the subgrade modulus methodology. For slabs bearing on unimproved fill, we recommend using a coefficient of vertical subgrade reaction of 90 kips per cubic foot (kcf) under dead plus live loading conditions for the initial slab design. This value may be increased by 50 percent for total load conditions including wind and seismic. The coefficient of vertical subgrade has been reduced to account for the size of the slab and is therefore not "k_{v1}" for a one-foot-square plate. This value may be used within the entire slab footprint.

Slabs should be designed to impose a maximum dead-plus-live load bearing pressure of 2,500 psf on the subgrade soil. This pressure may be increased by one-third for total (wind and seismic) load conditions. We anticipate that the average bearing pressure will be significantly lower. Once the structural engineer estimates the distribution of bearing stresses on the bottom of the slabs, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

7.1.2 Lateral Loads and Base Friction

Lateral loads may be resisted by the combination of passive pressure on vertical faces of the foundation elements and the friction that develops between the base of the foundation elements and the subsurface soils. To compute the lateral resistance of the foundation elements, we recommend using an allowable passive pressure of 2,000 psf (uniform distribution) for transient loads and an allowable equivalent fluid weight of 400 pcf (triangular distribution) for sustained loads. These values assume that concrete footings are cast neat against the walls of excavated surfaces or are backfilled against formed walls and compacted in accordance with the recommendations of Section 7.4.2.5.

Lateral resistance due to base friction can be determined by using a coefficient of sliding resistance of 0.30 between the bottom of concrete foundations and the underlying soil. This value assumes that the foundation is in direct contact with the soil (no vapor barrier). Where a vapor retarder is placed below the foundation, a base friction coefficient of 0.20 should be used. The frictional resistance values include a factor of safety of at least 1.5 and can be used in combination with passive pressure without reduction.

7.1.3 Consideration for Expansive Soils

Potentially expansive near-surface soils are subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of at-grade building foundations, sidewalks, and pavements. Differential ground movement from swelling/shrinking can cause damage to concrete slabs and cause cracks in structural and non-structural building materials. In general, the effects of expansive soil can be mitigated by moisture conditioning the expansive soil, providing select, non-expansive fill or lime-treated soil below sidewalks and pavements.

Based on the results of the single plasticity test performed for this study and our visual assessment of the near surface-soils, we judge the expansive potential of the existing site soils is low.

7.2 Retaining Walls

Permanent retaining walls should be designed to resist static lateral earth pressures, lateral earth pressures caused by earthquakes, and surcharge pressures, where appropriate. Lateral earth pressures on retaining walls will vary with depth and soil type adjacent to the walls. In areas where walls are supporting undocumented fill, we assume a soil unit weight of 125 pcf and a soil friction angle of 34 degrees. Where permanent walls will be restrained from movement at the top or sides, they should be designed for at-rest conditions. Unrestrained walls (i.e., cantilever retaining walls) should be designed for active earth pressure conditions. We recommend restrained below-grade walls at the site be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 55 pcf
- Active earth pressure based on an equivalent fluid weight of 35 pcf plus a seismic increment
  - Seismic Increment, MCE_R: Uniform 58H (psf per foot of wall)
  - Seismic Increment, design: Uniform 38H (psf per foot of wall)

A triangular distribution should be used for the at-rest and active static earth pressures, and the seismic increment is represented as a uniform distribution added to the active pressure. The recommended lateral earth pressures are based on level backfill conditions with no surcharge loads. Where below-grade walls may be subjected to vehicular loading within ten feet of the wall, an additional uniform lateral pressure of 50 psf should be applied to the upper ten feet of the wall. These pressures are applicable to walls that are back drained to prevent build-up of hydrostatic pressures from surface water infiltration.

Although the retaining walls will be above the groundwater table, surface water from precipitation and/or irrigation may infiltrate into the soil behind the wall. As such, all walls should be adequately drained using a suitable drain rock or a prefabricated drainage panel against the back of wall. The rock or panel should extend to a perforated PVC collector pipe at the bottom of the wall that collects water and drains it to a suitable discharge point. The pipe should be surrounded by at least four inches of 3/4-inch clean, open-graded drain rock or Caltrans Class 2 permeable base material wrapped in filter fabric. Alternatively, a proprietary, prefabricated collector drain system designed to work in conjunction with the drainage panel may be used instead of the perforated pipe described above. The drain pipe should be connected to a suitable discharge point; if needed, a sump and pump system may be required to drain collector pipes if the elevation is insufficient to gravity drain the system.

If backfill is required behind below-grade walls, the walls should be braced, or hand compaction equipment should be used so that unacceptable surcharges on walls can be prevented as determined by the structural engineer.

7.3 Temporary Cut Slopes and Shoring

We understand that the structures for the proposed JUHSD development are to be constructed at-grade. Therefore, deep excavations are not anticipated. Excavations that are deeper than five feet and will be entered by workers should be slope cut or shored in accordance with Cal/OSHA standards. The contractor should be responsible for the construction and safety of all excavations and temporary slopes. We judge that temporary cut slopes in unimproved fill soil, corresponding to Cal/OSHA Type B soil will be temporarily stable at no steeper than a 1H:1V cut slope, provided they are not surcharged by equipment or other building materials.

7.4 Earthwork

This section describes recommendations for the earthwork necessary to prepare the site for construction of the foundations and surface improvements of the proposed development.

7.4.1 Site Preparation

Most of the area of the proposed development is currently occupied by surface parking lots. Any asphalt, base materials, concrete curbs, sidewalks, and underground utilities that exist within the footprint of the new structures should be completely removed. Undesirable materials, including any vegetation, roots, wood fragments, concrete or construction debris, underground pipes, and any other material that could interfere with the performance or completion of the work should also be removed. Tree roots with diameters greater than ½-inch within three feet of the final subgrade should be removed.

If pipelines or utility conduits are to be abandoned in place, they should be backfilled with slurry cement meeting the controlled low strength material (CLSM) requirements described in Section 7.4.2.4. Any excavations created by demolition operations and that are located below new foundations should be properly backfilled with compacted fill or slurry cement under the direction of the field engineer.

7.4.2 Fill Materials and Compaction Requirements

Fill placement at the site is expected to include general site fill, utility trench backfill, foundation subgrade materials, wall backfill, and drainage and landscape grading. All fill should be placed in accordance with the requirements discussed below.

7.4.2.1 Select Fill

Select fill material (imported fill) should consist of soil that is free from construction debris and organic matter, be free of rocks or lumps greater than three inches in greatest dimension, have a plasticity index less than 12 and a liquid limit less than 40, and be approved by the geotechnical engineer. Select fill should be placed in lifts less than eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and be compacted in accordance with the requirements described in Section 7.4.2.5. Samples of proposed select fill should be provided to the geotechnical engineer at least three business days prior to use at site.

The grading contractor should provide analytical test results or other suitable environmental documentation indicating imported fill is free of hazardous material at least three days before importing the fill to the site. If this data is not provided, a minimum of two weeks may be required to perform any necessary analytical testing prior to importing the fill.

7.4.2.2 General Fill

In general, soils derived from native site materials (including undisturbed native soils and areas of existing undocumented fill) may be used as general site fill, provided they meet the same requirements as “Select Fill” described in Section 7.4.2.1. General fill should be placed in lifts less than eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and be compacted in accordance with the requirements described in Section 7.4.2.5.

7.4.2.3 Class 2 Aggregate Base

¾-inch Class 2 Aggregate Base (AB) may be used as an import material for use in general site fill, pavement subgrades, or and/or foundation subgrades. Class 2 AB should meet the requirements of the 2018 Caltrans Standard Specifications, Chapter 26 (Caltrans, 2018a).

