DRAFT

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Jefferson Union High School District Phase 2 Development - Serramonte Del Rey
Daly City, California

Prepared for:

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Prepared by:

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Project No.

18-007.01

DRAFT

SLATE
GEOTECHNICAL CONSULTANTS

April 20, 2021
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Mr. Chris White
Brookwood Group
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**Subject: PRELIMINERY GEOTECHNICAL INVESTIGATION REPORT
JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California**

Dear Mr. White:

Slate Geotechnical Consultants, Inc. (Slate) is pleased to present this report summarizing the findings of our preliminary geotechnical investigation to support the ongoing re-development planning of the Jefferson Union High School District (JUHSD) Serramonte Del Rey campus located at 699 Serramonte Boulevard in Daly City, California.

The roughly 22-acre campus is currently occupied by the historical JUHSD high school building and adjacent parking and landscape areas. The property is divided into six parcels (A through F), with Parcel A currently being developed as new staff and faculty housing for JUHSD employees. Parcels B through F are currently in conceptual planning, and include several multi-story, mixed-use developments. This report provides findings from our preliminary geotechnical investigation and recommendations that support further planning efforts for these developments, with particular focus on those parcels adjacent to the east-facing slope along Callan Boulevard.

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

Sincerely yours,

Slate Geotechnical Consultants, Inc.

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TABLE OF CONTENTS

TABLES

FIGURES

APPENDIX

DRAFT

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PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Jefferson Union High School District Phase 2 Development - Serramonte Del Rey
Daly City, California

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation performed by Slate Geotechnical Consultants, Inc. (Slate) to support the ongoing planning for re-development of the property located at 699 Serramonte Boulevard in Daly City, California. The site is currently occupied by the Jefferson Union High School District (JUHSD) historic high school and main offices and is conceptually divided into six parcels identified as Parcel A through Parcel F. Slate previously completed a geotechnical investigation for Parcel A, which is currently under construction for new JUHSD staff and faculty housing. The project site is located on the south side of Serramonte Boulevard between Callan Boulevard and Sir Francis Boulevard as shown on the Site Location Map, Figure 1.

It is our understanding that the new development on Parcels B through F under consideration includes mixed-use, low- and high-rise buildings with wood- or light steel-frame construction atop reinforced concrete-frame (podium) parking garage levels either at-grade or partially below grade. Figure 2 shows the most recent conceptual layout of the proposed developments. Current conceptual building footprints within Parcels B, C, and D are shown adjacent to the edge of an existing slope on the eastern end of the property up to roughly 55 feet to 60 feet high extending down to Callan Boulevard and Serramonte Boulevard at an inclination of about 2½(H):1(V). The western edge of the planned building in Parcel F is shown adjacent to the toe of an existing cut slope extending up to a row of single-family residences along St. Francis Boulevard to the west. The purpose of this report is to summarize preliminary findings of our geotechnical investigation and provide planning-level recommendations regarding suitable foundation types for the new structures being considered at Parcels B through F.

2.0 SCOPE OF SERVICES

To address the primary objectives of this project, we performed the following scope of services:

- Perform a detailed background review of existing subsurface information at the project site, readily available geologic maps, and historic aerial photos
- Perform a site reconnaissance to observe current conditions of the site conditions and visually assess the east and west slopes
- Perform a limited subsurface investigation across the site using cone penetration testing techniques
- Perform a preliminary seismic slope stability assessment of the eastern slope to evaluate slope performance under anticipated building loads
- Identify site-specific geologic hazards affecting the performance of the proposed structures
- Develop code-based earthquake loading parameters
- Provide assessment of most appropriate foundation types for the various proposed structures, foundation design recommendations, and preliminary estimates of total and differential foundation settlement, and
- Provide geotechnical considerations for construction.

3.0 FIELD EXPLORATION

Six cone penetration test (CPT) soundings were performed on May 8, 2020 by Gregg Drilling of Martinez, California under the guidance of Slate staff. The locations of the soundings are shown on Figure 2. 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, 1st Call Utility Locating, to clear proposed CPT locations of existing underground utilities.

The CPT soundings were performed by hydraulically pushing an instrumented 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground using a 30-ton truck-mounted rig. In general, the CPT probes were pushed until effective refusal was reached (tip resistance greater than 500 tons per square foot [tsf], or significant frictional impedance). An exception was CPT-E1, which was terminated at 80.2 feet below ground surface (bgs) and did not reach effective refusal. The other five CPT soundings reached depths ranging between 18.4 feet (CPT-B1) and 39.2 (CPT-D1) feet bgs. The CPT logs, showing tip resistance, friction ratio, and pore water pressure by depth, as well as correlated soil behavior type, are presented in Appendix A.

4.0 SITE AND SUBSURFACE CONDITIONS

The following subsections describe the Serramonte Del Rey site regional geologic, tectonic, and seismic settings. Also described are subsurface and groundwater conditions as understood from our background review, site reconnaissance, and as encountered during the preliminary geotechnical investigation 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.

As shown on Figure 3, 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 4.

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 Site Conditions

According to a geotechnical evaluation report by Woodward Clyde Consultants (WCC, 1985), and confirmed by historic aerial photography, the broader Serramonte Del Rey site straddles a ravine created by the former Chinese Creek drainage that ran southeasterly through the site. Mass grading for the original Serramonte Del Rey 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 (comprising most of Parcels B and D), 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 (comprising much of Parcels C, E, and F).

The final grade of the pad is relatively level (gently sloping to the south east) with a slope bordering the eastern edge of the property up to 60 feet high that is primarily comprised of cut native materials adjacent to Parcels B and D, and transitions to a thick undocumented fill wedge to the south (Figure 2). Fill thicknesses are expected to be up to 140 feet to the west and south of the existing JUHSD historic high school building, where proposed buildings for Parcels E and F are located. Based on aerial imagery and maps of historical topography (USGS, 1995), the thalweg of the former Chinese Creek was aligned from the northwestern corner of the property at Parcel A and ran south/southeast through the site across Parcels F and E (Figure 2).

Slate geologists conducted a limited reconnaissance of the Serramonte Del Rey site and surrounding slopes on March 11, 2020. Visual observations of the existing level grade were focused on the area in the southeast portion of the site in the area where the transition between cut and fill is expected, as indicated on the WCC (1995) site plan and shown on Figure 2. No obvious differences between expected cut and fill areas were observed on the graded portion of the site, including along the existing fire road, adjacent curbs, parking lot area, or grassy shoulder east of the fire road.

The slope east of the site is heavily vegetated and largely inaccessible by foot largely due to heavy underbrush growth and to a lesser extent debris placed or dumped on the slope (generally landscaping cuttings, downed trees and branches, and asphalt/concrete rubble). No changes in slope degradation or vegetation were observed along the length of the slope on the east side of the site, including the portion of the slope expected to be composed of a thick fill wedge (i.e., filled ravine). Although no outcrops of native materials were observed on the slope, this is not inconsistent with a lack of outcrops observed in the aerial photographs in the natural hillslopes prior to development.

The slope located west of the site appears to be similar to the east slope as it is a heavily vegetated and steep slope. The west slope was not accessed directly during the site reconnaissance because of its location behind other existing facilities and the ongoing construction activities on Parcel A. Review of the aerial photographs available indicate that the west slope has been not been modified since mass grading of the Serramonte Del Rey site was completed (in about 1969) and progressive photographs show no signs of major slope failure or slope instability since that time.

Based on the information collected for the site and reconnaissance, we deem the Merced Formation bedrock units comprise a majority of the slopes to the east and west of Parcels B through F with one exception at the southern portion of the east slope where the ravine was clearly filled in. Overall, the lack of major degradation of the slopes and consistency of slope appearance and vegetation over time are indicative of shallow native materials underlying the majority of the slopes.

4.4 Subsurface Conditions

In addition to the six CPT soundings performed for the current study, two other subsurface investigations performed for the design of the Parcel A development are considered to inform the understanding of the site-wide conditions. Those investigations by Slate (2019) and Rockridge Geotechnical (2019) generated five traditional soil borings, and five additional CPT soundings, as shown on Figure 2. Logs of borings and CPT soundings from these investigations are included in Appendix B. The Slate (2019) investigation generally classified the undocumented fill as consisting of variable compositions of sand, clayey sand, and sandy clay ranging in density from medium dense to very dense, and consistency from medium stiff to hard. Field blow counts in the fill ranged from 14 blows per foot (bpf) to 49 bpf. 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. CPT data from the Rockridge Geotechnical (2019) investigation suggests that the undocumented fill is lenticular and characterized with tip resistances ranging from 50 to 200 tsf. Native soil/rock is generally characterized with tip resistances ranging from 100 to 200 tsf in weaker horizons, and with tip resistances greater than 300 tsf in denser/stiffer horizons.

In general, the six CPT soundings performed for this study encountered materials similar to those encountered in the previous studies. CPT-B1, B2, C1, and C2 encountered what may be interpreted as weaker horizons of “shallow” native soil/rock underlain by denser/stiffer horizons of “deeper” native material at varying depths (15 feet in CPT-B1 and CPT-B2, 35 feet in CPT-C1, and 5 feet CPT-C2). CPT-D1 encountered about 35 feet of dense/stiff fill underlain by a dense/hard horizon of native soil/rock. CPT-E1 encountered about 35 feet of medium dense/medium stiff fill underlain by dense/stiff fill to the maximum depth explored of 80.2 feet. CPT-E1 was terminated before reaching the likely bottom of fill or refusal.

4.5 Groundwater Conditions

Groundwater was not encountered in any of the borings drilled at the site for the Slate (2019) investigation, nor was it identified in any of the CPT soundings from the Rockridge (2019) or current investigations. Prior geotechnical assessments of the project site (WCC, 1985) make no mention of groundwater concerns or provide considerations for managing groundwater during construction. For the purposes of a geotechnical site assessment, groundwater may be considered to be deeper than 50 feet below ground surface.

5.0 EARTHQUAKE-REALTED GEOLOGIC HAZARDS

Given the location of the Serramonte Del Rey site in relation to the many active faults within and surrounding the San Francisco Bay Area, any new developments will be subject to very strong to violent ground shaking from a future large earthquake. Therefore, there exists the potential for earthquake-related geologic hazards to impact the proposed Serramonte Del Rey site. We evaluated the potential for ground surface fault rupture, earthquake-induced landsliding, liquefaction triggering and associated ground failures, and earthquake-induced compaction/densification to impact the proposed development.

5.1 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 Serramonte Del Rey 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 Serramonte Del Rey site at the ground surface, or in the immediate vicinity of the Serramonte Del Rey 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 Serramonte Del Rey site is considered to be very low.

5.2 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 Serramonte Del Rey site is relatively level and developed, and the terrain is slightly southeast-sloping. To the west of the of the site is a moderately-sloped hillside cut, about 100 feet high, adjacent to Parcel F. To the east of the site is a 50- to 65-foot high slope extending down to Callan Boulevard, portions of which are formed from cut in natural soil/rock and others formed from undocumented fill. The east slope is adjacent to Parcels B, C, and D.

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 Serramonte Del Rey 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 developments is considered to be low. However, due to the very strong shaking anticipated at the site from a future earthquake, and the position of the proposed structures relative to the east slope, a slope stability evaluation is necessary to assess the potential for permanent slope deformations beneath building foundations. Section 6.2 presents the results of our preliminary stability evaluations and recommendations for minimum foundation offsets.

5.3 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 during the current investigation may be characterized as medium dense to very dense and medium stiff to hard fills and native soils/rock 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 Serramonte Del Rey 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.

5.4 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 subsurface surface soils (undocumented fills and weaker native soil/rock) as medium dense to dense and medium stiff to hard fills comprised of various compositions of sands, silts, and clays, there is potential for some surface manifestation of densified soil layers.

The potential for cyclic compaction/densification at the site considered the methodology by Robertson and Shao (2010) to estimate the post-shaking volumetric strains and corresponding “free-field” ground surface settlement. The computer program CLiq (version 3.0.3.2 by Geologismiki,) was used to evaluate the data collected from the CPT soundings during the current investigation. The analyses were performed considering an assumed high groundwater depth of 50 feet bgs and a peak ground acceleration (PGA) of 1.14 times gravity (g), which is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). A moment magnitude (M_W) of 8¼ was considered for the analyses, corresponding to the mean characteristic moment magnitude of the San Andreas fault.

The results of our CPT analyses suggest between ¼ to 1 inch of earthquake-induced densification is expected in the undocumented fill soils encountered at the site considering the MCE_R event. Buildings on Parcels C, E, and F may experience up to 1 inch of earthquake-induced densification depending on the thickness of undocumented fill. Due to the dense/stiff nature and composition (sufficient fines content) of the native materials, Parcels B and D are expected to experience negligible amounts of earthquake-induced densification.

6.0 PRELIMINARY GEOTECHNICAL EVALUATIONS AND RECOMMENDATIONS

The results of the subsurface investigation for the Serramonte Del Rey site described in Section 3.0 and 4.0 were used to develop preliminary geotechnical recommendations for the planning phase the proposed development. The recommendations presented in this report should not be used for final design purposes. The main geotechnical/geologic issues affecting the site are strong ground shaking from large earthquake events, permanent earthquake-induced slope deformations beneath structure foundations, and long-term differential settlement of structures built within areas of existing undocumented fill. The following sections summarize our preliminary evaluations and considerations for establishing seismic design parameters, building adjacent to existing slopes, estimated settlements under recommended foundation types, and geotechnical-related construction issues.

6.1 Site Class and Preliminary Seismic Design Parameters

Site Class is defined by ASCE 7-16 (ASCE/SEI, 2016) 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.

For Parcels B and D, the CPT soundings generally encountered dense soil/soft rock conditions from the ground surface to shallow refusal. This is consistent with a Site Class C designation. For Parcels C, E, and F, the available borings and CPT soundings performed at the site generally encountered undocumented fill of variable thickness with equivalent blow counts greater than 15 bpf over dense/stiff native soil/weathered rock. This is consistent with a Site Class D designation. Therefore, we consider both Site Class C and Site Class D classifications for the development of ground motions for the respective parcels at the Serramonte Del Rey site.

The preliminary acceleration response spectra for the Serramonte Del Rey site were developed in accordance with the 2019 California Building Code (CBSC, 2019) and ASCE/SEI 7-16 (ASCE/SEI, 2016). 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 2014 USGS National Seismic Hazard Mapping Project (NSHMP) design maps (UCERF3; Field et al, 2013). These values were obtained from the USGS/FEMA-NEHRP U.S. Seismic Design Maps web service tool (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). The values were then modified to develop code-based map-based design and Maximum Considered Earthquake (MCE_R) response spectra following ASCE/SEI 7-16. 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 C and 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}.

For Site Class C (Parcels B and D):

$$S_S = 2.417g \; ; \; F_a = 1.2 \; ; \; S_{MS} = 2.900g$$

$$S_1 = 1.013g \; ; \; F_v = 1.4 \; ; \; S_{M1} = 1.418g$$

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

For Site Class D (Parcels C, E, and F):

$$S_S = 2.417g \; ; \; F_a = 1.0 \; ; \; S_{MS} = 2.417g$$

$$S_1 = 1.013g \; ; \; F_v = 1.7 \; ; \; S_{M1} = 1.722g$$

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

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 ASCE/SEI 7-16 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 considering both Class Site C and Class Site D conditions for the project site are shown on Figure 5 and tabulated in Table 1.

6.2 Existing Slopes

It is our opinion that the majority of the eastern slope is generally suitable to support new loads from adjacent buildings, so long as they comply with the 2019 California Building Code (CBSC, 2019) requirements for minimum foundation offsets. The 2019 CBC (Section 1808.7.2) requires that the base of any foundation element (such as a slab, mat, or footing) be at a minimum horizontal offset from the face of the slope equal to H/3, where H is the height of the slope measured vertically from the toe to the crest. For a 60-foot-high slope, the minimum offset for a footing at-grade would be 20 feet. Given that the eastern slope is generally 2½(H):1(V), for every foot of foundation embedment, the building may be located about 2½ feet closer to the slope face. The 2019 CBC also permits alternative setbacks and clearances that are less than the minimum, subject to approval by the building official (in this case the City of Daly City). Such approval would be contingent on a satisfactory geotechnical investigation and slope stability assessment considering embankment material, height of slope, slope gradient, load intensity, and erosion characteristics of the slope material.

