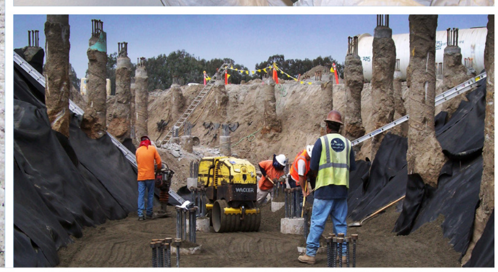
Geotechnical Evaluation

Job Training Center
2535 Pulgas Avenue
East Palo Alto, California

Sycamore Real Estate Investments

2555 Pulgas Avenue, Building A | East Palo Alto, California 94303

May 28, 2020 | Project No. 403645001



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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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May 28, 2020
Project No. 403645001

Mr. Lorenzo Brooks
Sycamore Real Estate Investments
2555 Pulgas Avenue, Building A
East Palo Alto, California 94303

Subject: Geotechnical Evaluation
Job Training Center
2535 Pulgas Avenue
East Palo Alto, California

Dear Mr. Brooks:

In accordance with your authorization, Ninyo & Moore performed a geotechnical evaluation for the design and construction of Project Thunder, a proposed job training center, on a 4-acre lot at 2535 Pulgas Avenue in East Palo Alto, California. Ninyo & Moore previously performed a geotechnical evaluation on the subject property and the adjacent parcel to the south for a job training center at 2519 Pulgas Avenue. This report presents the findings and conclusions from our previous evaluation, and our geotechnical recommendations for the job training center and related improvements now proposed for 2535 Pulgas Avenue.

As an integral part of our role as the geotechnical engineer-of-record, we request the opportunity to review the construction plans before they go to bid and to provide follow-up construction observation and testing services.

Ninyo & Moore appreciates the opportunity to be of service to you on this project.

Sincerely,
NINYO & MOORE

Gerardo Lopez, EIT
Senior Staff Engineer

Peter Connolly, PE, GE
Principal Engineer

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1 INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation for a proposed job training center on a 4-acre lot at 2535 Pulgas Avenue in East Palo Alto, California (Figure 1). This report presents the findings and conclusions from our evaluation, and our geotechnical recommendations for design and construction of the proposed improvements.

2 SCOPE OF SERVICES

Our scope of services consisted of the following:

- Review of readily available geologic and seismic literature pertinent to the project area including geologic maps and reports, regional fault maps, and seismic hazard maps.
- Performance of a site reconnaissance to observe the general site conditions and to mark the proposed locations for subsurface exploration.
- Coordination with Underground Service Alert to locate the underground utilities in the vicinity of the proposed exploratory boring.
- Performance of a private utility survey to further evaluate the exploration locations for conflicts with underground utilities.
- Procurement of a boring permit from San Mateo County Environmental Health Services.
- Subsurface exploration consisting of five hollow-stem auger borings, and six cone penetrometer test (CPT) soundings. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk soil samples for laboratory testing.
- Performance of percolation testing at one location to evaluate the infiltration characteristics of the near-surface soil for design of a storm water management system.
- Performance of a geophysical survey utilizing MAM techniques to evaluate subsurface variations in shear wave velocity.
- Laboratory testing on selected soil samples to evaluate in-place soil moisture content and density, grain size distribution, fines content, Atterberg limits, expansion index, consolidation characteristics, soil corrosivity, shear strength, and compressive strength.
- Compilation and engineering analysis of the field and laboratory data, and the findings from our background review.
- Preparation of this report presenting the findings and conclusions from our evaluation, and our geotechnical recommendations for design and construction of the project.

Ninyo & Moore previously performed a Phase I and Phase II Environmental Site Assessment for the site (Ninyo & Moore, 2019 & 2020).

3 SITE DESCRIPTION

The site consists of one rectangular parcel at 2535 Pulgas Avenue that covers approximately 4 acres. The site is bounded to the east by Pulgas Avenue, to the north by an undeveloped parcel and a light industrial property, to the west by commercial yards, and to the south by an undeveloped property (Figure 2). The site is currently developed as a yard for a trucking company with a few small buildings and paved areas for storage. The ground elevation on site ranges between approximately 12 feet above mean sea level (MSL) at the southwestern corner to about 9 feet above MSL in the northeastern quadrant of the site (Google, 2019) with an overall average gradient of approximately ½ percent across the site down to the northwest although large portions of the site, including the areas along the northern margin of the site, are flat or slope down to the southwest. The grade on site is generally consistent with the grade on the adjacent parcels and streets.