7.4.2.4 Controlled Low Strength Material

Controlled low strength material (CLSM) or slurry cement backfill may be considered as an alternative to fill below building foundations, sidewalks, or pavement. CLSM is a flowable slurry cement mixture that should meet the requirements in the 2018 Caltrans Standard Specifications, Chapter 19 (Caltrans, 2018a). CLSM is an ideal backfill material when there is inadequate room for conventional compaction equipment or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined compressive strength of 50 pounds per square inch (psi).

7.4.2.5 Compaction Requirements

Depending on the fill material used, compaction effort moisture-conditioning will vary. We recommend fill be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, be moisture-conditioned, and compacted relative to ASTM D1557 in accordance with the following:

Fill Type/Location	Required Relative Compaction [%]	Moisture Requirement
Site Fill (Select/General Fill)	≥ 90	Above Optimum
Utility Trench Backfill (Select/General Fill)	≥ 90	Above Optimum
Utility Trench Backfill (Clean Sand or Gravel)	≥ 95	Near Optimum
Pavement Subgrade (Select/General Fill)	≥ 95	Above Optimum
Pavement Subgrade (Aggregate Base)	≥ 95	Near Optimum
Foundation Subgrade (Select/General Fill)	≥ 95	Above Optimum
Foundation Subgrade (Aggregate Base)	≥ 95	Near Optimum

It should be noted that "moisture-conditioning" refers to either moistening or drying of the soil to achieve the desired moisture content.

7.4.3 Subgrade Preparation

Grading of foundation/building subgrades should be performed under the observation of our field engineer. Should zones of weak or loose soil extending deeper than eight inches be encountered during grading, the material should be over-excavated down to firm material, as determined by our field engineer, and replaced with engineered Select Fill, Class 2 AB, or CLSM, as outlined in Section 7.4.2.5, above. Foundation/building subgrades should be kept moist (but not saturated) until covered by a vapor retarder or concrete.

Driveways, sidewalks, patios, and any other concrete slabs may consist of conventional concrete slabs-on-grade. In addition, Ancillary structures (such as the vehicular parking scissor lifts) are planned to be constructed at-grade. Building pads for Ancillary structures and subgrade for conventional concrete slabs-on-grade should be prepared by scarifying the upper 12 inches of soil, moisture-conditioning, and recompacting in accordance with the recommendations in Section 7.4.2.5 prior to placement new fill or improvements. General grading should be performed under the observation of the field engineer. Should zones of weak or loose soil extending deeper than eight inches be encountered during grading, the material should be over-excavated down to firm material, as determined by our field engineer, and replaced with engineered Select Fill, Class 2 AB, or CLSM, as outlined in Section 7.4.2.5, above.

7.4.4 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe; all temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor. Temporary excavations should comply with all state and/or federal requirements, including those of Cal/OSHA.

Pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel to provide uniform support. After testing and inspection (if required) of the pipes and conduits, they should be covered to a depth of six inches with clean sand or fine gravel, which should be mechanically tamped. Because backfill for utility trenches and other excavations is considered fill, it should be placed and compacted in accordance with the recommendations presented in Section 7.4.2.5, above.

Jetting of trench backfill should not be permitted and special care should be taken when backfilling utility trenches in paved areas. Poor compaction can cause excessive settlements, resulting in damage to improvements above the fill and uneven surfaces.

7.4.5 Drainage and Landscaping

Proper site drainage is important for the long-term performance of the proposed buildings and site improvements. Positive surface drainage should be provided around the buildings to direct surface water away from foundations. To reduce the potential for ponding adjacent to the buildings, we recommend the ground surface within five feet of the building perimeters slope down, away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. Roof downspouts should be discharged away from the foundations into controlled drainage facilities.

7.5 Flexible (Asphalt Concrete) Pavement Design

We understand that the proposed development will include construction of flexible asphalt concrete (AC) pavement sections for parking areas. The California Department of Transportation (Caltrans) methodology for the design of AC pavement sections was used to develop recommendations for the proposed AC pavement sections at the site (Caltrans, 2017). A design R-value of 35 was used for anticipated subgrade soils based on an R-value test performed on a bulk sample collected in the upper 5 feet of the undocumented site fill. The R-value for aggregate base is assumed as 78, which is the minimum required value for Caltrans Class 2 Aggregate Base (Caltrans, 2018a).

A typical pavement section includes asphaltic concrete and aggregate base overlying the existing soil subgrade. The subgrade for pavement should be prepared consistent with the recommendations for subgrade preparation in Section 7.4.3. The table below presents our AC pavement section recommendations for traffic indices (TIs) of 4.5 through 6.0 based on the assumptions described above. The project Civil Engineer should confirm that the TIs presented in this report are appropriate for the intended pavement use depending on the amount of anticipated loading and traffic. We can provide additional pavement sections for different TIs, if requested.

Recommended Asphalt Pavement Sections

Traffic Index	Pavement Component Thickness (inches)	
	Asphalt Concrete	Caltrans Class 2 Aggregate Base R-Value=78
4.5	2½	5
5.0	2¾	6
5.5	3	7
6.0	3¼	7

The subgrade should be proof-rolled under the observation of our field engineer to confirm that it is non-yielding prior to placement of aggregate base. The aggregate base should be moisture-conditioned and compacted to the requirements presented in Section 7.4.2.5. The aggregate base should also be proof-rolled under the observation of our field engineer to confirm that it is non-yielding prior to paving.

If pavements are adjacent to irrigated landscaped areas, curbs adjacent to those areas should extend through the aggregate base and into the underlying soil by a minimum of three inches to reduce the potential for irrigation water to infiltrate the pavement section and subsurface soils below the pavement.

7.6 Soil Corrosivity

Corrosivity analyses of the near-surface undocumented fill soils in Borings B-2 and B-5 were performed by Cooper Testing Laboratory, Inc. The corrosivity test included soil resistivity, pH, and sulfate and chloride contents. The results of the corrosivity tests are included in Appendix B.

Caltrans defines a “corrosive environment” in terms of the resistivity, pH, and soluble salt content of the soil (Caltrans, 2018b). Areas may be considered corrosive if measured resistivity is less than 1100 ohm-cm. Both samples were tested with resistivities less than 800 ohm-cm which indicate the presence of high quantities of soluble salts and higher propensity for corrosion. For structural elements a site is considered corrosive if one or more of the following conditions exist for the representative soil samples:

- Chloride concentration ≥ 500 ppm;
- Sulfate concentration ≥ 1500 ppm; or
- pH ≤ 5.5.

Both samples tested with sulfate concentrations greater than 3000 ppm, indicating a “corrosive” environment for foundations per Caltrans (2018b). Accordingly, all buried metallic pipes or foundation elements including, but not limited to iron, steel, cast iron, ductile iron, galvanized steel, and buried concrete should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with the soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Slate should have the opportunity to review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, excavation, grading, fill placement and compaction, and foundation installation. These observations will allow us to compare actual with anticipated soil conditions and to confirm that the contractor’s work conforms with the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

The evaluations made during this study and recommendations presented in this report are based on the assumption that the soil and groundwater conditions across the project site do not deviate appreciably from those described herein, and have been disclosed in the subsurface exploration performed. If any variations or undesirable conditions are encountered during construction, we should be informed to provide additional or supplemental recommendations, if deemed necessary. The information provided in this report was prepared for the proposed development described in this report, specifically for use by Jefferson Union High School District, its agents, and the project design team. Significant changes in location, type, or embedment of the structure, or loading conditions should be evaluated as to their effects on the enclosed information. The recommendations presented in this report are not valid for other locations and construction in the project vicinity.

In the performance of our professional services, Slate Geotechnical Consultants Inc., its employees, and its agents comply with the standards of care and skill commonly used as state-of-practice in our profession practicing in the same or similar localities. We are responsible for the evaluations contained in this report; however, in the event that conclusions based on the data and information provided herein are made by others, such conclusions are not our responsibility unless we have been given an opportunity to review and concur in writing with such conclusions.

10.0 REFERENCES

American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), 2010, ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures; Reston, VA.

ASTM International (2012) Standard D1557 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)).