Figure 6 features the current Parcel B, C, and D conceptual development plans on existing topography of the Serramonte Del Rey site. For Parcels C and D, the nearest building edges to the east slope are offset about 69 feet and 27 feet, respectively, from the slope edge. Given that the height of the east slope adjacent to these parcels is about 55 feet, both building footprints satisfy the CBC 2019 offset requirement (minimum 18 feet) if the foundations elements are to be constructed at-grade. Parcel B, however, currently features a portion of a structure that extends to the edge of the existing slope. It is our understanding that the grade across Parcel B will likely be lowered about 5 feet from existing grade. If so, the modified slope would about 35 feet high, which would require the minimum offset to be about 12 feet; the nearest building edge to the slope would be about 7 feet from the slope edge, assuming the foundation elements are constructed at-grade. Therefore, a stability assessment is required to determine the suitability of the planned offset relative to the slope edge. Though the planned structure offsets for Parcels C and D are code-compliant, our preliminary assessment included evaluation of conditions along the east slope at each of the adjacent parcels to estimate potential permanent seismic deformations on those slopes.

Three preliminary slope stability numerical models were developed to assess the long-term (static) and seismic stability of the east slope. Figure 6 shows the locations of the three cross sections adjacent to Parcels B, C, and D. For each of the three models (Figure 7), we made simplifying assumptions about the topography, stratigraphy, shear strength parameters, and building loads based on available information, as summarized in Table 2. These models should be revised following a more detailed subsurface investigation and laboratory testing program of the site soils.

Preliminary stability analyses were performed for each cross section using limit-equilibrium methods and the commercially available program SLOPE/W (GeoStudio 2018 R2 version 9.1.2.17441; GEOSLOPE, 2012). The analyses assumed no groundwater in the embankment. Spencer's method of slices was used for computing factors of safety, which satisfies horizontal and vertical force equilibrium and overall moment equilibrium and accounts for inter-slice forces. SLOPE/W uses an "entry-exit" search scheme to generate a series of circular or wedge-type surfaces within user-defined boundaries to find the surface that generates the lowest factor of safety. The search criteria imposed for the critical slip surfaces were:

- at least the closest structure edge is intersected by the entry point of the slip surface;
- the exit point of the slip surface is below at least two-thirds of the overall slope height; and
- the potential slide mass thickness is at least 20 percent of overall slope height.

The search criteria were intended to discount shallower slip surfaces and those outside the footprints of the proposed buildings, which do not generally pose a significant risk to the structures.

Long-term (static) slope stability evaluations assumed drained loading conditions, which considers effective stress strength parameters (Table 2). The calculated static factors of safety (FS_{STATIC}) for the critical slide masses shown on Figure 8 are 2.7 (Parcel B), 3.1 (Parcel C), and 2.6 (Parcel D), which are generally acceptable for long-term performance of slopes.

Seismic slope stability evaluations assumed undrained loading conditions, which considers total stress strength parameters (Table 2). The calculated post-earthquake factors of safety (FS_{PE}) for the critical slide masses shown on Figure 9 are 4.1 (Parcel B), 2.6 (Parcel C), and 3.2 (Parcel D). These factors of safety indicate that the slope will be generally stable following an earthquake but may still sustain permanent downslope deformations as a result of strong ground shaking.

Post-earthquake deformation potential of the three idealized slope slip surfaces were estimated using the simplified relationship developed by Bray and Travasarou (2007). This method is based on a series of nonlinear, one-dimensional, fully coupled, Newmark-type sliding block analyses on several different embankment heights and material stiffnesses to generate a probabilistic-based deformation estimate. The method uses as input the initial predominant period of the slide mass, T_s (which can be calculated as a function of the height of the slope and dynamic stiffness of the embankment material), the spectral acceleration of the input motion at the degraded period of the sliding mass (assumed to be 1.5 times the initial predominant period, $S_a(1.5T_s)$, the yield acceleration of the sliding mass, k_y, and the earthquake magnitude, $M_{W.}$

The yield coefficient, k_y, of a potential slide mass is the minimum pseudo-static horizontal acceleration required to produce instability along defined slip surface. k_y is estimated by evaluating the critical slip surface (slip surface generating the lowest post-earthquake factor of safety) for a range of horizontal seismic coefficients and identifying which generates a pseudo-static factor of safety of 1.0. The critical slip surfaces, their post-earthquake factors of safety (FS_{PE}), and associated yield coefficients, for each of the three slopes are shown graphically on Figure 9 and summarized in Table 3.

Preliminary median deformation estimates for each slip surface using the method described above, are presented in Table 3. Because the Bray and Travasarou (2007) method requires an understanding of the initial dynamic stiffness of the embankment, a range of potential stiffnesses (i.e., shear wave velocities, V_S) were considered in a sensitivity analysis of the deformation potential. In general, the estimated range of shear wave velocities for the materials comprising the slopes was assumed to be 1,200 to 2,000 ft/s. Analyses considered both the design and MCE_R spectra developed in Section 6.1. The deformation values reported in Table 3 are consistent with the V_S values that result in the largest deformation estimates for that particular slide mass.

The results of the preliminary seismic slope stability and deformation analyses suggest that the east slope has the potential for significant permanent displacements under the considered earthquake loads, which could adversely undermine portions of the proposed structure foundations on Parcels B, C, and D. Preliminary calculations indicate up to about 4 inches of slope deformation below a foundation element that is founded at grade for Parcel B, 18 inches for Parcel C, and 9 inches for Parcel D, considering the MCE_R event. The design earthquake event suggests smaller potential deformations. It should be noted that these procedures for estimating deformations utilize simplified, chart-based solutions, and are often used initially for screening processes. Estimates may be better refined considering more rigorous analysis approaches, such as an equivalent-linear site response or nonlinear analysis, and/or with the collection of additional material-specific shear strength data.

Potential deformations may be mitigated by deepening the foundation elements to an elevation below the anticipated failure plane, transferring building loads to a lower elevation with deep foundation elements, and/or positioning the building further from the slope edge. A structural engineer should be consulted to assess acceptable levels of deformation to the superstructure for the various earthquake loading scenarios. Deformation estimates should be revised following a more detailed subsurface investigation and laboratory testing program in the future.

6.3 Preliminary Foundation Assessment

Anticipated building loads for the proposed structures on Parcels B through F were estimated for this assessment based on the building type tabulation summarized on the JUHSD Precise Plan Site Exhibit A2.7 by Seidel Architects dated 03/12/2020. The tabulation suggests that most buildings are proposed to be light frame construction (wood and/or steel) ranging from 3 to up to 11 stories atop 1 to 4 stories of podium-type parking garage levels. We understand that the proposed buildings are currently planned to be founded at-grade. Contact stresses were estimated based on assumed dead plus live loads for steel frame construction and concrete podium levels, and anticipated use. The following summarizes the estimated column load and contact stresses for the various structures, assuming loads are uniformly distributed across the footprint of the building.

Parcel	Podium Levels	Light Frame Levels	Estimated Column Loads (DL+LL) [kips]	Estimated Contact Stress for Mat Foundation [psf]
B	3	4	965	1100
C	1	3	520	600
D	4	3	1020	1200
	4	11	1900	2200
E	3	3	855	1000
	3	11	1735	2000
F	2	4	800	900
Podium Only	2	0	265	300

In general, the native and undocumented fill materials 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. The following sections provide preliminary foundation assessments for structures founded primarily on undisturbed native material (Parcels B and D) and those founded primarily in undocumented fill zones (Parcels C, E, and F).

6.3.1 Parcels B and D

Structures on Parcels B and D are anticipated to be founded on dense/stiff undisturbed native materials. We preliminarily recommend that any low-rise structures up to 7 stories may be supported by conventional shallow (individual spread, or strip) foundations. Foundations should bear on undisturbed native soils, be at least 18 inches wide, and extend at least 24 inches below lowest adjacent grade. We preliminary recommend an allowable bearing capacity, q_{all}, of 4,000 pounds per square foot (psf) for dead plus live loads. The allowable bearing pressure can be increased one-third for total design loads, including wind and seismic loads. The allowable bearing pressures for dead plus live and total design loads include factors of safety of about 2.0 and 1.5, respectively. Individual foundation elements are expected to experience less than ¼ inch of total vertical settlement over the lifetime of the building under normal loading conditions. Differential settlements are expected to be much less than ¼ inch over a horizontal distance of 30 feet.

Alternatively, a structural mat foundation may be considered to distribute loads of either low- or of high-rise structures (such as the proposed 11-story podium structure on Parcel D) more uniformly across the building footprint. A structural mat is expected to experience less than ¼ inch of total vertical settlement over the lifetime of the building under normal loading conditions, considering a contact stress of 2,200 psf. Differential settlements are expected to much less than ¼ inch over a horizontal distance of 30 feet.

Portions of the proposed structures that are immediately adjacent to the east slope may consider the addition of deep foundation elements, such as cast-in-drilled-hole (CIDH) piers, to mitigate potential displacements resulting from permanent slope deformation during a strong shaking event (see Section 6.2).

6.3.2 Parcels C, E, and F

Structures on Parcels C, E, and F are proposed in areas comprised either partially, or wholly, of undocumented fill soils of varying thickness from 0 to 140 feet. Large foundation loads in these areas have the potential to cause erratic settlements between buildings of varying construction type and height, and across transitions from shallow native soil/rock to thicker areas of undocumented fill. In this regard, it may not be feasible to support podium-type structures on traditional shallow spread footings. Structural mat foundations in combination with ground improvement may be considered to more evenly distribute column loads and mitigate total and differential settlements. A deep foundation system may also be considered to further minimize settlements to a tolerable level. Non-podium structures under three stories, however, may be supported on spread or strip footings that are lightly loaded.

For Parcels C, E, and F, buildings founded on unmodified undocumented fill are expected to experience total and differential vertical settlements over the lifetime of the building under normal loading conditions, considering the following load schedule:

Contact Stress (DL+LL) [psf]	Expected Total Vertical Settlement [inches]	Expected Differential Settlement over 50 feet [inches]
500	< ¼	< ¼
1000	up to 1	up to ½
2000	up to 2	up to 1

A contact stress of 500 psf is consistent with a mat foundation supporting the planned 4-story building on Parcel C. A contact stress of 2,000 psf is consistent with a mat foundation supporting a 14-story building on Parcel E. Differential settlements will likely be most pronounced at the transitions across structures that bear directly on native soils at one end the proposed building to thicker, undocumented fill zones to the west and south of the Serramonte Del Rey property, such as those on Parcel C, E, and F. In addition, we judge there is the potential for up to 1 inch of earthquake-induced densification in the undocumented fill soils encountered at the site considering the MCE_R event (see Section 5.4). A structural engineer should be consulted to determine if expected settlements are within acceptable tolerance levels.

There are several different types of ground improvement methods that may be feasible for this project. We consider soil-cement mix columns or drilled displacement columns to be the most appropriate ground improvement methods for this site. Soil-cement mix columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers. Drilled displacement columns are installed by advancing a hollow-stem auger equipped with a displacement head that mostly displaces the soil and then pumping sand-cement into the hole under pressure as the auger is withdrawn. Drilled displacement columns generate fewer drilling spoils to off-haul. Both soil-cement mix columns and drilled displacement columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the columns should be determined by the design-build contractor based on the desired level of improvement. If soil improvement is to be considered, we recommend that preliminary design criteria, including calculations of static and seismic settlement be prepared by the ground improvement contractor and submitted for our review.

Where total or differential settlements are not tolerable, the proposed buildings may be supported on deep foundations. Although there are many deep foundation options, we preliminarily judge that proprietary systems such as auger-cast-in-place (ACIP) piles or torque-down piles (TDPs) are the most appropriate deep foundation systems for this site. These systems require less soil off-haul compared to CIDH piles and do not generate high vibrations and noise like driven piles. ACIP piles are installed by advancing a continuous flight, hollow-stem auger into the ground and backfilling the hole with sand-cement grout or concrete under pressure as the auger is removed. After the auger is removed, steel reinforcement can be installed while the cement-grout or concrete is still fluid. Partial displacement augers may be used to minimize the volume of soil off-haul. ACIP piles are available in a variety of diameters between 12 and 48 inches; however, 16 inches is the most common. ACIP can be installed to depths up to 100 feet. A TDP is a steel pipe with a closed conical end with pitched flights that allow the pipe pile to be screwed into the soil resulting in displacement and densification of the surrounding soil. The pipe typically used for the TDPs has an outside diameter of 12.75 inches and a wall thickness of 0.375 (3/8) inches. When the pipe pile is advanced to the design tip elevation, it is filled with structural concrete to provide additional bending resistance. TDPs are displacement piles installed with little spoils created to reduce off-haul. The advantages of ACIP piles and TDPs is they can be installed with minimal vibration and noise, as compared to driven piles.

6.4 Other Foundation Considerations

The overall structural evaluation of the proposed buildings will include seismic and wind loads, which may require the consideration of lateral load- and uplift-resisting foundation elements. Foundations that bear at-grade may develop lateral resistance from sliding friction along the base of the foundation. However, they provide little to no passive resistance against the leading face of the foundation element. Additional passive (lateral) resistance may be provided by embedding foundations at a nominal depth below adjacent grade, inclusion of partial, or full basement levels, or with a deep foundation system. Overturning resistance may be provided from the installation of a deep foundation system.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary assessment and recommendations contained in this report were based on planning-level site development information, limited subsurface exploration data, a review of available subsurface explorations for the development of Parcel A, and our experience in the area with similar projects. Recommendations are not intended for final design. As site conditions vary significantly between the CPT soundings performed during this investigation, we recommend that we be retained to perform a design-level geotechnical investigation specific to each parcel development that includes additional borings and/or CPT soundings. New explorations in shallow native material may extend to depths of approximately 60 feet bgs. New explorations in undocumented fill may extend to depths up to 150 feet bgs once detailed site development plans are available.

Once our final field investigation and report has been completed and the project team has selected a foundation system, 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.

8.0 LIMITATIONS

The evaluations made for 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. The information provided in this report was prepared for a planning-level study for the proposed development(s) at the JUHSD Serramonte Del Rey site described in this report, specifically for use by the Jefferson Union High School District, its agents, and the project planning 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.

9.0 REFERENCES

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TABLES

Table 1

CODE-BASED MAP-BASED DESIGN AND MCE_R LEVEL RESPONSE SPECTRA ORDINATES
Preliminary Geotechnical Investigation Report
JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

T (sec)	Spectral Acceleration, S_A (g)			
	Site Class C		Site Class D	
	Map-Based Design Reponse Spectrum	Map-Based MCE_R Spectrum	Map-Based Design Reponse Spectrum	Map-Based MCE_R Spectrum
0.01	0.892	1.338	0.712	1.069
0.02	1.011	1.516	0.780	1.170
0.03	1.129	1.694	0.848	1.272
0.05	1.367	2.050	0.984	1.476
0.075	1.663	2.495	1.153	1.730
0.1	1.934	2.900	1.323	1.984
0.15	1.934	2.900	1.611	2.417
0.2	1.934	2.900	1.611	2.417
0.25	1.934	2.900	1.611	2.417
0.3	1.934	2.900	1.611	2.417
0.4	1.934	2.900	1.611	2.417
0.5	1.891	2.836	1.611	2.417
0.75	1.261	1.891	1.531	2.296
1	0.945	1.418	1.148	1.722
1.5	0.630	0.945	0.765	1.148
2	0.473	0.709	0.574	0.861
3	0.315	0.473	0.383	0.574
4	0.236	0.355	0.287	0.431
5	0.189	0.284	0.230	0.344
7.5	0.126	0.189	0.153	0.230
10	0.095	0.142	0.115	0.172

Table 2

SUMMARY OF PROPERTIES USED IN SLOPE STABILITY ANALYSES
Preliminary Geotechnical Investigation Report
JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

Parcel	Slope Height (feet)	Foundation Load (psf)	Foundation Offset (feet)	Material	Unit Weight (pcf)	Shear Strength Parameters			
						Effective		Total	
						ϕ' (degrees)	c' (psf)	ϕ (degrees)	c (psf)
B	35	1100	7	Shallow Native	130	40	0	20	1000
				Deep Native	135	45	0	25	2000
C	55	600	69	Undocumented Fill	130	40	0	20	1000
D	55	2200	27	Shallow Native	130	40	0	20	1000
				Deep Native	135	45	0	25	2000

Table 3

RESULTS OF PRELIMINARY SLOPE STABILITY AND DEFORMATION ANALYSES
Preliminary Geotechnical Investigation Report
JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

Parcel	**FS_{STATIC}**	**FS_{PE}**	**k_y**	**Deformation (inches)[1]**	
				Map-Based Design Reponse Spectrum	**Map-Based MCE_R Response Spectrum**
B	2.7	4.1	1.12	1	4
C	3.1	2.6	0.44	8	18
D	2.6	3.2	0.78	3	9

Note

1. Median deformations calculated using Bray & Travasarou (2007)

FIGURES

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Figure created in ESRI ArcMap. Basemap is World Topo Map by ESRI and others.