4 PROJECT DESCRIPTION

The new job training center will consist of a 4-story building with a footprint area of approximately 25,000 square feet constructed within a foot or two of the existing grade. The building will be located near the eastern edge of the site (Figure 2). Ancillary project improvements may consist of an 8,000-square foot carpentry space adjacent to the northeast corner of the proposed building, a 2,500-square foot play area adjacent to the southwest corner of the building, surface parking with double stackers, and a transformer.

5 FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance and subsurface exploration that consisted of five borings, six CPT soundings, one percolation test, and a geophysical survey. The approximate locations of the borings and soundings are shown on Figure 2. Prior to commencing the subsurface exploration, USA was notified for field marking of the existing utilities and a drilling permit was obtained from San Mateo County Health Services. A private utility survey by electro-magnetic scanning was performed and the exploration locations were initially hand-excavated to a depth of about 5 feet to check for underground utilities.

Borings B-4 and B-5 were drilled on November 11, 2019. Borings B-1, B-2, and B-3 were drilled on November 12, 2019. The borings were drilled with hollow stem auger to depths of up to approximately 50 feet below the ground surface. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings, and collected bulk and relatively undisturbed soil samples from the borings. The samples were then transported to our geotechnical laboratory for testing. The borings were backfilled in accordance with the boring permit requirements shortly after drilling. Detailed logs of the borings are presented in Appendix A.

The excavated soil generated during the drilling was collected in drums left on site. Soil samples collected from the drums were analytically tested for waste characterization. The results of the analytical testing are reported under separate cover.

The CPT soundings were performed on November 11, 2019 and November 26, 2019 using a truck-mounted rig with a 20-ton reaction capacity. After hand excavation to a depth of 5 feet to check for underground utilities, the soundings were pushed to depths of up to approximately 101 feet below the ground surface. Cone tip resistance, sleeve friction, and pore pressure were electronically measured and recorded at vertical intervals of approximately 2 inches while the cone was advanced. The normalized soil behavior type (Qtn) and soil behavior type index (I_c) and corresponding soil behavior for the subsurface materials encountered was assessed using correlations (Robertson, 2009 & 1990, respectively) based on the cone penetration data and sleeve friction. The CPT sounding logs are presented in Appendix B.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content and density, particle size distribution, Atterberg limits, expansion index, fines content, direct shear strength, triaxial shear strength, consolidation characteristics, soil corrosivity, and unconfined compressive strength. The results of the in-situ moisture content and density tests are presented on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix C.

A percolation test was performed on November 22, 2019 at the location shown on Figure 2. The percolation test results and procedures utilized are presented in Appendix D. The test hole was backfilled with the soil cuttings after testing.

A seismic survey using passive surface wave techniques was performed at the site on November 22, 2019. The purpose of the study was to evaluate seismic site characterization and the variation in shear wave velocity with depth for the subsurface materials. The passive source method included Microtremor Array Measurement (MAM) and consisted of one linear profile of seismic data collection. The passive source method provided a shear wave (S-wave) velocity profile to a depth of approximately 100 feet below the ground surface and the weighted average of the shear wave velocity over that interval (Vs100) for seismic site classification (CBSC, 2019). The location of the seismic survey line is noted on Figure 2. The seismic study results are provided in Appendix E along with a summary of the field methods and analytical procedures utilized. The results indicate that the characteristic Vs100 is approximately 1,246 feet per second with a corresponding seismic site classification of Class C.

6 GEOLOGIC AND SUBSURFACE CONDITIONS

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions are provided in the following sections.

6.1 Regional Geologic Setting

The site is located along the western margin of San Francisco Bay in the Coast Ranges Geomorphic Province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

6.2 Site Geology

Regional mapping by Dibblee & Minch (2007) indicates that the site is underlain by alluvial fan deposits of Holocene age consisting of fine-grained sand, silt, and gravel (Figure 4). Regional mapping by Brabb et al., (1998 & 2000) indicate that the site is underlain by basin deposits of Holocene age that are found at the distal edges of alluvial fans and consist of silty clay to clay.

6.3 Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation. More detailed descriptions are presented on the logs in Appendix A.