ASTM International (2017) Standard D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedures).

Brabb, E.E., and Pampeyan, E.H., 1972, Preliminary Map of Landslide Deposits in San Mateo County, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-344, scale 1:62,500.

Brabb, E.E., Pampeyan, E.H., and Bonilla, M.G., 1978, Landslide susceptibility in San Mateo County, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-360, scale 1:62,500.

Brabb, E.E., Graymer, R.W., and Jones, D.L., 1998, Geology of the onshore part of San Mateo County, California: A digital database: U.S. Geological Survey Open-File Report 98-137, https://pubs.usgs.gov/of/1998/of98-137/.

California Building Standards Commission (CBSC), 2016, 2016 California Building Code, California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

California Geological Survey (CGS), 2000, Earthquake Zones of Required Investigation, San Francisco South Quadrangle: CGS Earthquake Fault Zones and Seismic Hazards Zones Maps, scale 1:24,000.

Caltrans, 2018a, Standard Specifications, State of California, California State Transportation Agency, Department of Transportation.

Caltrans, 2018b, Corrosion Guidelines Version 3.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Branch, March.

Caltrans, 2017, Highway Design Manual, Chapter 630, Flexible Pavement.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D, Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parson, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, J.J. II, and Zeng, Y., 2013, “Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time-Independent Model,” U.S. Geological Survey Open-File Report 2013-1165.

Idriss, I.M., and Boulanger, R.W., 2008, Soil liquefaction during earthquakes, Monograph MNO-12: Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Mark, R.K., 1992, Map of Debris-Flow Probability, San Mateo County, California: U.S. Geological Survey Miscellaneous Investigations Series Map I-1257-M, scale 1:62,500.

Tinsley, J.C., III, Egan, J.A., Kayen, R.E., Bennett, M.J., Kropp, A., and Holzer, T.L., 1998, Appendix: Maps and descriptions of liquefaction and associated effects: in Holzer, T.L., ed., The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction: U.S. Geological Survey Professional Paper 1551-B.

Tokimatsu, K., and Seed H.B., 1987, Evaluation of settlements in sand due to earthquake shaking, Journal of Geotechnical Engineering, v. 113, no. 8.

United State Geological Survey (USGS), 1995, Historical Topographic Map, South San Francisco, CA Quad, 1:24,000 scale.

U.S. Geological Survey (USGS) and California Geological Survey (CGS), 2018, Quaternary Fault and Fold Database for the United States, accessed December 12, 2018, from USGS website: https://earthquake.usgs.gov/hazards/qfaults/.

Woodward-Clyde Consultants, 1985, Preliminary Geotechnical Evaluation Serramonte Del Ray, Daly City, California, letter report prepared for First Southwest, Project No. 16598A, June 28.

Working Group on California Earthquake Probabilities (WGCEP), 2008, The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437 and California Geological Survey Special Report 203, http://pubs.usgs.gov/of/2007/1437/.

Youd, T.L., and Hoose, S.N., 1978, Historical ground failures in Northern California triggered by earthquakes: U.S. Geological Survey Professional Paper 993.

Youd, T.L., and Perkins, D.M., 1978, Mapping of liquefaction induced ground failure potential: Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, v. 104, no. 4, p. 433-446.

TABLES

Table 1

LABORATORY TESTING PROGRAM RESULTS
Geotechnical Investigation Report - Jefferson Union High School District Faculty and Staff Housing
Daly City, California

Boring No.	Sample No.	Sample Depth [ft]	USCS Class.	Dry Unit Weight [pcf]	Moisture Content [%]	Grain-Size Distribution			Atterberg Limits			Other
						% Fines	% Sand	%Gravel	LL	PL	PI	
B-1	5	11-12.5	SC			35	65	0				
B-1	8	23.5-25	CL			73	27	0				
B-2	1-2	3.5-4	CL	107.8	16.2				28	22	6	
B-2	2-2	7.5-8	CL		21.2							Corr.
B-2	7-1	20.5-21	SC			22	78	0				
B-3	4	8.5-10	SC			29	71	0				
B-4	Bulk	0-5	SC									R-Value=37
B-4	3-2	7-7.5	CL			51	49	0				
B-4	4	10-11.5	SC			47	53	0				
B-5	2-2	4.5-5	SC		16.4							Corr.
B-5	7	23.5-24	SC			26	74	0				

Table 2

CODE-BASED MAP-BASED DESIGN AND MCE_R LEVEL RESPONSE SPECTRA ORDINATES
Geotechnical Investigation Report - Jefferson Union High School District Faculty and Staff Housing
Daly City, California

T (sec)	Spectral Acceleration, S_A (g)	
	Map-Based Design Reponse Spectrum	Map-Based MCE_R Spectrum
0.01	0.768	1.153
0.02	0.841	1.261
0.03	0.913	1.370
0.05	1.058	1.588
0.075	1.240	1.859
0.1	1.421	2.131
0.15	1.740	2.610
0.2	1.740	2.610
0.25	1.740	2.610
0.3	1.740	2.610
0.4	1.740	2.610
0.5	1.740	2.610
0.75	1.671	2.506
1	1.253	1.880
1.5	0.835	1.253
2	0.627	0.940
3	0.418	0.627
4	0.313	0.470
5	0.251	0.376
7.5	0.167	0.251
10	0.125	0.188

FIGURES

Project No: 18-007.00	Date: 2/5/2019	Created By: CBJ	Checked By: CBJ	Figure No. 1

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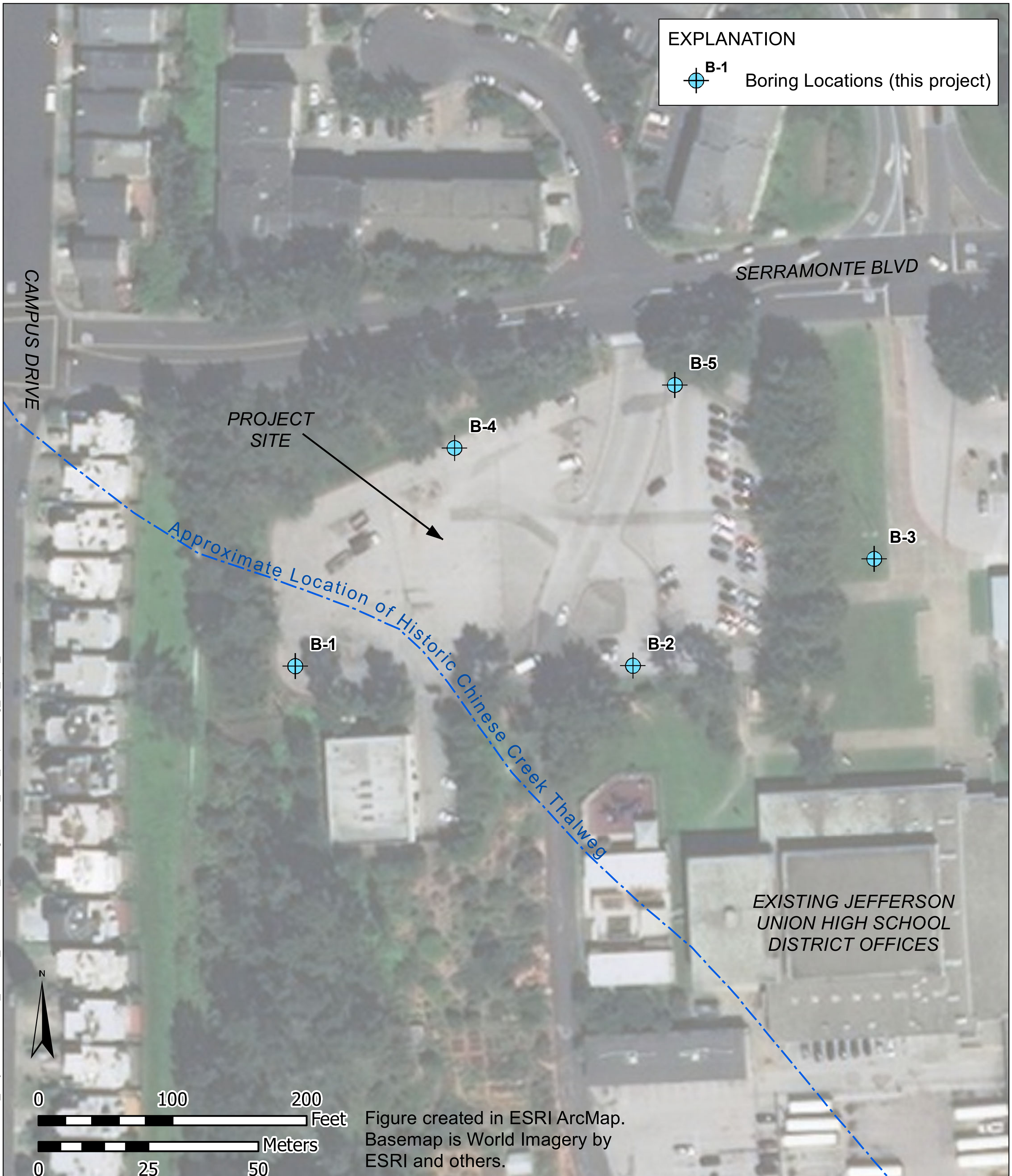
SITE LOCATION MAP