Project No: 18-007.01	Date: 3/31/2020	Created By: SAM	Checked By: --	Figure No. 1

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SITE LOCATION

Preliminary Geotechnical Investigation Report, JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

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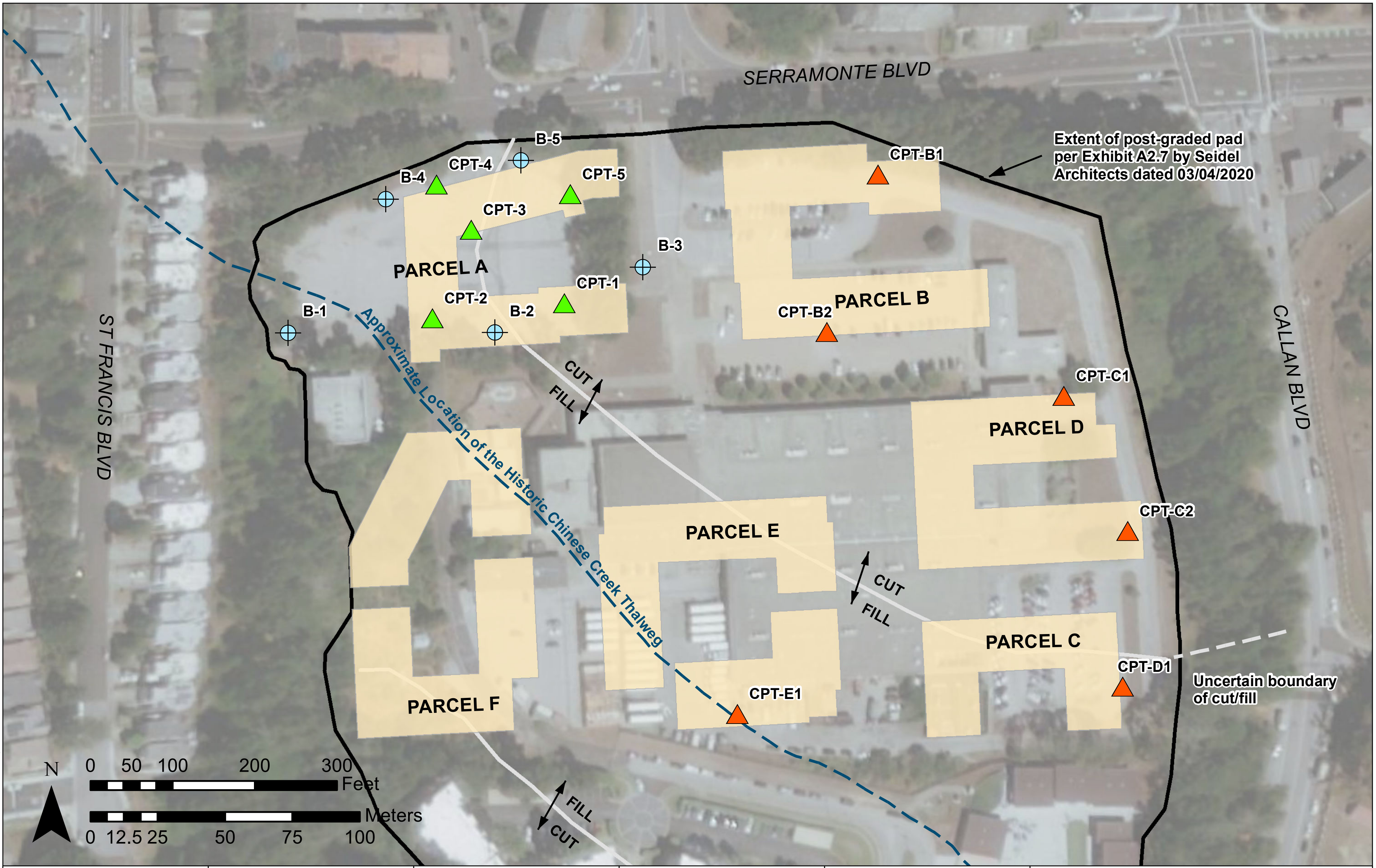

Project No: 18-007.01 | Date: 5/21/20 | Created By: SAM | Checked By: -- | Figure: 2

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SITE PLAN

Preliminary Geotechnical Investigation Report, JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

EXPLANATION

▲ CPT-1	Rockridge (2019)
⊕ B-1	Slate (2019)
▲ CPT-B1	Slate (2020)

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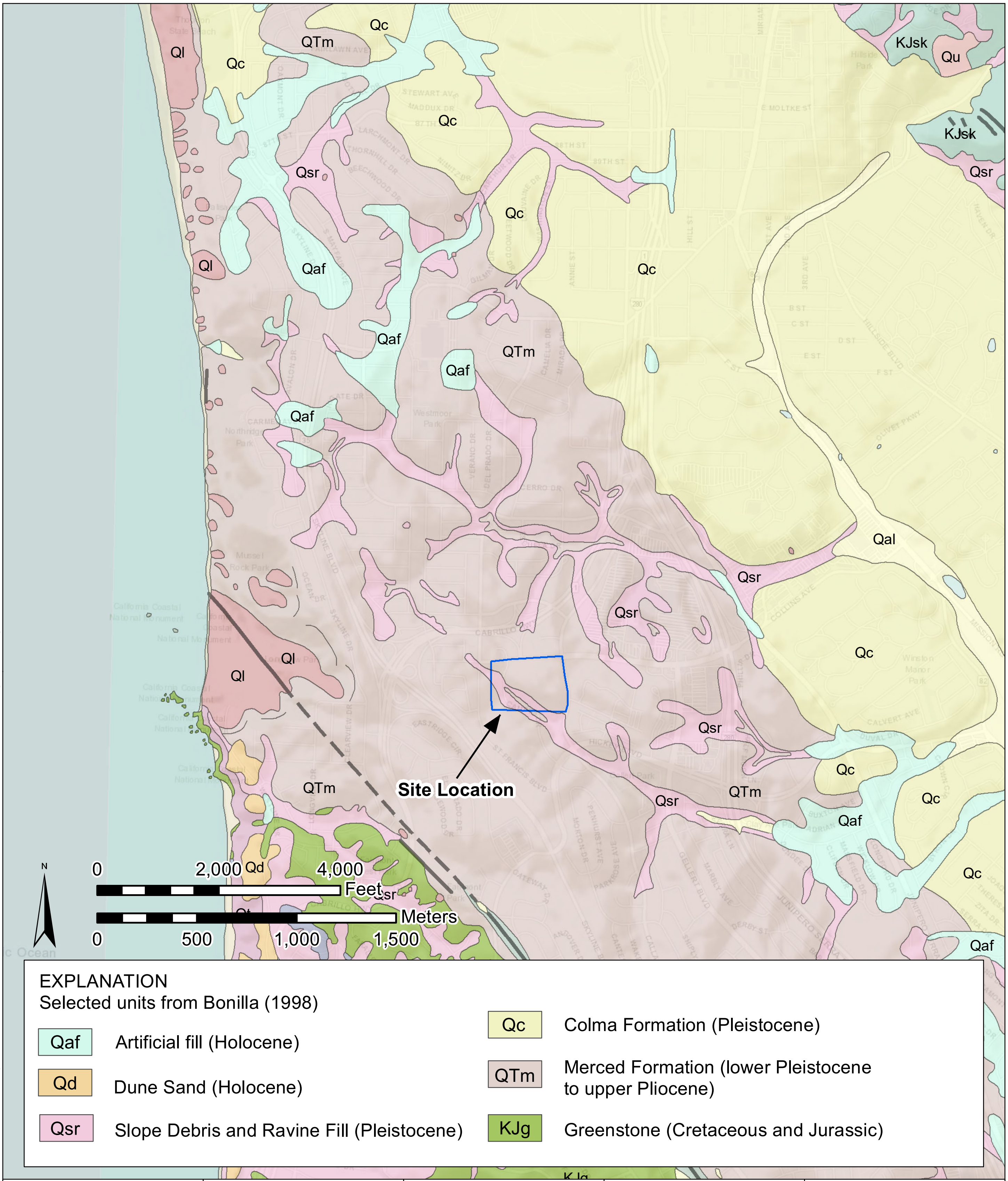

EXPLANATION
Selected units from Bonilla (1998)

Qaf	Artificial fill (Holocene)
Qd	Dune Sand (Holocene)
Qsr	Slope Debris and Ravine Fill (Pleistocene)
Qc	Colma Formation (Pleistocene)
QTm	Merced Formation (lower Pleistocene to upper Pliocene)
KJg	Greenstone (Cretaceous and Jurassic)

Project No: 18-007.01
Date: 5/22/2020
Created By: SAM
Checked By: JDP
Figure No: **3**

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REGIONAL GEOLOGIC MAP

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Daly City, California

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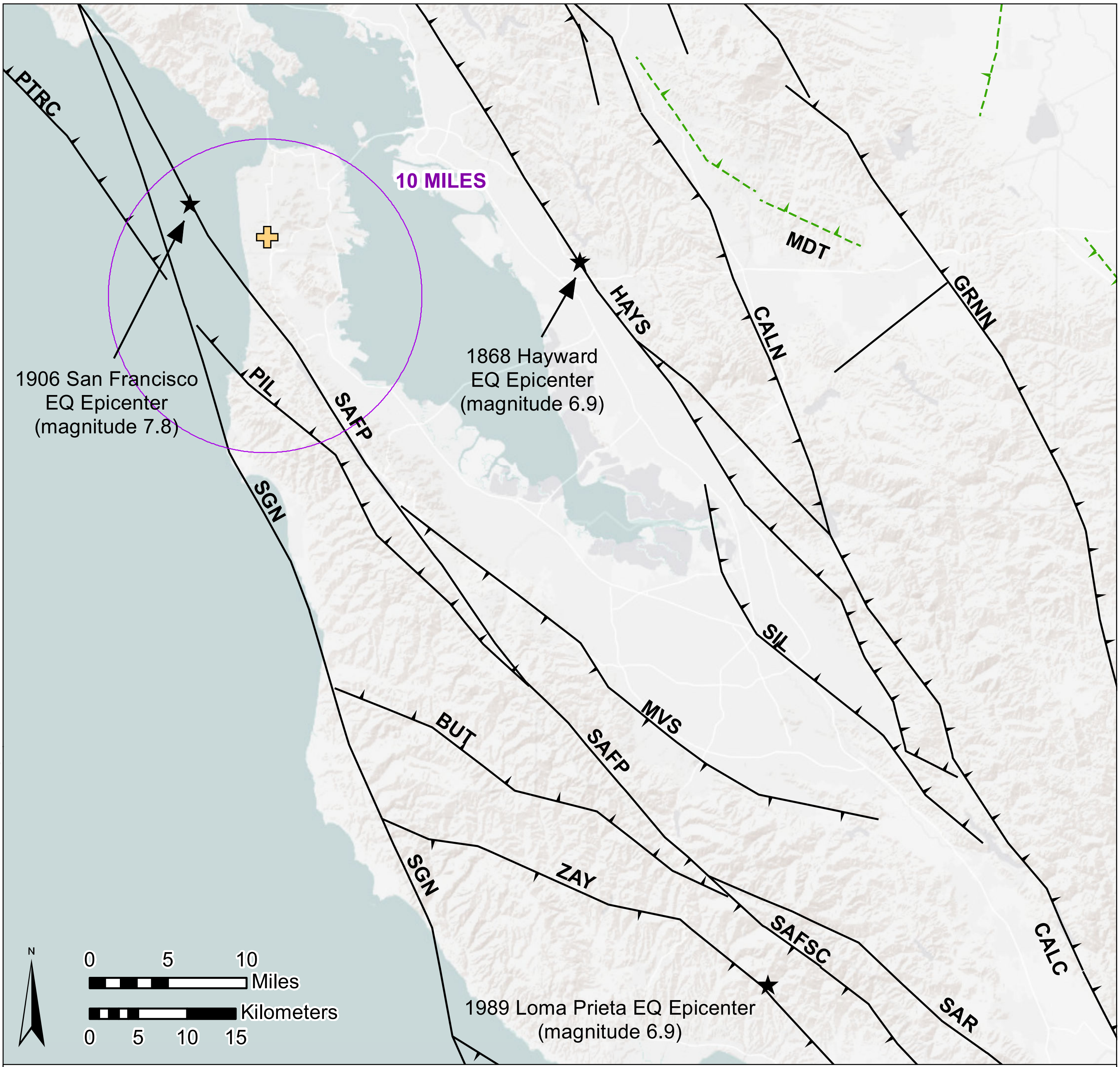

EXPLANATION

UCERF Model 3.1 Faults (Field et al., 2013)

— Vertical fault

—┬— Dipping fault (teeth point down dip)

- -┬- Buried, dipping fault (teeth point down dip)

Figure created in ESRI ArcMap. Basemap is World Terrain Base by ESRI and others.