6.3.1 Pavement

Borings B-4 and B-5 were drilled through asphalt concrete pavement. The pavement section encountered in these borings consisted of approximately 4 to 4½ inches of asphalt concrete over approximately 2 to 8 inches of aggregate base. Variations in the thickness of the asphalt concrete and aggregate base layers, within and beyond the ranges observed, may be encountered due to past maintenance, utility work, or other factors.

6.3.2 Fill

Fill was encountered in the borings below the pavement section, where present, or from the ground surface to depths that ranged between approximately 1½ feet (Boring B-3) and 6 feet (Boring B-5). The fill, as encountered, generally consisted of brown to dark brown, and olive gray to black, moist, firm to stiff, lean to sandy clay.

6.3.3 Alluvium

Alluvium was encountered in the borings below the fill to the depths explored. The alluvium, as encountered, generally consisted of brown and yellowish brown, moist to wet, firm to hard, lean to sandy clay and fat clay with layers of very loose to very dense sand and clayey sand.

6.4 Groundwater

Groundwater was encountered in the borings during drilling at depths that ranged between approximately 6½ feet (Boring B-2) and 8 feet (Boring B-4) below the ground surface. Groundwater was measured to range between approximately 7½ feet (Boring B-3) and 12 feet (Boring B-4) below the ground surface about 15 minutes after drilling. Groundwater may rise to a higher elevation than was encountered in our exploratory borings due to the short time available for seepage of water into the borings. Based on pore pressure measurements collected during cone penetration testing, the depth to groundwater was estimated to range between approximately 4.7 feet (Sounding CPT-3) and 8.4 feet (Sounding CPT-4) below the ground surface at the time of testing. The groundwater levels estimated from the cone penetration testing correspond to elevations that range between approximately 4 and 5 feet above mean sea level. Regional records indicate that the historic high groundwater levels in the site vicinity are less than 10 feet below the ground surface (CGS, 2006a).

The depth to groundwater within the limits of the study area is subject to spatial variations in topography and the elevation of the phreatic surface. Furthermore, groundwater levels may fluctuate in response to seasonal variations in precipitation, nearby groundwater pumping or dewatering, changes in irrigation practices adjacent to or within the study area, or other factors. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Piezometers can be installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels over time.

7 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considered a number of issues relevant to the proposed construction, including seismic hazards, landsliding, settlement of compressible soil layers from static loading, unsuitable materials, excavation considerations, infiltration characteristics, soil corrosivity, and expansive soils. These issues are discussed in the following subsections.

7.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, dynamic settlement, lateral spreading, and sand boil induced ground subsidence. These potential hazards are discussed in the following subsections.

7.1.1 Historical Seismicity

The site is located in a seismically active region. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Records of historic ground effects related to seismic activity compiled by Knudsen et al. (2000), indicate that the water level in a monitoring well about 1,500 feet from the site to the southwest rose approximately 1½ feet in response to the 1989 Loma Prieta earthquake (Tinsley et al., 1998). No other ground effects related to historic seismic activity (e.g. liquefaction, sand boils, lateral spreading, ground cracking) have been reported for the site vicinity.

7.1.2 Faulting and Ground Surface Rupture

There are numerous recognized faults in northern California. Selected characteristics, as evaluated by the Working Group on California Earthquake Probabilities (WGCEP, 2013), for recognized and postulated faults (Caltrans, 2019) near the site are presented in Table 1. The fault characteristics in the table are presented in order of decreasing peak ground acceleration (PGA) based on a deterministic seismic hazard analysis utilizing the Chiou & Youngs (2013) and Campbell & Bozorgnia (2013) attenuation relationships.

The site is not located within an Alquist-Priolo Earthquake Fault Zone established by the state geologist (CGS, 1974) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,000 years (CGS, 2018). The closest fault rupture hazard zone is associated with the San Andreas Fault and is approximately 7 miles from the site to the southwest (CGS, 1974).