Geotechnical Investigation Report
Jefferson Union High School District Faculty and Staff Housing
Daly City, California

Filepath: SlateGeotech\SlateDrive - Documents_Projects\18-007.00_JUHSD_Geotechnical_Investigation\07_GIS_Graphics\Fig_01_Site_Location_Map.mxd



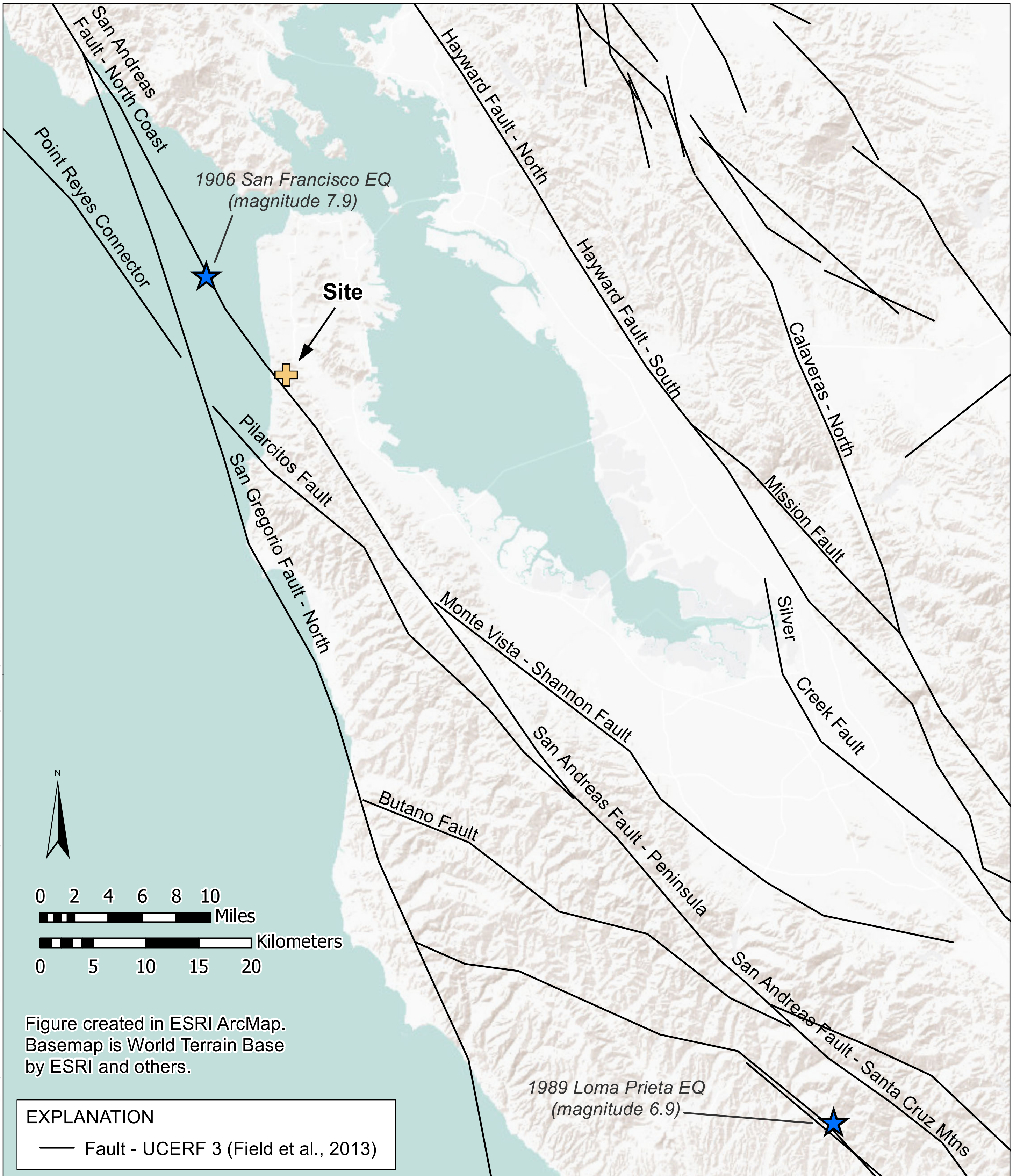
Project No: 18-007.00	Date: 3/19/2019	Created By: CBJ	Checked By: CBJ	Figure No. **2**

SLATE GEOTECHNICAL CONSULTANTS

SITE PLAN AND BORING LOCATIONS

Geotechnical Investigation Report
Jefferson Union High School District Faculty and Staff Housing
Daly City, California

Filepath: SlateGeotech\SlateDrive - Documents_Projects\18-007.00_JUHSD_Geotechnical_Investigation\07_GIS_Graphics\Fig_02_Site_Plan.mxd



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Project No: 18-007.00	Date: 3/19/2019	Created By: CBJ	Checked By: --	Figure No. **3**