SELECTED FAULTS:
Butano [BUT]; Calaveras Central [CALC]; Calaveras North [CALN]; Greenville North [GRNN]; Hayward South [HAYS]; Monte Vista - Shannon [MVS]; Mount Diablo Thrust [MDT]; Pilarcitos [PIL]; Point Reyes Connector [PTRC]; San Andreas Peninsula [SAFP]; San Andreas Santa Cruz Mountains [SAFSC]; San Gregorio North [SGN]; Sargent [SAR]; Silver Creek [SIL]; Zayante - Vergeles [ZAY]

Project No: 18-007.01	Date: 3/31/2020	Created By: SAM	Checked By: --	Figure No: **4**

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REGIONAL FAULT MAP

Preliminary Geotechnical Investigation Report, JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

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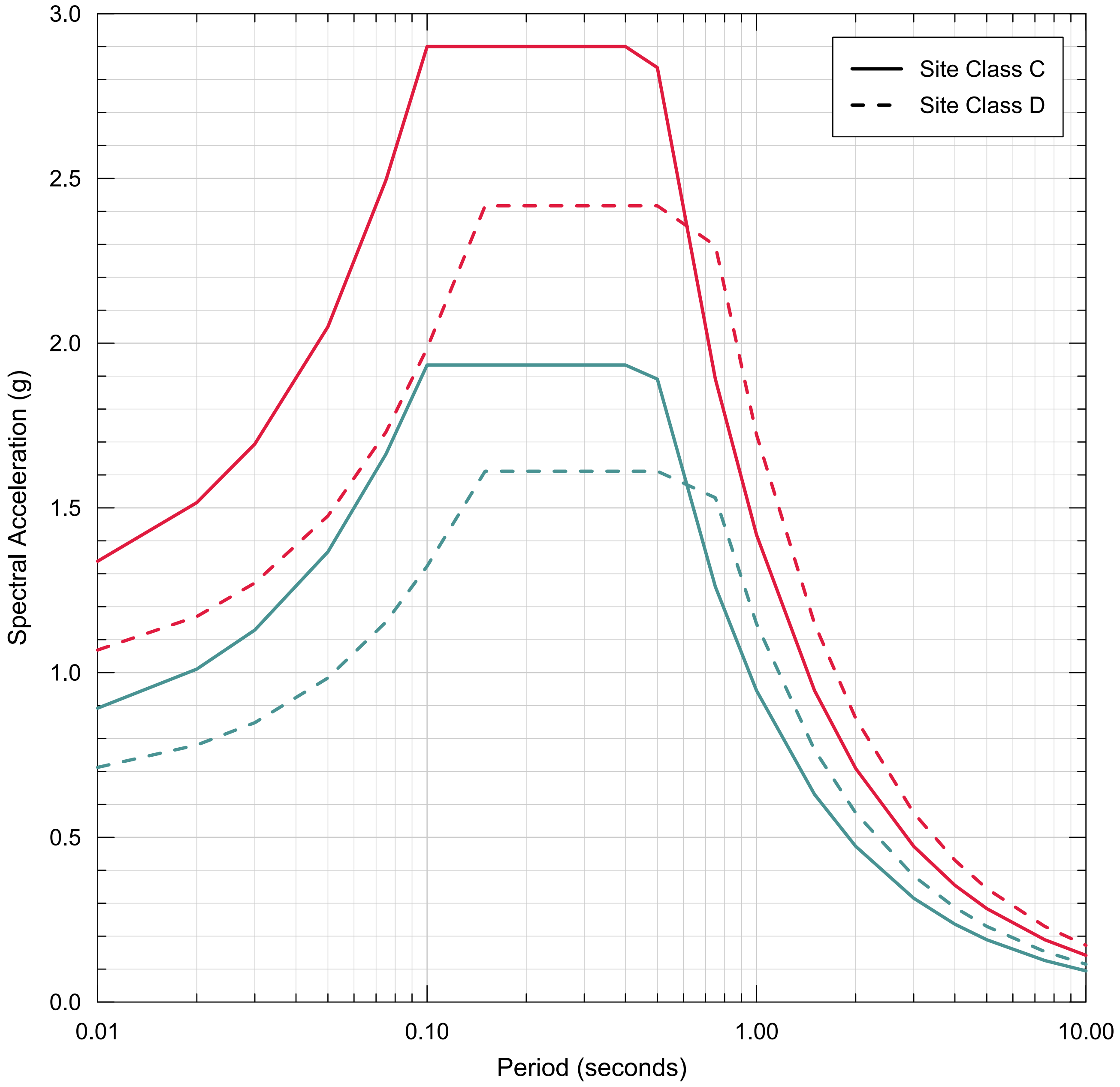

━	MCE_R Level Spectrum	━	Map-Based Design Response

Note:
Spectra are damped 5%, except PGA

Project No: 18-007.01	Date:	Created By: HMC	Checked By: JDP	Figure No: **5**

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CODE MAP-BASED DESIGN AND MCE_R LEVEL RESPONSE SPECTRA

Preliminary Geotechnical Investigation Report – JUHSD Phase 2 Development – Serramonte Del Rey
Daly City, California

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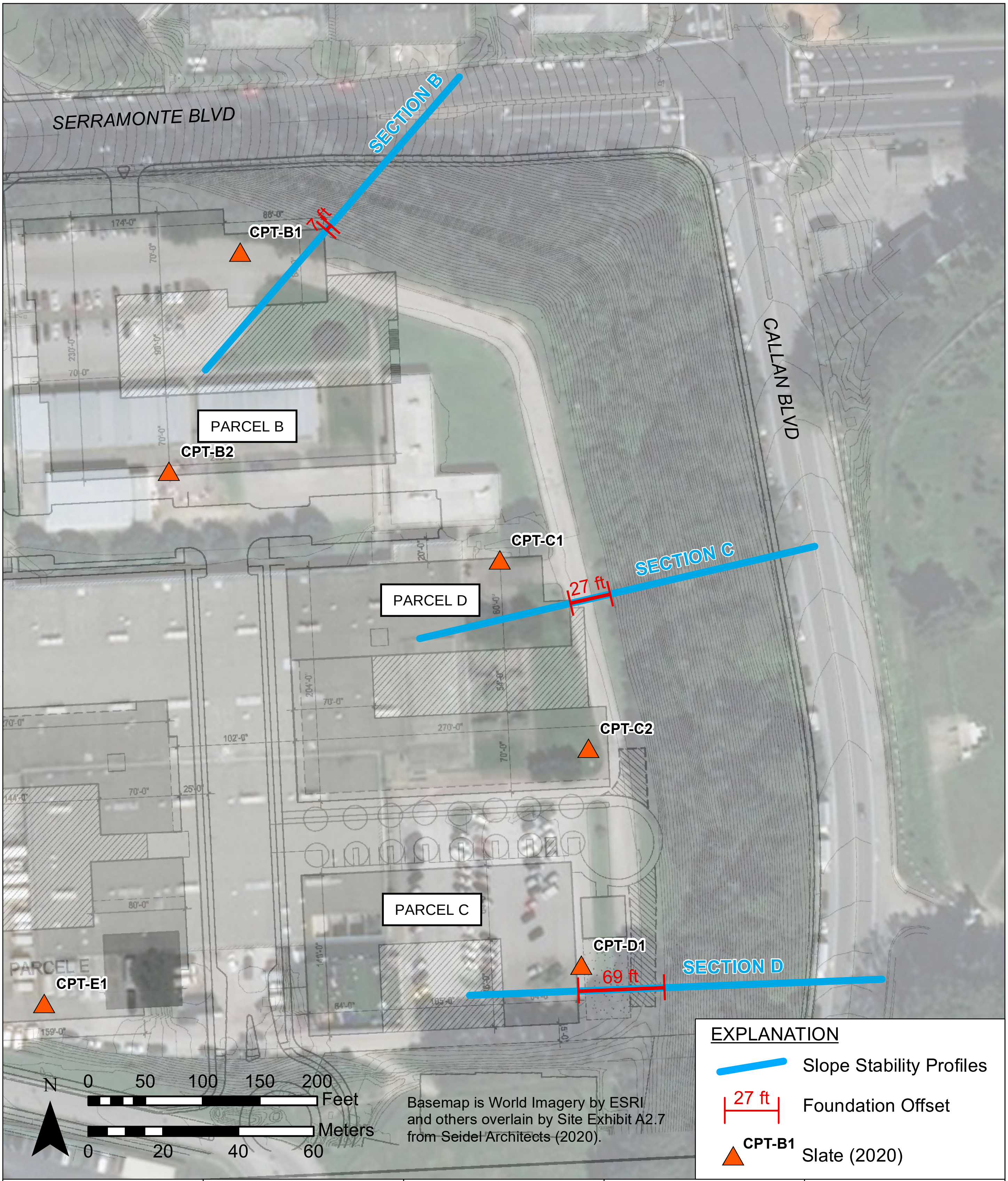

Basemap is World Imagery by ESRI and others overlain by Site Exhibit A2.7 from Seidel Architects (2020).

Project No: 18-007.01	Date: 5/22/20	Created By: SAM	Checked By: --	Figure: 6

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Locations of Conceptual Cross Sections for Slope Stability Analyses

Preliminary Geotechnical Investigation Report, JUHSD Phase 2 Development - Serramonte Del Rey
Daly City, California

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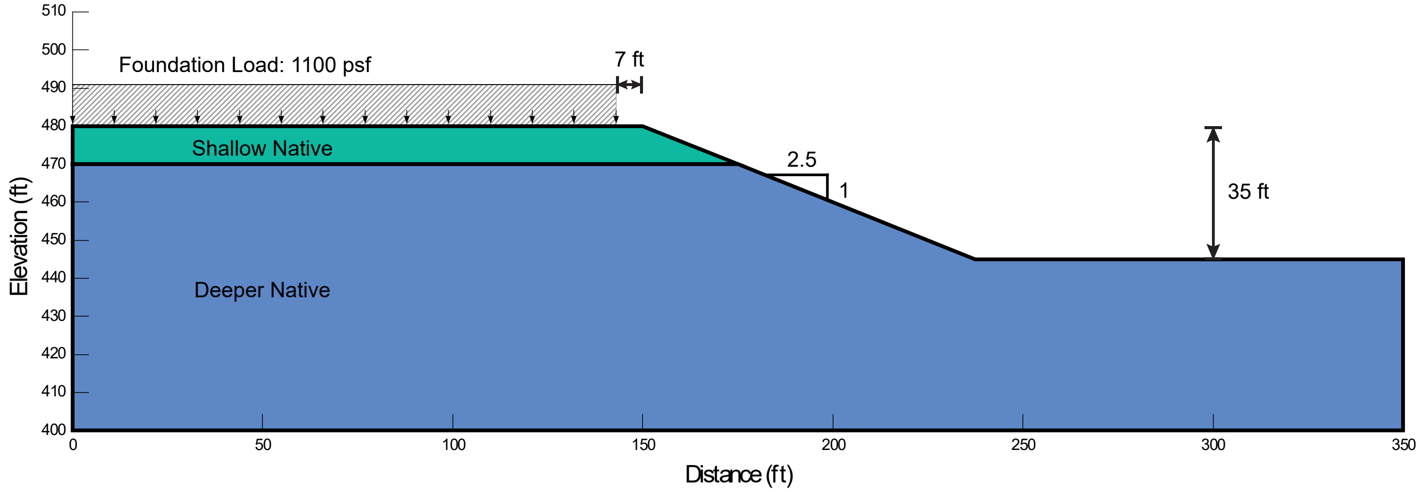
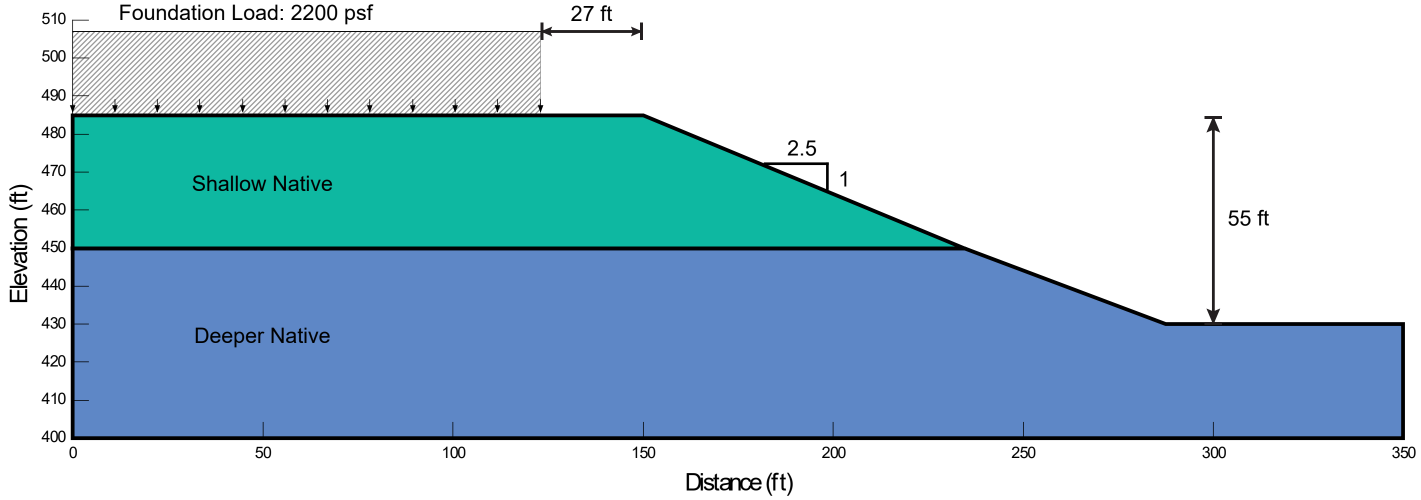
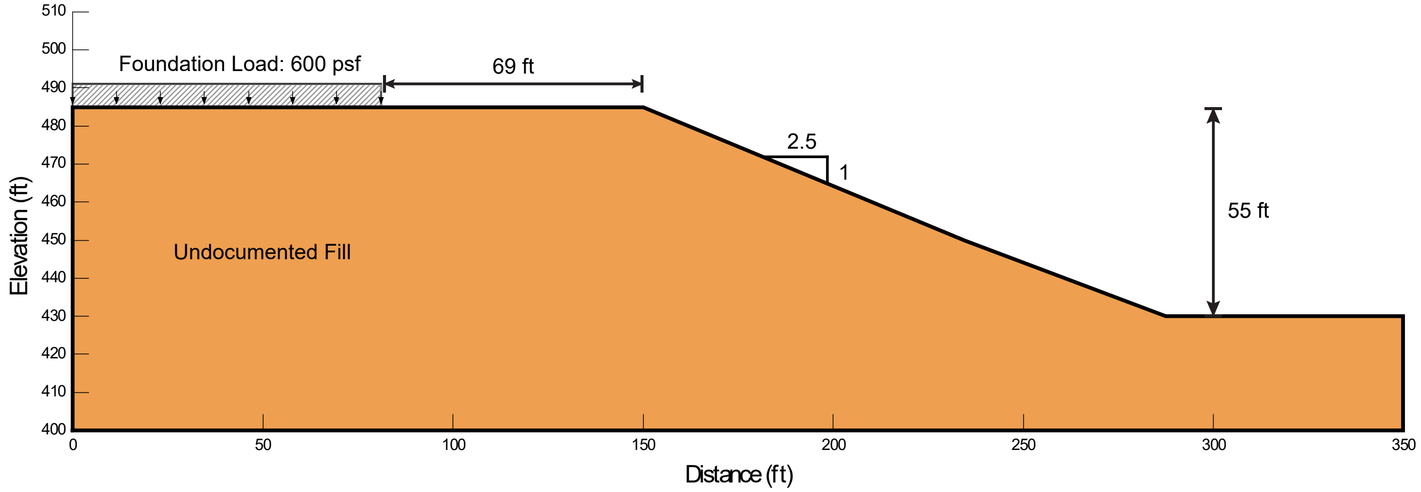
SECTION B (PARCEL B)



SECTION C (PARCEL D)



SECTION D (PARCEL C)



Project No: 18-007.01	Date: 5/20/2020	Created By: HMC	Checked By: JDP	Figure No: **7**

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CONCEPTUAL CROSS SECTIONS USED FOR SLOPE STABILITY ANALYSES

Preliminary Geotechnical Investigation Report – JUHSD Phase 2 Development – Serramonte Del Rey
Daly City, California

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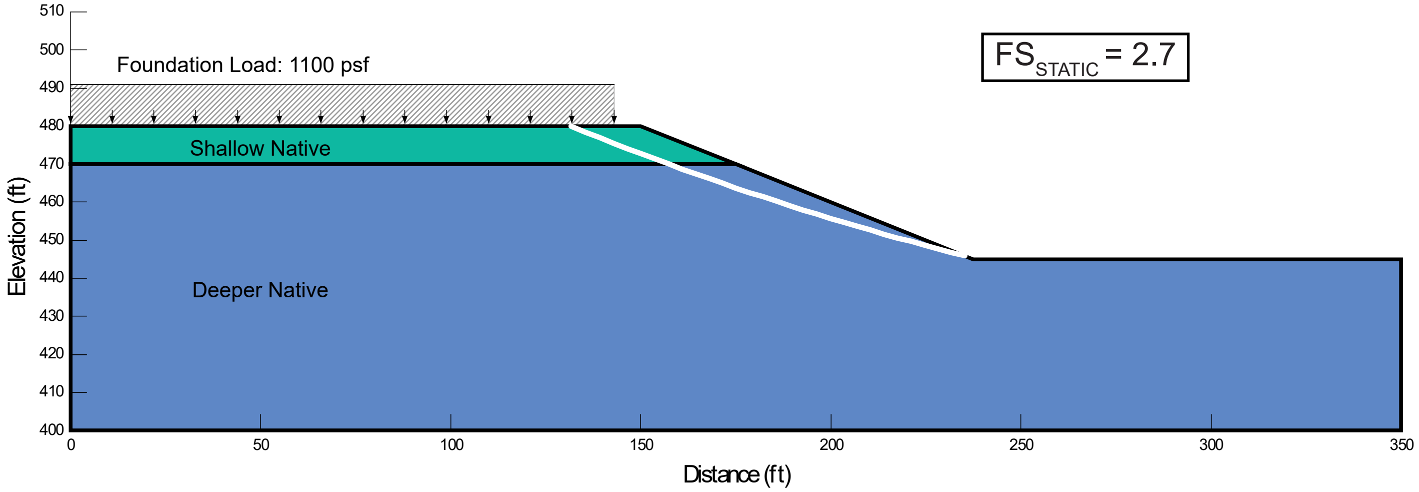
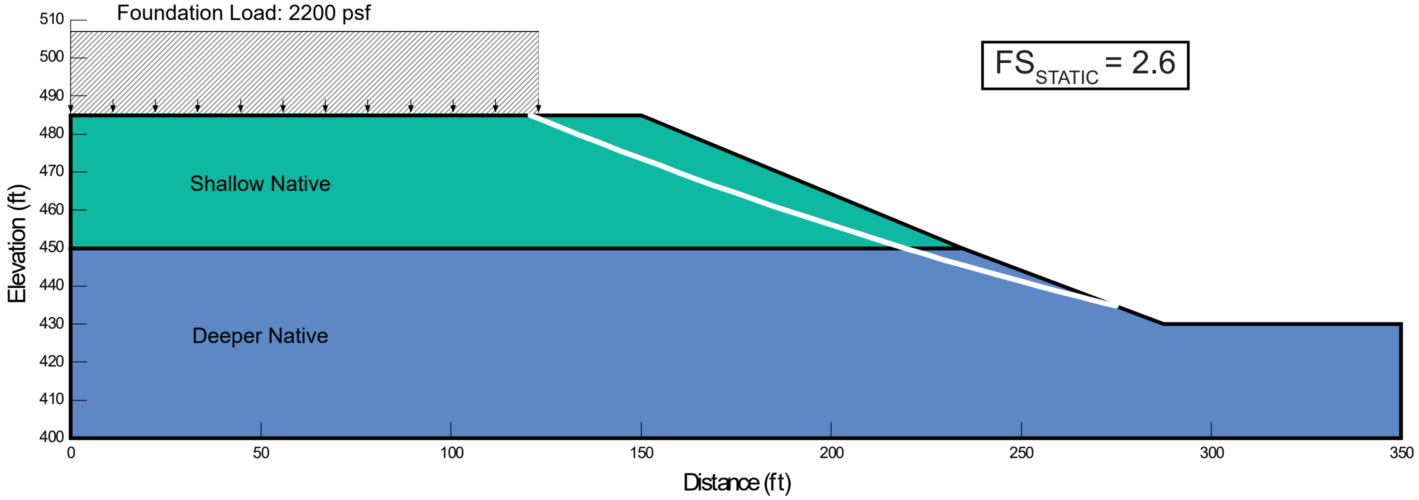
DRAFT

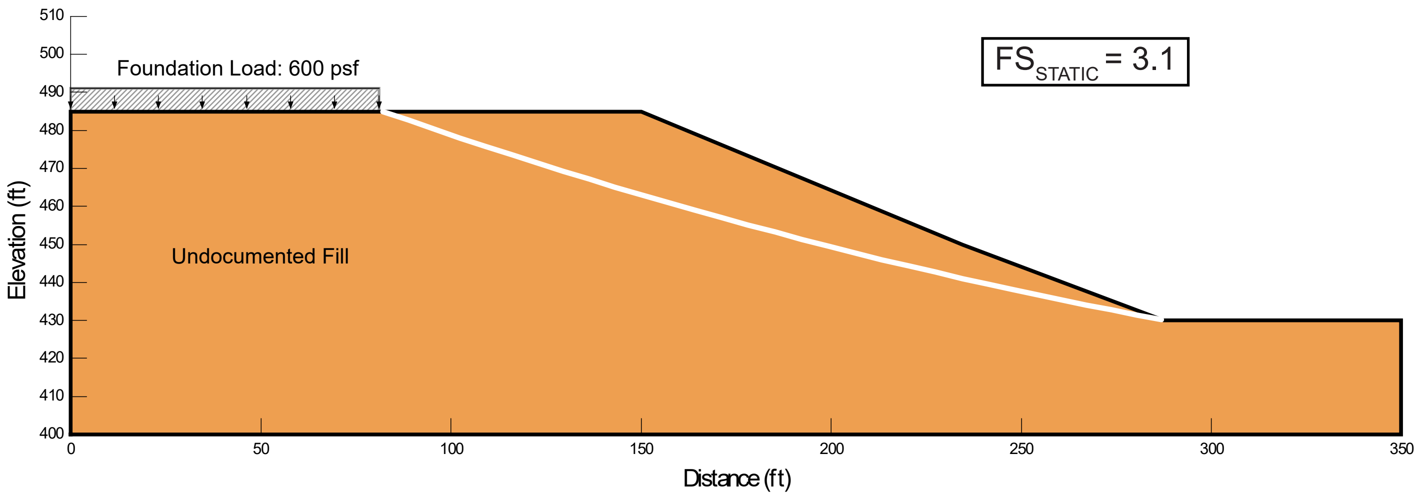
SECTION B (PARCEL B)



SECTION C (PARCEL D)



SECTION D (PARCEL C)



Project No: 18-007.01	Date: 5/20/2020	Created By: HMC	Checked By: JDP	Figure No: **8**

SLATE
GEOTECHNICAL CONSULTANTS

RESULTS OF LONG-TERM (STATIC) SLOPE STABILITY ANALYSES

Preliminary Geotechnical Investigation Report – JUHSD Phase 2 Development – Serramonte Del Rey
Daly City, California

Filepath: SlateGeoTech\SlateDrive - Documents_Projects\18-007.01_JUHSD_Phase 2 Geotech Assessment\04_Eval_Analysis\00_Figures\Fig 08_Static Stability.ai

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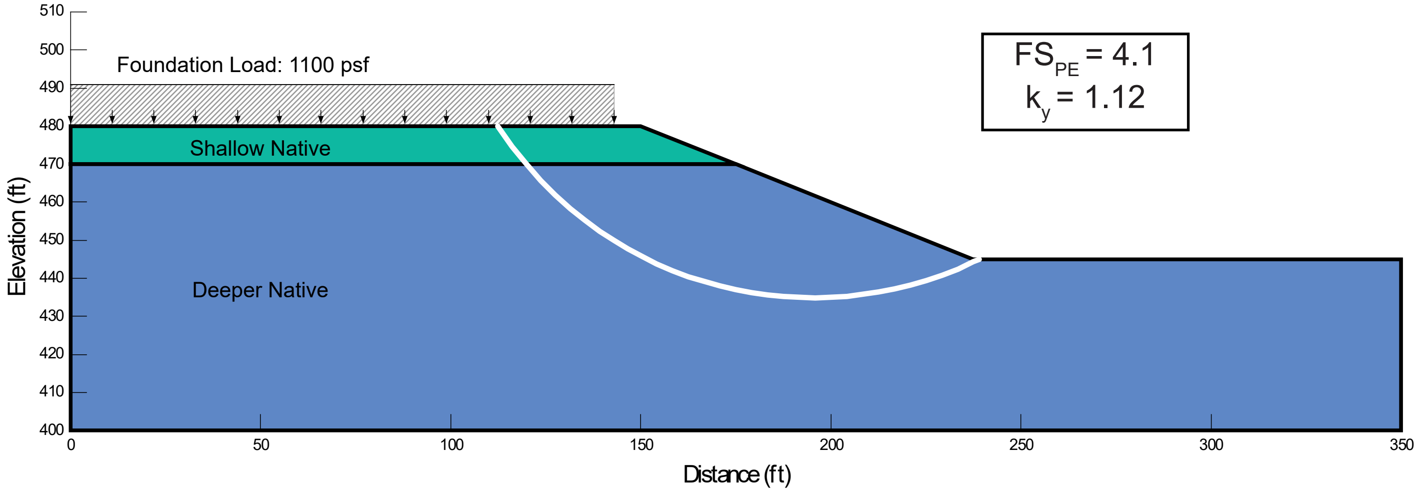
SECTION B (PARCEL B)



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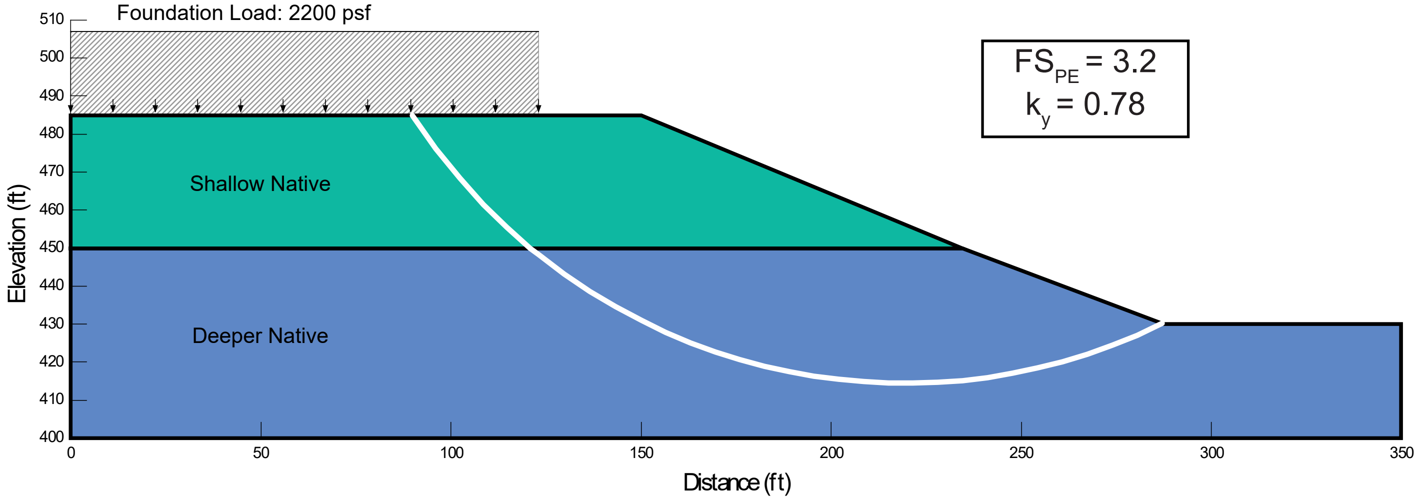
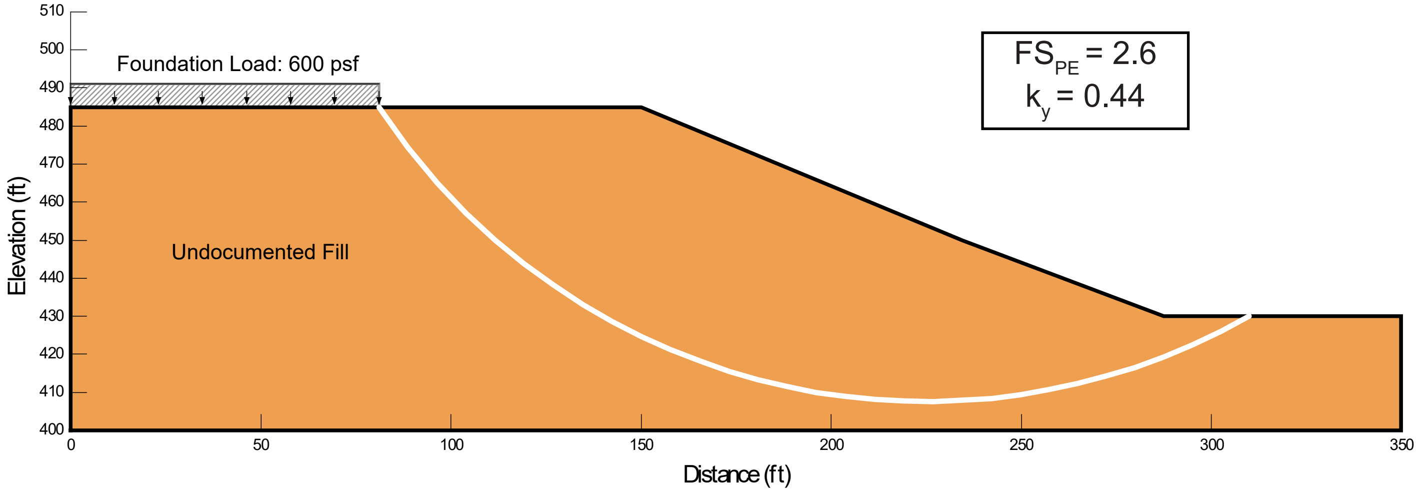


SECTION D (PARCEL C)



Project No: 18-007.01	Date: 5/20/2020	Created By: HMC	Checked By: JDP	Figure No: **9**

SLATE
GEOTECHNICAL CONSULTANTS

RESULTS OF SEISMIC (POST-EARTHQUAKE) SLOPE STABILITY ANALYSES

Preliminary Geotechnical Investigation Report – JUHSD Phase 2 Development – Serramonte Del Rey
Daly City, California

Filepath: SlateGeoTech\SlateDrive - Documents_Projects\18-007.01_JUHSD_Phase 2 Geotech Assessment\04_Eval_Analysis\00_Figures\Fig 09_PE Stability.ai

APPENDIX A

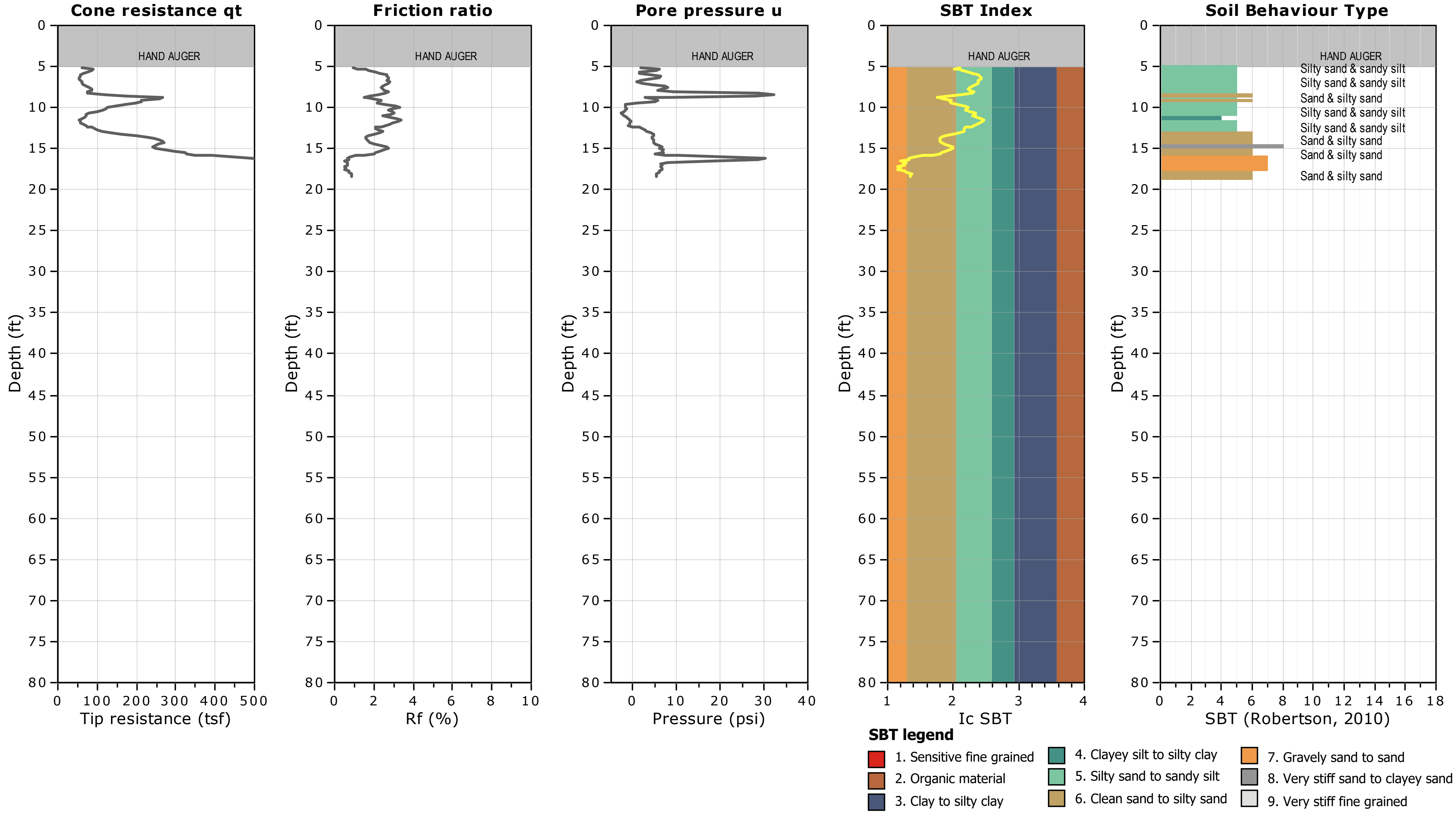
CONE PENTRATION TEST RESULTS

Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

DRAFT

CPT: CPT-B1
Total depth: 18.37 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**