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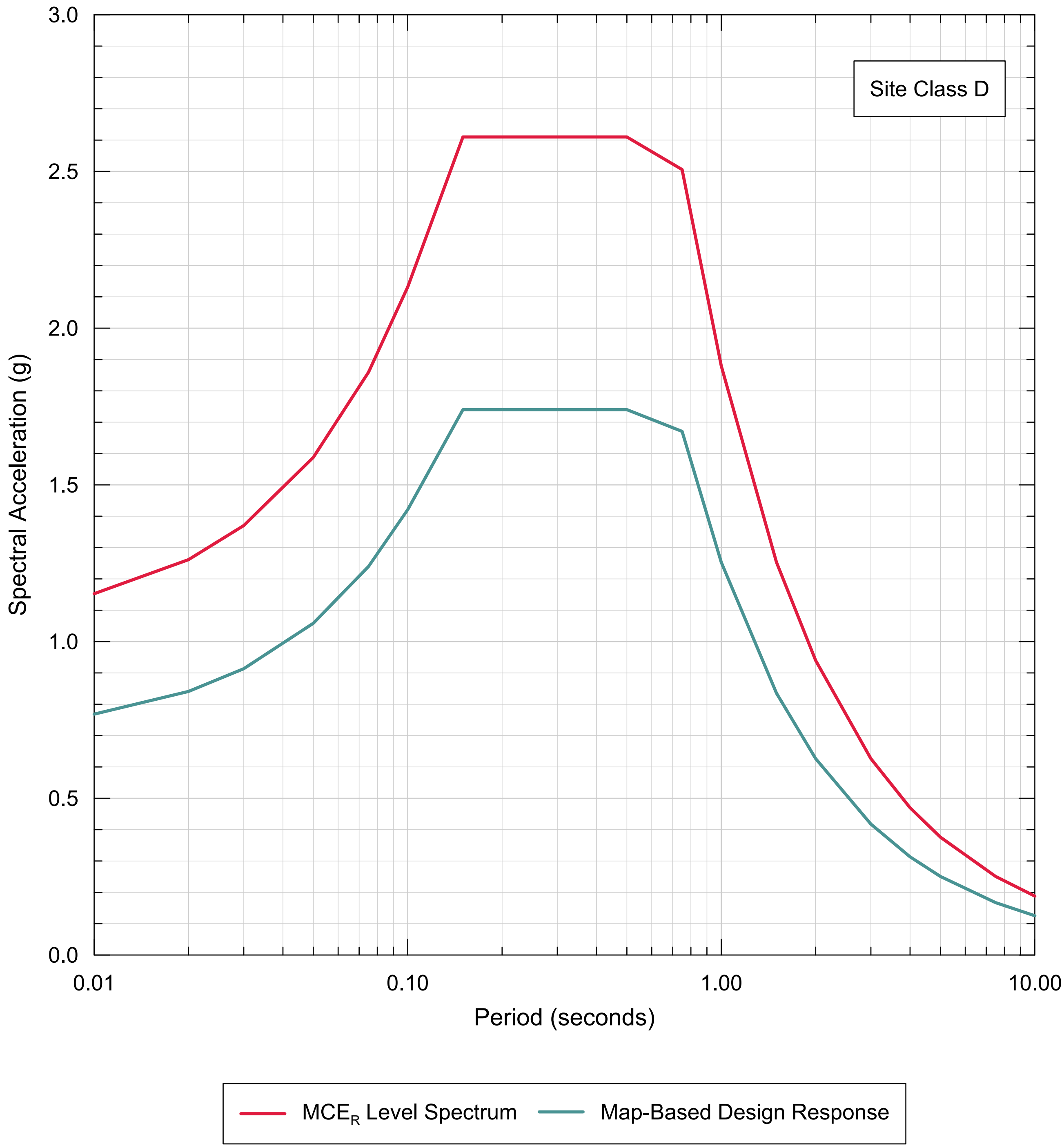
REGIONAL FAULT MAP

Geotechnical Investigation Report
Jefferson Union High School District Faculty and Staff Housing
Daly City, California



Note:
Spectra are damped 5%, except PGA

Project No: 18-007.00	Date: 3/18/19	Created By: HMC	Checked By: JDP	Figure No: **4**

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CODE-BASED MAP-BASED DESIGN AND MCE_R LEVEL RESPONSE SPECTRA

Geotechnical Investigation Report
Jefferson Union High School District Faculty and Staff Housing
Daly City, California

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APPENDIX A

BORING LOGS

UNITED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS				SYMBOL	GROUP NAME
COARSE-GRAINED SOILS	MORE THAN 50% RETAINED ON THE NO. 200 SIEVE	GRAVELS (MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE)	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	WELL-GRADED GRAVEL OR GRAVEL-SAND MIXTURES
				GP	POORLY-GRADED GRAVEL OR GRAVEL-SAND MIXTURES
			GRAVELS WITH FINES (MORE THAN 12% FINES)	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURES
		SANDS (50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE)	CLEAN SANDS (LESS THAN 5% FINES)	SW	WELL-GRADED SAND OR GRAVELLY SAND
				SP	POORLY-GRADED SAND OR GRAVELLY SAND
			SANDS WITH FINES (MORE THAN 12% FINES)	SM	SILTY SAND, SAND-SILT MIXTURES
				SC	CLAYEY SAND, SAND-CLAY MIXTURES
FINE-GRAINED SOILS	50% OR MORE PASSES THE NO. 200 SIEVE	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)	INORGANIC	ML	SILT, CLAYEY SILT, SANDY GRAVELLY SILT, SANDY SILT
				CL	LEAN CLAY, GRAVELLY LEAN CLAY, SANDY LEAN CLAY
			ORGANIC	OL	ORGANIC SILT AND ORGANIC CLAY
		SILTS AND CLAYS (LIQUID LIMIT 50 OR MORE)	INORGANIC	MH	ELASTIC SILT
				CH	FAT CLAY, GRAVELLY FAT CLAY, SANDY FAT CLAY
			ORGANIC	OH	ORGANIC SILT AND ORGANIC CLAY
HIGHLY ORGANIC SOILS				PT	PEAT

SOIL GRAIN SIZE

CLASSIFICATION	RANGE OF GRAIN SIZES: US STANDARD SIEVE SIZE	RANGE OF GRAIN SIZES: GRAIN SIZE IN MILLIMETERS
BOULDERS	ABOVE 12”	ABOVE 305
COBBLES	12” TO 3”	305 TO 76.2
GRAVEL	3” TO NO. 4	76.2 TO 4.76
COARSE	3” TO 3/4"	76.2 TO 19.1
FINE	3/4" TO NO. 4	19.1 TO 4.76
SAND	NO. 4 TO NO. 200	4.76 TO 0.075
COARSE	NO. 4 TO NO. 10	4.76 TO 2.00
MEDIUM	NO. 10 TO NO. 40	2.00 TO 0.420
FINE	NO. 40 TO NO. 200	0.420 TO 0.075
SILT AND CLAY	BELOW NO. 200	BELOW 0.075

CONSISTENCY OF FINE-GRAINED SOILS

BLOWS/FOOT: SPT	BLOWS/FOOT: MOD-CAL	DESCRIPTION
0-2	0-3	VERY SOFT
2-4	3-6	SOFT
4-8	6-12	MEDIUM STIFF
8-15	12-23	STIFF
15-30	23-46	VERY STIFF
>30	>46	HARD

MOISTURE CONDITIONS

DRY – DRY TO THE TOUCH
MOIST – DAMP, BUT NO VISIBLE FREE WATER
WET – VISIBLE FREE WATER

STABILIZED GROUNDWATER LEVEL

GROUNDWATER AT TIME OF DRILLING (UNSTABALIZED)

INFERRED LAYER TRANSITION

DENSITY OF COARSE-GRAINED SOILS

BLOWS/FOOT: SPT	BLOWS/FOOT: MOD-CAL	DESCRIPTION
0-4	0-6	VERY LOOSE
5-10	6-15	LOOSE
11-30	15-46	MEDIUM DENSE
31-50	46-77	DENSE
>50	>77	VERY DENSE

SAMPLER TYPE

SPT – STANDARD PENETRATION TEST
SPLIT-BARREL SAMPLER WITH 2.0-INCH
OUTSIDE DIAMETER AND 1.5-INCH INSIDE DIAMETER

MCA – MODIFIED CALIFORNIA (MOD-CAL)
SPLIT-BARREL SAMPLER WITH 3.0-INCH
OUTSIDE DIAMETER AND 2.5-INCH INSIDE DIAMETER

SAMPLE DRIVE LENGTH } SAMPLE RECOVERY

LABORATORY TESTS

CORROSIVITY	CORROSIVITY TESTING
PI	PLASTICITY INDEX
LL	LIQUID LIMIT
G_S	SPECIFIC GRAVITY
TX-UU	UNCOLSOLIDATED-UNDRAINED TRIAXIAL TEST

Project No: 18-007.00	Date: 3/14/2019	Created By: HMC	Checked By: JDP	Figure No: **A-0**

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BORING LOG EXPLANATION

Geotechnical Investigation Report
Jefferson Union High School District Faculty and Staff Housing
Daly City, California

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 1/17/19 Date finished: 1/17/19
Drilling method: Hollow Stem Auger 4 inch ID, 7 inch OD
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), Modified California (MC)