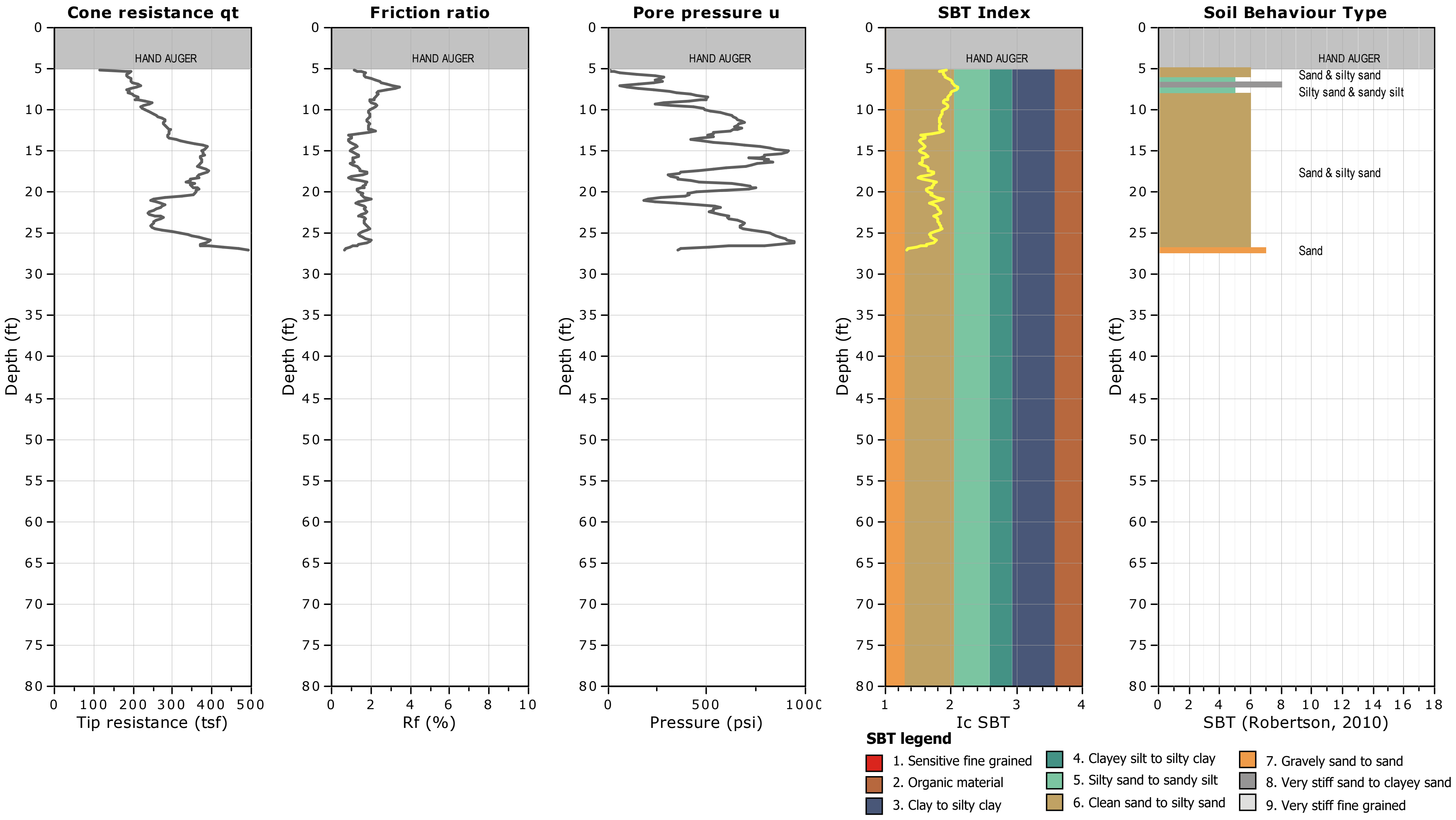


Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

DRAFT

CPT: CPT-B2
Total depth: 27.07 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**

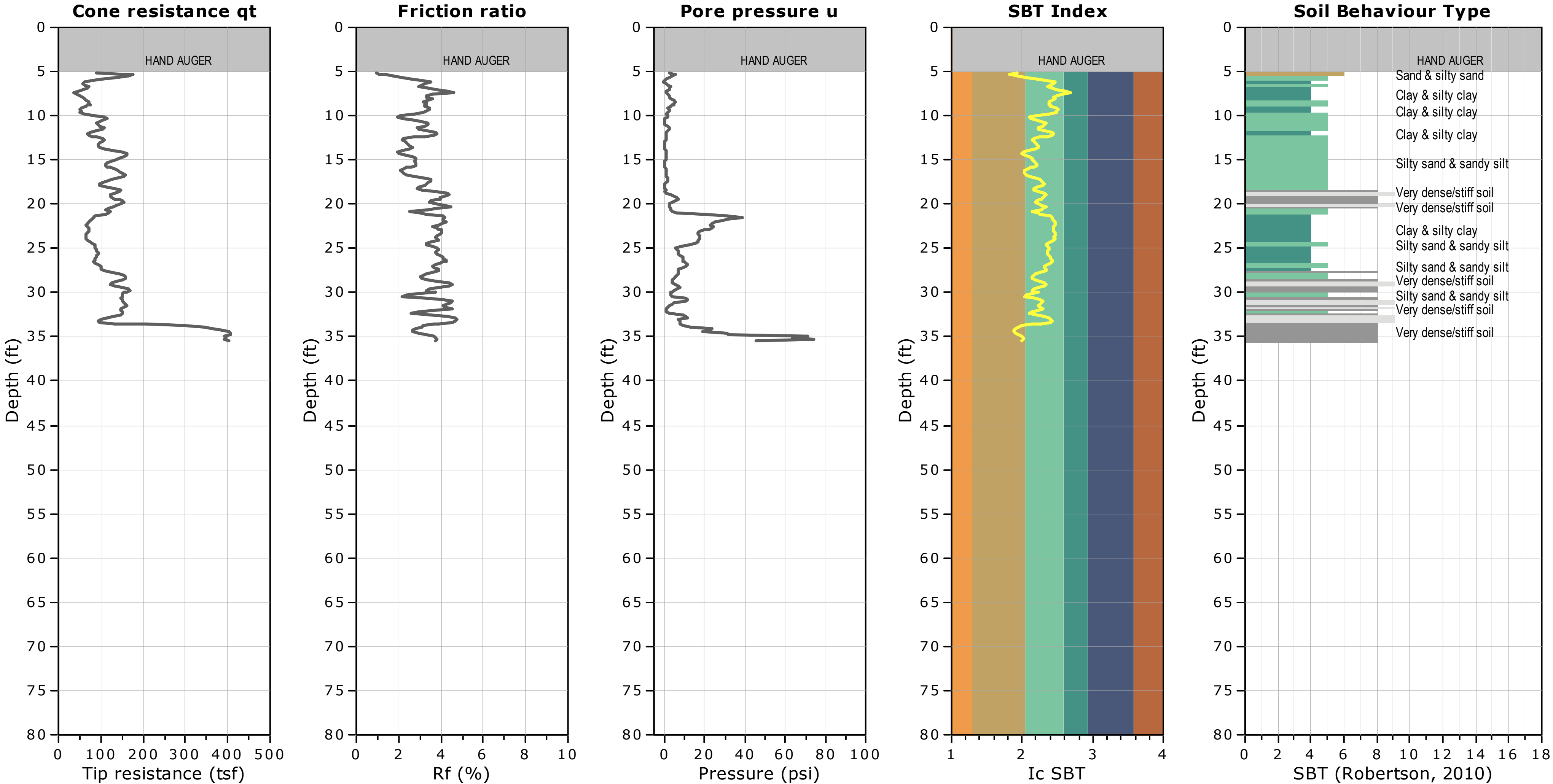


Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

DRAFT

CPT: CPT-C1
Total depth: 35.43 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**



SBT legend

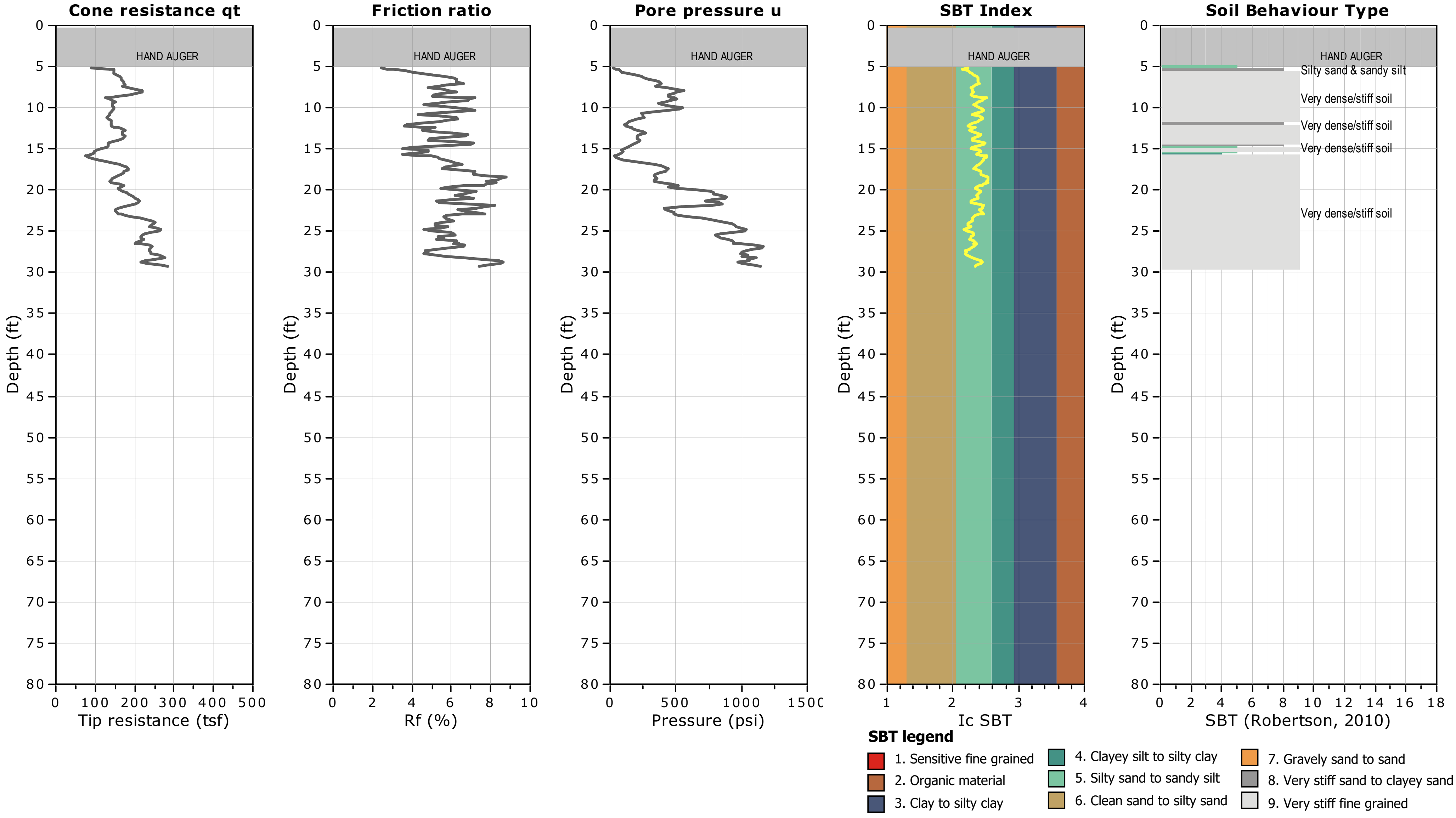
🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

DRAFT

CPT: CPT-C2
Total depth: 29.20 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**

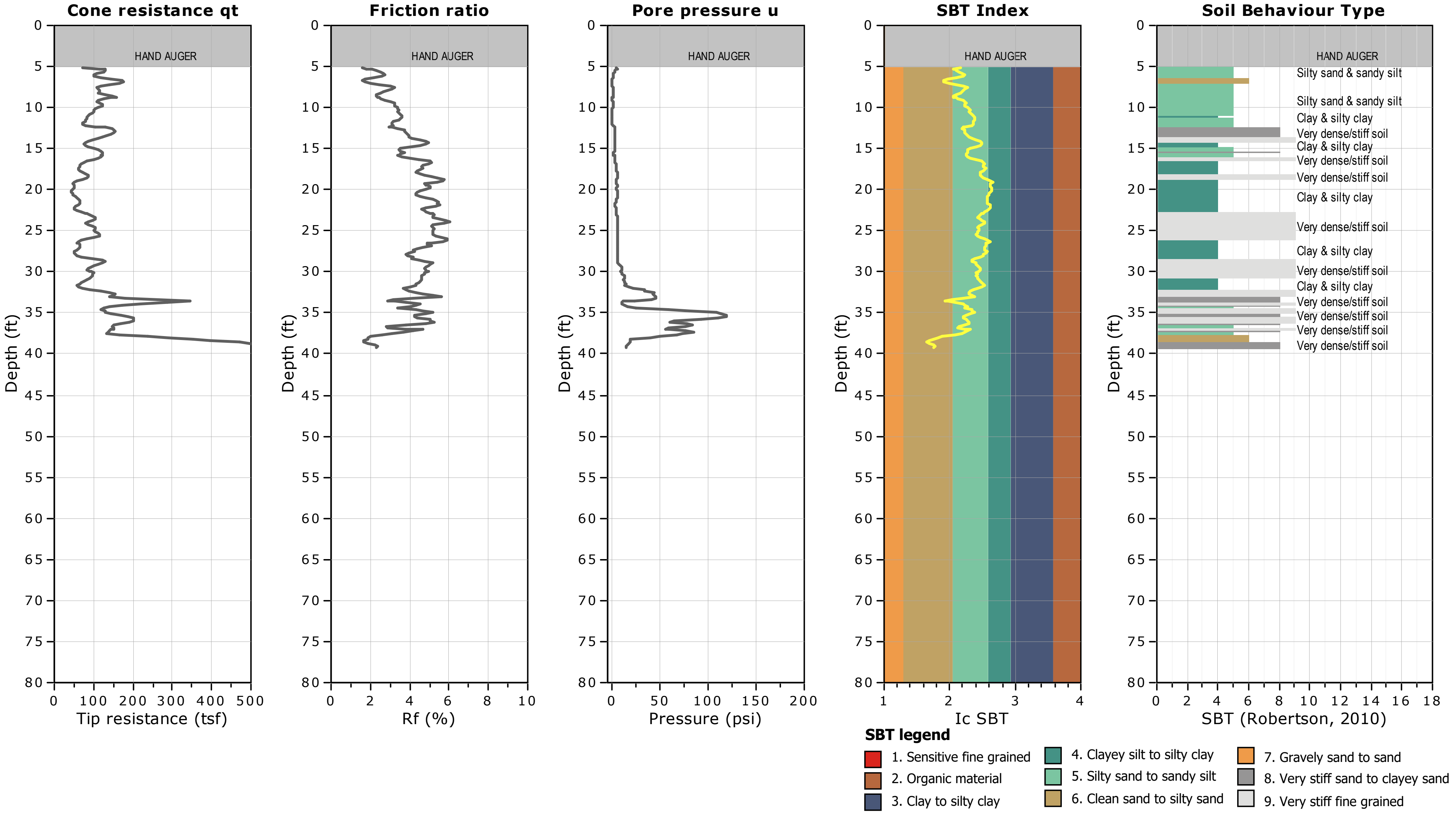


Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

DRAFT

CPT: CPT-D1
Total depth: 39.21 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

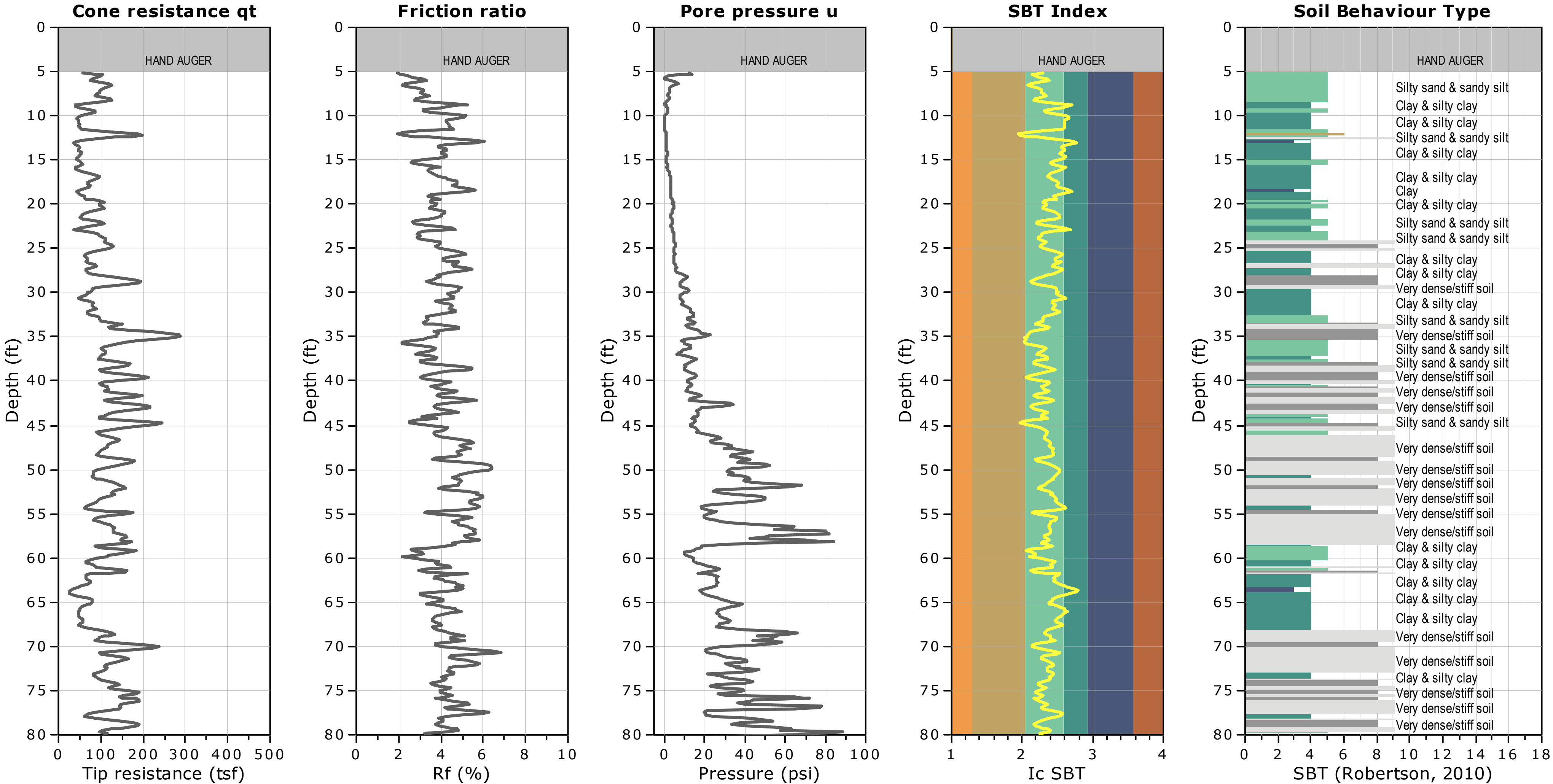
Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**



Slate Geotechnical Consultants, Inc.
Oakland, CA
www.slategeotech.com

CPT: CPT-E1
Total depth: 80.22 ft, Date: 5/13/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Gregg Drilling, LLC

Project: **Preliminary Geotechnical Investigation Report - JUHSD Phase 2 Development - Serramonte Del Rey**
Location: **Daly City, California**



SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

APPENDIX B

LOGS OF BORINGS AND CPT SOUNDING PROFILES FROM PREVIOUS INVESTIGATIONS

SLATE, 2019

DRAFT

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	**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		DESCRIPTION
SPT	MOD-CAL	
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		DESCRIPTION
SPT	MOD-CAL	
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

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**

SLATE
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BORING LOG EXPLANATION

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

Filepath: S:\SlateDrive - Documents_Projects\18-007.00_JUHSD_Geotechnical_Investigation\05_Field\Boring Logs

DRAFT

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

Log of Boring B-1

PAGE 1 OF 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)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	Field N-Value[1]			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches of asphalt 2 inches of aggregate base						
1					SC	CLAYEY SAND (SC) Dark grayish brown (2.5Y 4/2) with yellowish brown (10YR 5/6) mottling, medium dense, moist, fine sand						
2	MC	■	10 11 11	22								
3					CL							
4	MC	■	9 13 14	27		SANDY LEAN CLAY (CL) Very dark greenish gray (Gley 1 3/1), stiff, moist, fine sand						
5					SC	CLAYEY LEAN SAND (SC) Very dark greenish gray (Gley 1 3/1), medium dense, moist, fine sand						
6												
7	MC	■	7 24 25	49		Becomes dense						
8						SANDY LEAN CLAY (CL) Black (10YR 2/1), very stiff, moist, fine sand, low plasticity						
9	SPT	◪	10 14 15	29	CL							
10						CLAYEY SAND (SC) Yellowish brown (10YR 5/6), medium dense, moist, fine sand						
11					SC							
12	SPT	◪	10 12 13	25		Olive brown (2.5Y 4/3), black and yellowish brown mottling (10YR 5/8), with clay lenses				35		
13						SANDY LEAN CLAY (CL) Interbedded dark greenish gray (Gley 1 4/1), and yellowish brown (10YR 5/8), hard, moist, fine sand						
14	SPT	◪	6 18 17	35								
15					CL							
16						FILL?						
17						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						
18												
19	SPT	◪	12 10 19	29	SC							
20												
21												
22						LEAN CLAY with SAND (CL) Stratified yellowish brown (10YR 5/6) and dark greenish gray (Gley 1 3/1), very stiff, moist, fine sand						
23												
24	SPT	◪	11 15 13	28						73		
25												
26					CL							
27												
28												
29	SPT	◪	8 9 13	22		Mottled grayish brown (2.5Y 5/2) and strong brown (7.5YR 5/8)						
30												
31												

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

SLATE
GEOTECHNICAL CONSULTANTS

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

DRAFT

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

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: Field N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Liquid Limit (%)	LABORATORY TEST DATA: Plastic Limit (%)	LABORATORY TEST DATA: Plasticity Index (%)	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
32					CL	SANDY LEAN CLAY (CL) (continued)	FILL?						
33													
34	SPT	◪	7 9 14	23									
35													
36													
37													
38													
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.

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GEOTECHNICAL CONSULTANTS

Project No.: 18-007

Figure: A-1b

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

Log of Boring B-2

PAGE 1 OF 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)	SAMPLES Sampler Type	Sample	Blows/ 6"	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						2.5 inches of asphalt						
1					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
7	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	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	MC	■	5 8 14	22	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						
14	MC	■	9 10 18	28	SC	CLAYEY SAND (SC) Yellow brown (10YR 5/6) mottled with brown (10YR 4/3), medium dense, moist, fine sand						
0–20						FILL?						
20	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	SPT	◪	39 50/3"	50/3"								
30	SPT	◪	50/5"	50/5"								
31												

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

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GEOTECHNICAL CONSULTANTS

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

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

Log of Boring B-3

PAGE 1 OF 1

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 (%)	LABORATORY TEST DATA: Plastic Limit (%)	LABORATORY TEST DATA: Plasticity Index (%)	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: 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 FILL?						
2–3	BULK	☒			SC	CLAYEY SAND (SC) Light olive (2.5Y 5/4), moist, fine sand						
4.5–5					CL	SANDY LEAN CLAY (CL) Very dark brown (10YR 2/2), hard, moist, fine sand						
5	MC	■	18 50/5"	50/5"								
6	SPT	◪	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"						29		
14	SPT	◪	28 50/3"	50/3"		Trace shell fragments						
19	SPT	◪	17 50/5"	50/5"		Few medium sand						
24	SPT	◪	19 50/5"	50/5"		Few fine gravel, decomposed and fractured						
27.5					ML	LEAN SILT with SAND (ML) Dark greenish gray (Gley 1 4/1), hard, moist, fine sand, trace shell fragments						
29	SPT	◪	25 50/5"	50/5"								

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

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

Log of Boring B-4

PAGE 1 OF 1

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	Sample	Blows/ 6"	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
						3 inches of asphalt						
1						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						
2	MC		6 7 7	14								
3												
4					SC							
5												
6						Coarse gravel at 6 feet						
7	MC		9 9 11	20		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				51		
8					CL							
9	MC		4 4 12	16								
10						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						
11	SPT		9 11 13	24						47		
12												
13												
14	SPT		10 13 14	27		2 inch clay seam, medium plasticity fines						
15												
16												
17												
18	SPT		8 10 16	26								
19					SC							
20												
21												
22												
23												
24	SPT		9 14 16	30		Yellowish brown (10YR 5/8) with strong brown (7.5YR 5/6) mottling, medium dense						
25												
26												
27												
28												
29	SPT		7 12 14	26								
30												
31												

FILL?

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

DRAFT

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)	SAMPLES Sampler Type	Blows/ 6"	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	MC	8 11 9	20		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	
7	MC	11 11 12	23	SC	No fine gravel, trace subrounded to angular FILL?						
9	SPT	8 11 15	26		Very dark greenish gray (Gley 1 3/1), no medium sand, increase fine content						
12					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"	CL							
17					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								
24	SPT	36 50/3"	50/3"	SC	Dark yellowish brown (10YR 4/6) with white lenses				26		
29	SPT262	22 50/4"	50/4"								
30					No white lenses						
31											

SLATE GEOTECHNICAL CONSULTANTS

Project No.: 18-007

Figure: A-5a

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

Log of Boring B-5

PAGE 2 OF 2

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						LEAN CLAY with SAND (CL) Yellowish brown (10YR 5/6), hard, moist, fine sand, low plasticity						
33												
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					CL							
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.

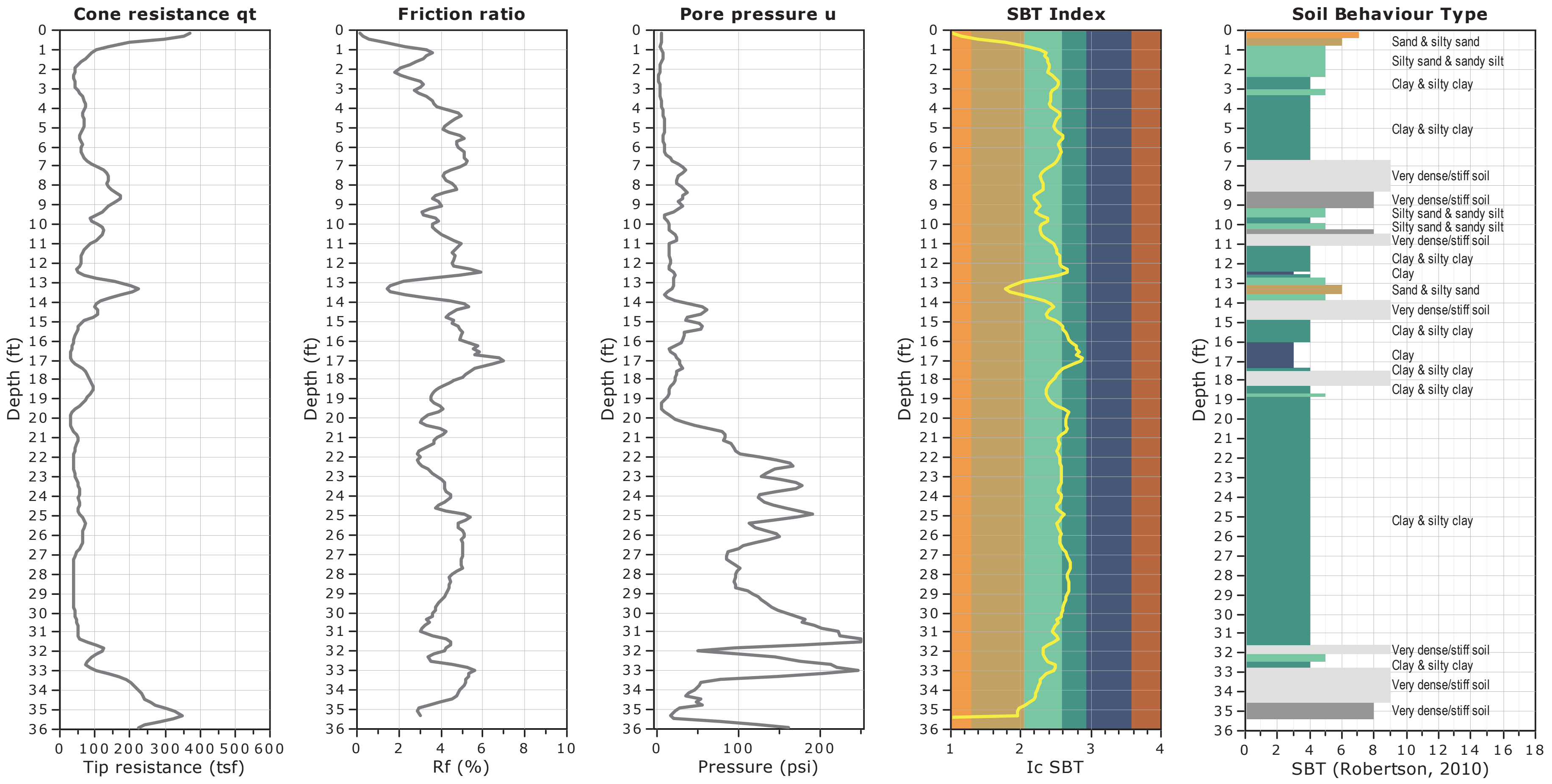
SLATE
GEOTECHNICAL CONSULTANTS

Project No.: 18-007

Figure: A-5b

ROCKRIDGE GEOTECHNICAL, 2019

DRAFT



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 36 ft, Date: 10/10/2019
Groundwater not Measured
Cone Operator: Middle Earth Geo Testing, Inc.