Logged by: H. Curran
Drilled by: Gregg Drilling & Testing
Rig: Mobile B-61

DEPTH (feet)	Sampler Type	Blows/ 6"	Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4 inches of asphalt						
					2 inches of aggregate base						
1–2	MC	10 11 11	22	SC	CLAYEY SAND (SC) Dark grayish brown (2.5Y 4/2) with yellowish brown (10YR 5/6) mottling, medium dense, moist, fine sand						
3–4	MC	9 13 14	27	CL	SANDY LEAN CLAY (CL) Very dark greenish gray (Gley 1 3/1), stiff, moist, fine sand						
5–7	MC	7 24 25	49	SC	CLAYEY LEAN SAND (SC) Very dark greenish gray (Gley 1 3/1), medium dense, moist, fine sand Becomes dense						
8–9	SPT	10 14 15	29	CL	SANDY LEAN CLAY (CL) Black (10YR 2/1), very stiff, moist, fine sand, low plasticity						
10–12	SPT	10 12 13	25	SC	CLAYEY SAND (SC) Yellowish brown (10YR 5/6), medium dense, moist, fine sand Olive brown (2.5Y 4/3), black and yellowish brown mottling (10YR 5/8), with clay lenses				35		
13–16	SPT	6 18 17	35	CL	SANDY LEAN CLAY (CL) Interbedded dark greenish gray (Gley 1 4/1), and yellowish brown (10YR 5/8), hard, moist, fine sand						
17–21	SPT	12 10 19	29	SC	CLAYEY SAND (SC) Yellowish brown (10YR 5/8) with olive brown (2.5Y 4/3) mottling and black (Gley 2.5/N), lenses, medium dense, moist, fine sand						
22–25	SPT	11 15 13	28	CL	LEAN CLAY with SAND (CL) Stratified yellowish brown (10YR 5/6) and dark greenish gray (Gley 1 3/1), very stiff, moist, fine sand				73		
28–31	SPT	8 9 13	22	CL	Mottled grayish brown (2.5Y 5/2) and strong brown (7.5YR 5/8)						

FILL? (0–31 feet)

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007
Figure: A-1a

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CL	SANDY LEAN CLAY (CL) (continued)						
32												
33												
34	SPT		7 9 14	23								
35												
36												
37												
38						FILL?						
39	SPT		12 20 24	44		Becomes hard						
40												
41												
42												
43												
44	SPT		18 30 34	64		Yellowish brown (10YR 5/6) with trace greenish brown (2.5Y 5/2) mottling						
45												
46												
47					SC	CLAYEY SAND (SC) Yellowish brown (10YR 5/6), very dense, moist, fine sand						
48												
49	SPT		19 38 50/5"	88/ 11"								
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007

Figure: A-1b

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2
Date started: 1/17/19 Date finished: 1/17/19
Drilling method: Hollow Stem Auger 4 inch ID, 7 inch OD
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), Modified California (MC)

Logged by: H. Curran
Drilled by: Gregg Drilling & Testing
Rig: Mobile B-61

DEPTH (feet)	Sampler Type	Blows/ 6"	Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2.5 inches of asphalt						
0–2.8				CL	LEAN CLAY with SAND (CL) Olive brown (2.5Y 4/4), moist, fine sand, few angular gravel up to 1 inch diameter						
3	MC	10 21 23	44	CL-ML	SANDY LEAN SILTY CLAY (CL-ML) Olive brown (2.5Y 4/4), hard, moist, fine sand, low plasticity	28	22	6		16.2	108
6.5	MC	6 9 15	29		Becomes dark grayish brown (2.5Y 4/2), very stiff; Corrosivity Test						
8	MC	7 10 19	29	SC	CLAYEY SAND (SC) Dark yellowish brown (10YR 5/6), medium dense, moist, fine sand						
10.5	MC	8 14 18	32	CL	SANDY LEAN CLAY (CL) Dark grayish brown (2.5Y 4/2), very stiff, moist, fine sand, low plasticity						
12				CL	LEAN CLAY with SAND (CL) Very dark gray (2.5Y 3/1) to black (2.5Y 2.5/1), stiff, moist, fine sand, medium plasticity, trace rootlets						
13	MC	5 8 14	22								
13.5				SC	CLAYEY SAND (SC) Yellow brown (10YR 5/6) mottled with brown (10YR 4/3), medium dense, moist, fine sand						
15.5	MC	9 10 18	28								
0–20					FILL?						
20.5	MC	29 50/4"	50/4"	SC	CLAYEY SAND (SC) Yellowish brown (10Y 5/8), very dense, moist, fine sand, poorly graded sand				22		
25.5	SPT	39 50/3"	50/3"								
30	SPT	50/5"	50/5"								

Boring terminated at a depth of 30.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007 Figure: A-2

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2
Date started: 1/18/19 | Date finished: 1/18/19
Drilling method: Hollow Stem Auger 4 inch ID, 7 inch OD
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), Modified California (MC)

Logged by: H. Curran
Drilled by: Gregg Drilling & Testing
Rig: Mobile B-61

DEPTH (feet)	Sampler Type	Blows/ 6"	Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1.75				CL	SANDY CLAY (CL) Very dark brown (10YR 2/2), moist, fine sand, abundant rootlets (HAND AUGER 0–4.5 ft; FILL?)						
1.75–4.5	BULK (2–3)			SC	CLAYEY SAND (SC) Light olive (2.5Y 5/4), moist, fine sand						
4.5–5.5	MC (5)	18 50/5"	50/5"	CL	SANDY LEAN CLAY (CL) Very dark brown (10YR 2/2), hard, moist, fine sand						
5.5–27	SPT (6)	25 50/5"	50/5"	SC	CLAYEY SAND (SC) Light olive brown (2.5Y 5/4) mottled with gray (10Y 4/1), very dense, fine sand. Becomes dark greenish gray (Gley 1 4/1) at 6 feet. Trace medium sand at 6 to 7 feet						
9	SPT	23 44 50/3"	94/5"	SC					29		
14	SPT	28 50/3"	50/3"	SC	Trace shell fragments						
19	SPT	17 50/5"	50/5"	SC	Few medium sand						
24	SPT	19 50/5"	50/5"	SC	Few fine gravel, decomposed and fractured						
27–29.5	SPT (29)	25 50/5"	50/5"	ML	LEAN SILT with SAND (ML) Dark greenish gray (Gley 1 4/1), hard, moist, fine sand, trace shell fragments						

Boring terminated at a depth of 29.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007 | Figure: A-3

SLATE 18-007.GPJ TR.GDT 3/14/19

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2

Date started: 1/18/19 | Date finished: 1/18/19

Drilling method: Hollow Stem Auger 4 inch ID, 7 inch OD

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Standard Penetration Test (SPT), Modified California (MC)

Logged by: H. Curran
Drilled by: Gregg Drilling & Testing
Rig: Mobile B-61

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						3 inches of asphalt						
1–2.5	MC		6 7 7	14	SC	CLAYEY SAND (SC) Dark yellowish brown (10YR 4/6) with light olive brown (2.5Y 5/3) mottling, loose, moist, trace subangular fine gravel, trace rootlets						
6–7.5	MC		9 9 11	20		Coarse gravel at 6 feet				51		
7					CL	SANDY LEAN CLAY (CL) Dark yellowish brown (10YR 4/6) with black (10YR 2/11) and strong brown (7.5YR 5/8) mottling, stiff, no fine gravel						
9–10.5	MC		4 4 12	16	SC	CLAYEY SAND (SC) Interbedded very dark greenish gray (Gley 1 3/1) to black (Gley 12.5/N) and dark yellowish brown (10YR 5/6) with strong brown (2.5Y 4/3) and olive brown (7.5YR 5/8) mottling, medium dense to dense, moist, low plasticity fines						
10–11.5	SPT		9 11 13	24						47		
14–15.5	SPT		10 13 14	27		2 inch clay seam, medium plasticity fines						
18–19.5	SPT		8 10 16	26								
24–25.5	SPT		9 14 16	30		Yellowish brown (10YR 5/8) with strong brown (7.5YR 5/6) mottling, medium dense						
29–30.5	SPT		7 12 14	26								