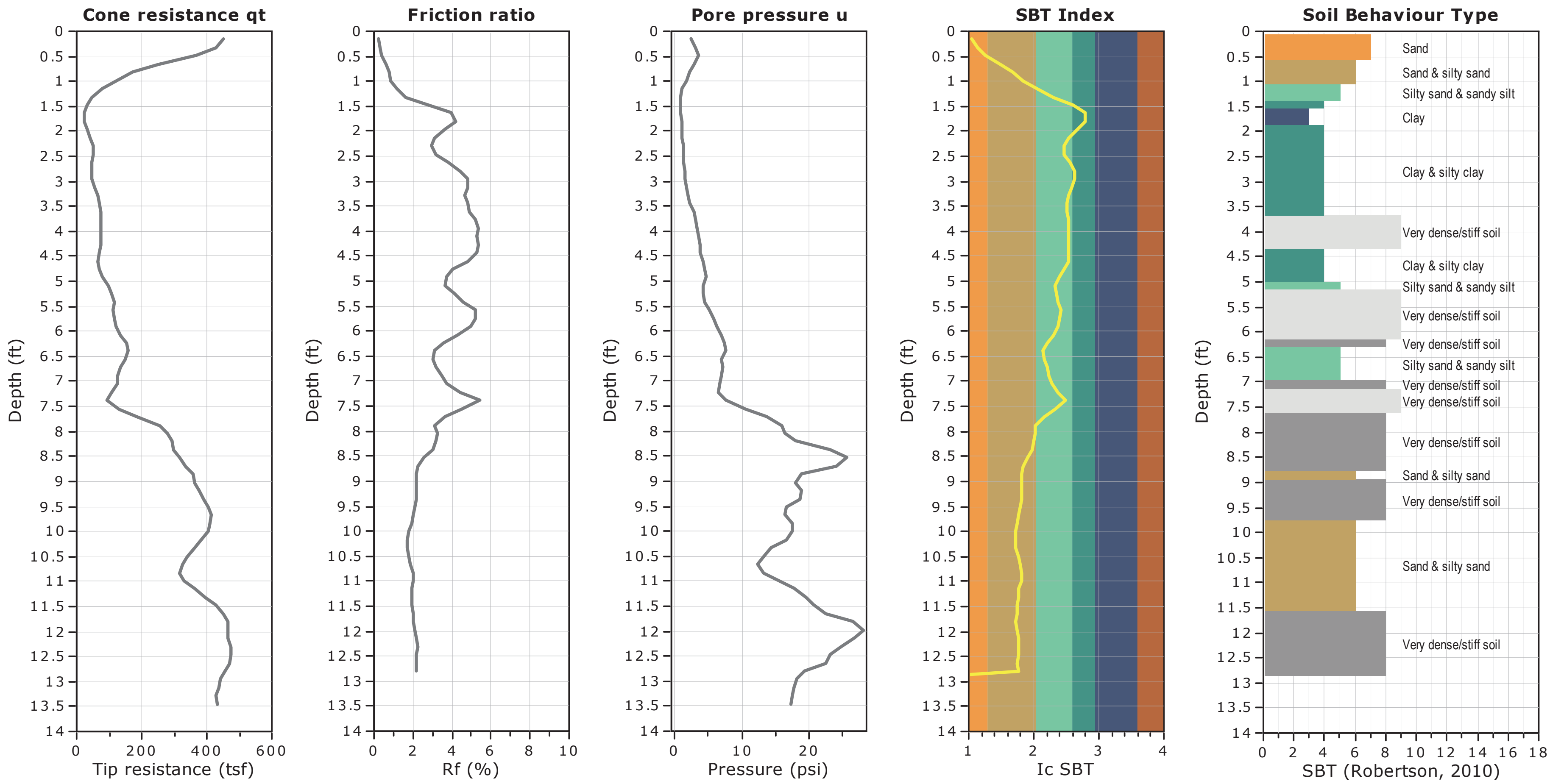
PROPOSED FACULTY AND STAFF HOUSING
JEFFERSON UNION HIGH SCHOOL DISTRICT
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 10/30/19	Project No. 19-1753	Figure B-1

DRAFT



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 13.5 ft, Date: 10/10/2019
Groundwater not Measured
Cone Operator: Middle Earth Geo Testing, Inc.

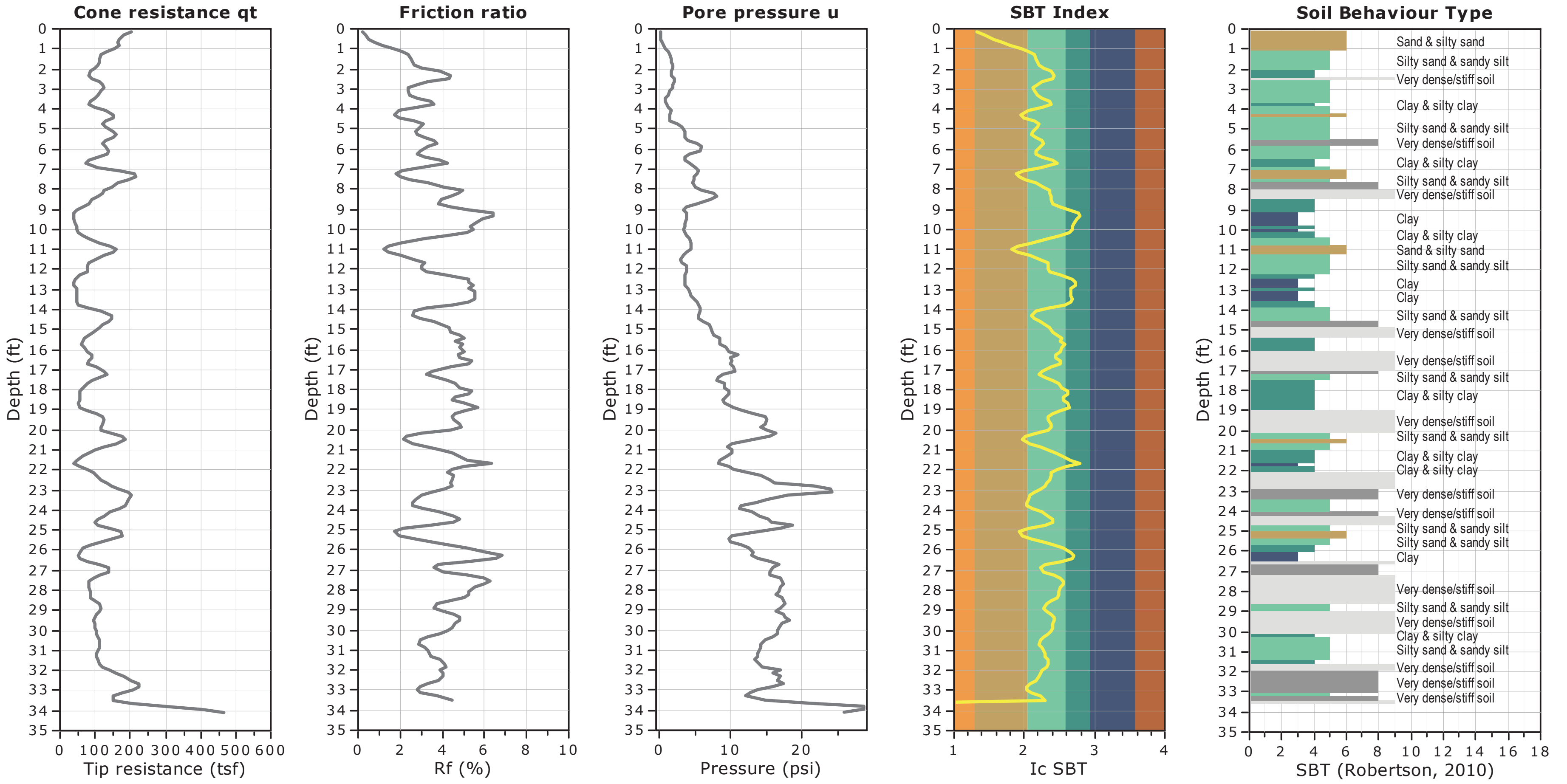
PROPOSED FACULTY AND STAFF HOUSING
JEFFERSON UNION HIGH SCHOOL DISTRICT
Daly City, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 10/30/19	Project No. 19-1753	Figure B-2

DRAFT



SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Total depth: 34 ft, Date: 10/10/2019
Groundwater not Measured
Cone Operator: Middle Earth Geo Testing, Inc.

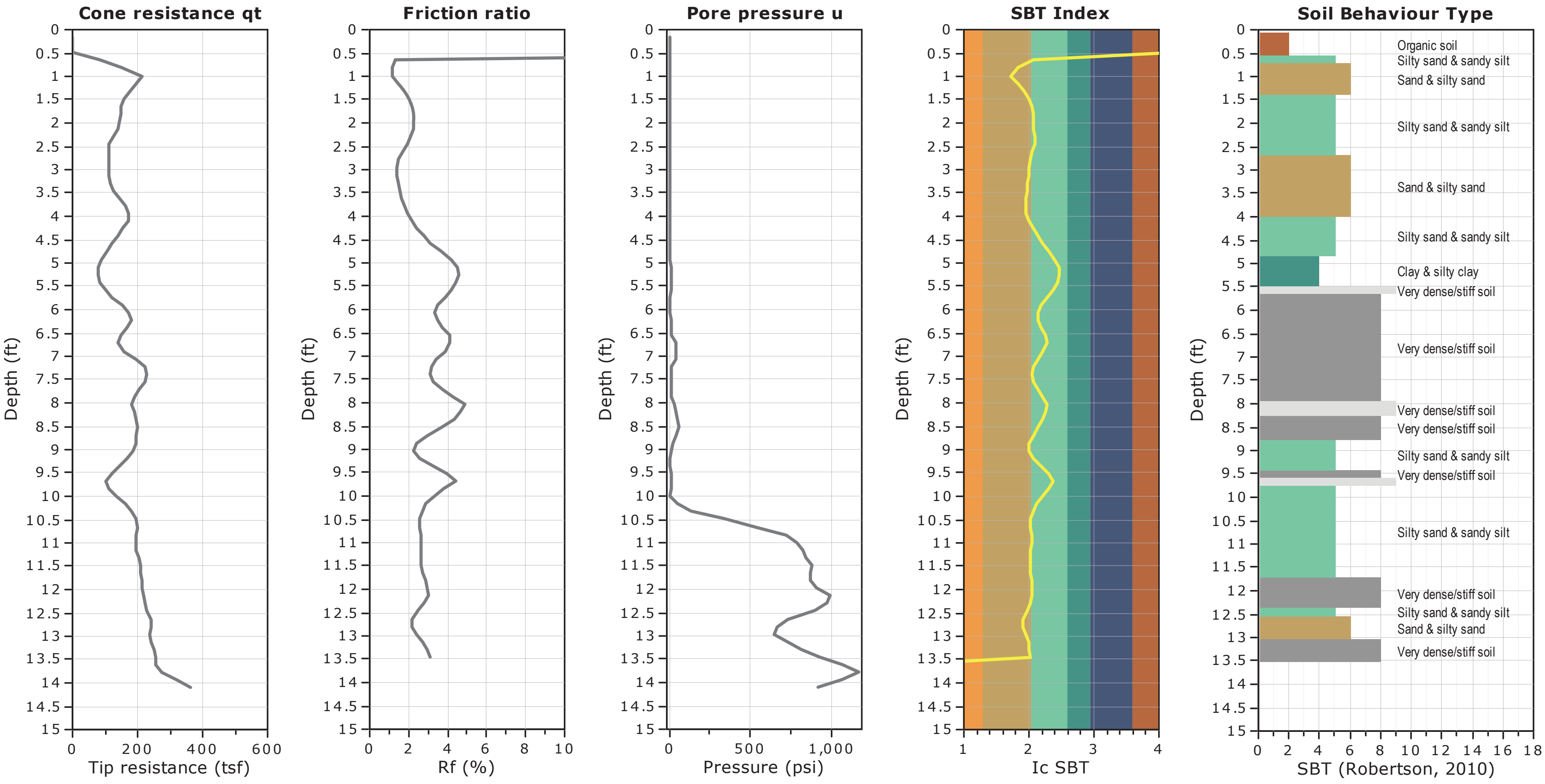
PROPOSED FACULTY AND STAFF HOUSING
JEFFERSON UNION HIGH SCHOOL DISTRICT
Daly City, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 10/30/19	Project No. 19-1753	Figure B-3

DRAFT



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 14 ft, Date: 10/10/2019
Groundwater not Measured
Cone Operator: Middle Earth Geo Testing, Inc.

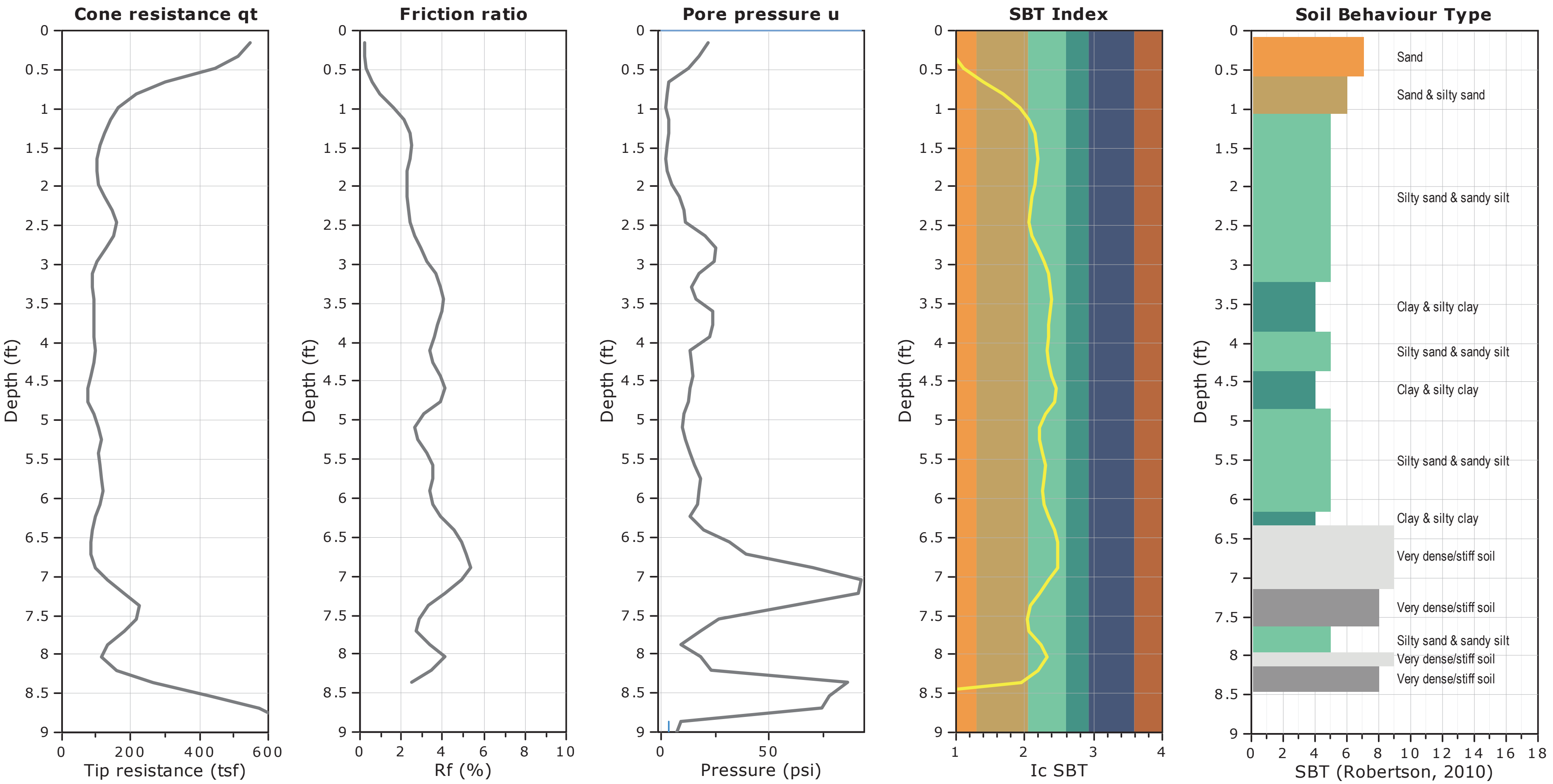
PROPOSED FACULTY AND STAFF HOUSING
JEFFERSON UNION HIGH SCHOOL DISTRICT
Daly City, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 10/30/19	Project No. 19-1753	Figure B-4

DRAFT



SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Total depth: 9 ft, Date: 10/10/2019
Groundwater not Measured
Cone Operator: Middle Earth Geo Testing, Inc.

PROPOSED FACULTY AND STAFF HOUSING
JEFFERSON UNION HIGH SCHOOL DISTRICT
Daly City, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 10/30/19	Project No. 19-1753	Figure B-5