FILL? (0 to 30 feet)

Boring terminated at a depth of 30 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007 | Figure: A-4

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2
Date started: 1/17/19 | Date finished: 1/17/19
Drilling method: Hollow Stem Auger 4 inch ID, 7 inch OD
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), Modified California (MC)

Logged by: H. Curran
Drilled by: Gregg Drilling & Testing
Rig: Mobile B-61

DEPTH (feet)	Sampler Type	Blows/ 6"	Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches of asphalt						
1–2	MC	8, 11, 9	20	SC	CLAYEY SAND (SC) Yellowish brown (10YR 5/6) with light gray (10YR 7/1) inclusions, medium dense, moist, fine sand						
4	MC	6, 7, 12	19		Trace subangular fine gravel; Corrosivity Test					16.4	
6–7	MC	11, 11, 12	23		No fine gravel, trace subrounded to angular						
9	SPT	8, 11, 15	26		Very dark greenish gray (Gley 1 3/1), no medium sand, increase fine content						
0–12					FILL?						
12				CL	SANDY LEAN CLAY (CL) Mottled gray (10YR 6/1) and yellowish brown (10YR 5/6), hard, moist, fine sand						
14	SPT	13, 38, 50/5"	88/ 11"								
17				SC	CLAYEY SAND (SC) Yellowish brown (10YR 5/6) mottled with gray (10YR 6/1), very dense, moist, fine sand, few to little medium sand						
19	SPT	15, 49, 37	86								
23–24	SPT	36, 50/3"	50/3"		Dark yellowish brown (10YR 4/6) with white lenses				26		
29	SPT262	22, 50/4"	50/4"								
30					No white lenses						

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007
Figure: A-5a

PROJECT: **JUHSD FACULTY AND STAFF HOUSING**
Daly City, California

Log of Boring B-5

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SC	CLAYEY SAND (SC) (continued)						
32												
33					CL	LEAN CLAY with SAND (CL) Yellowish brown (10YR 5/6), hard, moist, fine sand, low plasticity						
34	SPT		13 16 26	42								
35												
36												
37												
38												
39	SPT		15 35 40	85		Dark yellowish brown (10YR 4/4) with strong brown (7.5YR 4/6) mottling with white lenses						
40												
41												
42												
43												
44	SPT		16 42 50/5"			Dark yellowish brown (10YR 4/4) with reddish brown (5YR 4/4), lenses						
45												
46												
47												
48												
49	SPT		36 50/5"	50/5"								
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												

Boring terminated at a depth of 49.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007

Figure: A-5b

SLATE 18-007.GPJ TR.GDT 3/14/19

APPENDIX B

LABORATORY TEST RESULTS

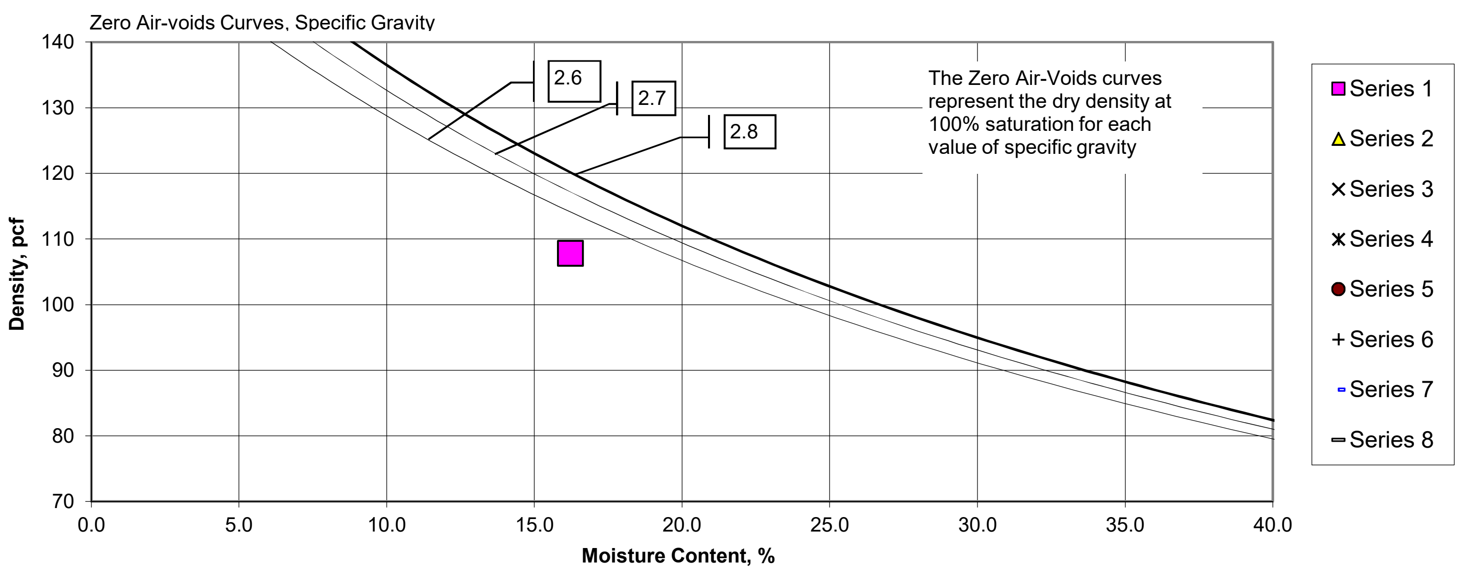
Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	1072-002	**Project No.**	18-007	**By:**	RU
Client:	Slate Geotechnical Consultants	**Date:**	01/29/19		
Project Name:	JUHSD	**Remarks:**			

Boring:	B-2							
Sample:	1-2							
Depth, ft:	3.4-4							
Visual Description:								
Actual G_s								
Assumed G_s	2.70							
Moisture, %	16.2							
Wet Unit wt, pcf	125.3							
Dry Unit wt, pcf	107.8							
Dry Bulk Dens.ρb, (g/cc)	1.73							
Saturation, %	77.6							
Total Porosity, %	36.1							
Volumetric Water Cont,Θw,%	28.0							
Volumetric Air Cont., Θa,%	8.1							
Void Ratio	0.56							
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

Moisture-Density



#200 Sieve Wash Analysis
ASTM D 1140

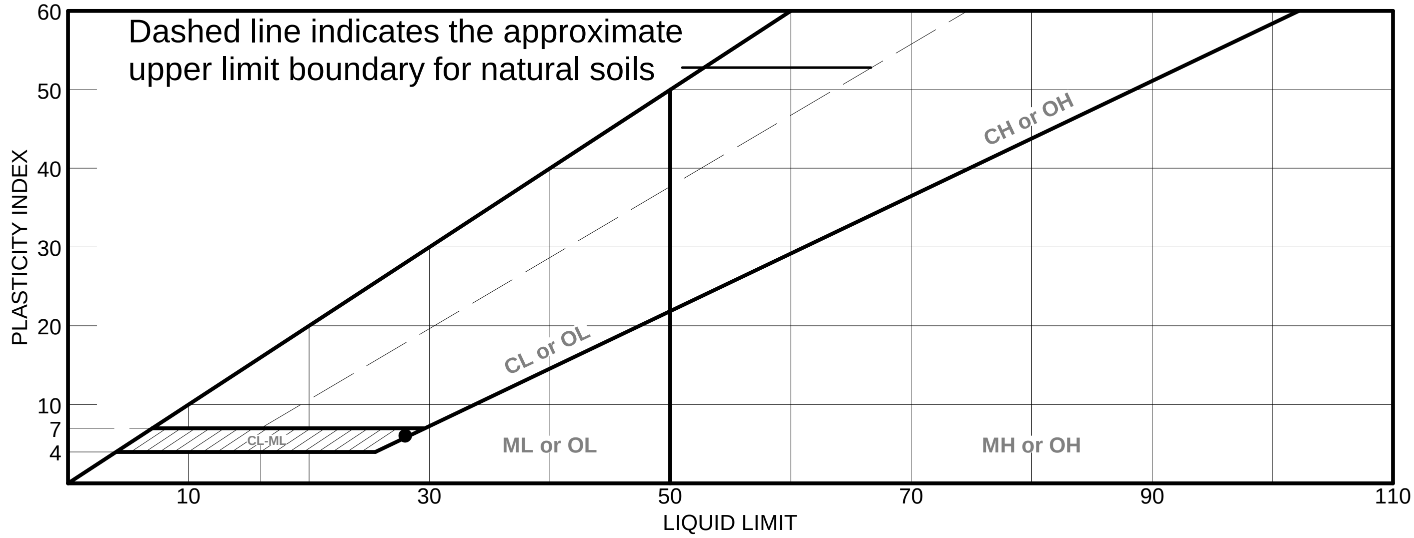
COOPER TESTING LABORATORY

Job No.:	1072-002	**Project No.:**	18-007	**Run By:**	MD
Client:	Slate Geotechnical Consultants	**Date:**	1/30/2019	**Checked By:**	DC
Project:	JUHSD				

Boring:	B-1	B-1	B-2	B-3	B-4	B-4	B-5	
Sample:	5	8	7-1	4	3-2	4	7	
Depth, ft.:	11-12.5	23.5-25	20.5-21	8.5-10	7-7.5	10-11.5	23.5-24	
Soil Type:								
Wt of Dish & Dry Soil, gm	508.9	449.1	862.5	617.6	627.1	647.5	554.7	
Weight of Dish, gm	174.9	172.8	173.8	174.7	175.1	174.0	173.1	
Weight of Dry Soil, gm	334.1	276.3	688.7	442.9	452.0	473.5	381.6	
Wt. Ret. on #4 Sieve, gm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Wt. Ret. on #200 Sieve, gm	217.1	75.6	540.8	314.9	223.1	250.1	281.2	
% Gravel	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	
% Sand	**65.0**	**27.4**	**78.5**	**71.1**	**49.4**	**52.8**	**73.7**	
% Silt & Clay	**35.0**	**72.6**	**21.5**	**28.9**	**50.6**	**47.2**	**26.3**	

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		28	22	6			

Project No. 1072-002 **Client:** Slate Geotechncial Consultants

Project: JUHSD - 18-007

● **Source:** B-2 **Sample No.:** 1-2 **Elev./Depth:** 3.4-4'

Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

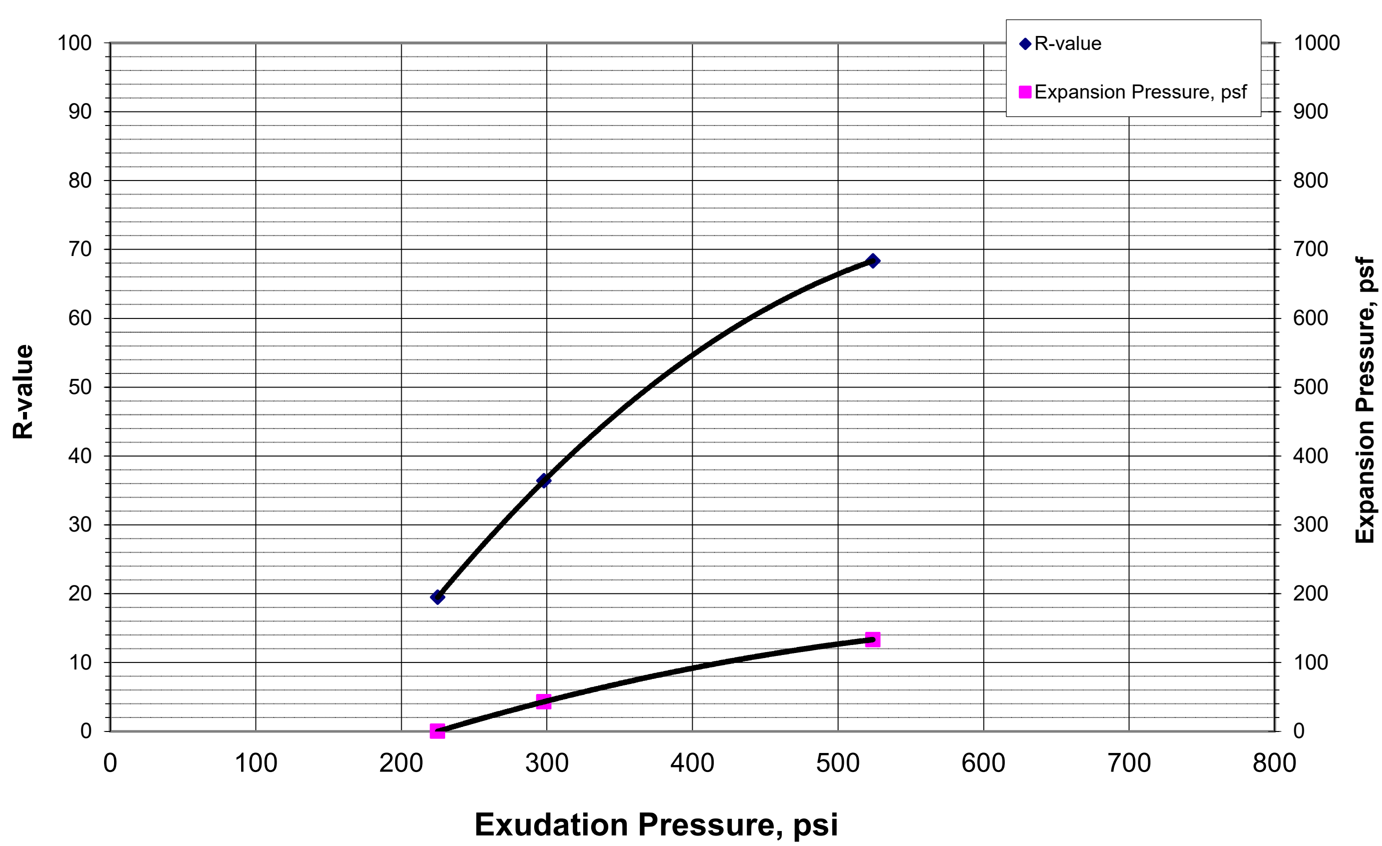
COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

Job No.:	1072-002	**Date:**	01/31/19	**Initial Moisture,**	12.1
Client:	Slate Geotechnical Consultants	**Tested**	PJ	**R-value**	**37**
Project:	18-007	**Reduced**	RU		
Sample	B-4_Bulk @ 0-5'	**Checked**	DC	**Expansion Pressure**	**40** psf
Soil Type:					

Specimen Number	A	B	C	D
Exudation Pressure, psi	225	298	524	
Prepaired Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	30	15	0	
Weight of Soil & Mold, grams	3208	3179	3159	
Weight of Mold, grams	2106	2099	2098	
Height After Compaction, in.	2.54	2.52	2.41	
Moisture Content, %	14.9	13.5	12.1	
Dry Density, pcf	114.5	114.5	119.1	
Expansion Pressure, psf	0	43	133	
Stabilometer @ 1000				
Stabilometer @ 2000	113	79	33	
Turns Displacement	4.30	4.48	4.10	
R-value	19	36	68	

Remarks:



Corrosivity Test Summary

COOPER TESTING LABORATORY

CTL # 1072-002 **Date:** 2/11/2019 **Tested By:** PJ **Checked:** PJ
Client: Slate Geotechnical Consultants **Project:** JUHSD **Proj. No:** 18-007
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-2	2-2	7.5-8	-	582	-	11	3,249	0.3249	5.9	-	21.2	
B-5	2-1	4.5-5	-	742	-	6	5,199	0.5199	6.4	-	16.4	