Table 1 – Parameters for Nearby Faults				
Fault (Segment)	**ID**	**Type**	**Max Moment Magnitude**	**Distance to Site (kilometers)**
San Andreas (Peninsula)	134	Strike Slip	8.0	13.0
Silver Creek	148	Strike Slip	6.9	10.4
San Andreas (Santa Cruz Mts)	158	Strike Slip	8.0	22.6
Cascade Fault	153	Reverse	6.7	10.4
Hayward (South)	137	Strike Slip	7.3	16.6
Monte Vista Shannon	154	Reverse	6.4	11.1
San Gregorio (San Gregorio)	127	Strike Slip	7.4	28.1
Hayward (Southern Extension)	149	Strike Slip	6.7	19.6
San Andreas (North Coast)	80	Strike Slip	8.0	55.9
Hayward (North)	123	Strike Slip	7.3	29.7

Based on our review of the referenced geologic maps, known active faults are not mapped on the site and the site is not located within a fault-rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

7.1.3 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. Seismic design criteria to address ground shaking are provided in Section 9.1. A site-specific ground motion hazard analysis was performed in accordance with Chapter 21 of the American Society of Civil Engineers (ASCE) Standard 7-16 to evaluate the peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) in accordance with the 2019 California Building Code (CBC). The results of our site-specific ground motion hazard analysis indicate that the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) is 0.616g. The assumptions and models utilized for this analysis are listed on Figure 5.

7.1.4 Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface.

Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface. The seismic hazard zones for the site vicinity are presented on Figure 6. Regional studies of liquefaction susceptibility (Witter et al., 2006) indicate that the liquefaction susceptibility at the site is very high.

During our subsurface exploration, we encountered sand below the groundwater level. To further evaluate the potential for liquefaction, we performed an analysis in accordance with the method presented by Boulanger and Idriss (2014) using the CPT data collected during our subsurface exploration and the computer program CLiq (GeoLogismiki, 2018). Our analysis considered a PGA of 0.616g corresponding to a Magnitude 8 earthquake on the San Andreas fault and a groundwater level of 7 feet below the existing ground surface. Based on characteristics provided by Bray & Sancio (2006) and the result of our laboratory index testing, the fine-grained soil (silt and clay) encountered at the site is not consistent with soil considered to be susceptible to liquefaction. Therefore, soil with a behavior type index (Ic) of 2.4 or less, consistent with sand and silty sand, was evaluated for susceptibility to liquefaction and related hazards. The results of our analysis, presented in Appendix F, indicate, based on a safety factor against liquefaction of less than one, that thin layers of sand and silty sand between approximately 25 and 38 feet below the ground surface will liquefy under the considered ground motion along with a few, very thin, scattered layers between 7 and 25 feet. Based on the distribution and relative thickness of the liquefiable layers, we do not regard reduction in foundation bearing capacity due to liquefaction as a design consideration for shallow foundations. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading, are addressed in the following sections.

The cohesive soils encountered during our subsurface exploration are not particularly sensitive based on the observed moisture content and estimates of undrained and remolded shear strength from CPT tip resistance and sleeve friction, respectively, below the depth of hand excavation. As such we do not regard seismically induced strain-softening behavior as a design consideration.

7.1.5 Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

We evaluated the potential for dynamic settlement due to liquefaction of saturated soil using the computer program CLiq (GeoLogismiki, 2018) to evaluate the CPT data collected during our field investigation with the methodology of Boulanger and Idriss (2014). Our analysis considered a Magnitude 8.0 earthquake producing a PGA of 0.616g and a groundwater level of 7 feet below the ground surface. The results of our analysis, presented in Appendix F, indicate that the free-field total dynamic settlement following the considered seismic event will be approximately 2 inches. Differential dynamic settlement is estimated to be about 1 inch over a horizontal distance of approximately 30 feet. Recommendations for shallow foundations are provided.

7.1.6 Sand Boil Induced Ground Subsidence

Sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in ground subsidence due to loss of material that is in addition to dynamic settlement. The Liquefaction Potential Index (LPI) described by Iwasaki et al (1978) was computed from the results of our liquefaction analysis with the CPT data to evaluate the potential for surface manifestation of liquefaction such as sand boils. The computed values of the LPI, presented in Appendix F, indicate that the potential for surface manifestation of liquefaction or sand boils is low with an LPI of approximately 5 or less.

7.1.7 Lateral Spreading

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. Lateral spreading can occur on sloping ground or on flat ground adjacent to an exposed face. A free-face condition does not exist near the proposed improvements and the ground slope on site is relatively gentle and inconsistent with areas of flat ground or a reversed gradient. Consequently, we do not regard lateral spreading as a design consideration.

7.2 Landsliding and Slope Stability

The site is relatively flat with little topographic variation and the proposed project does not include the construction significant slopes. Based on the existing topography, we do not regard slope stability or landsliding of existing slopes as a design consideration for this project.

7.3 Static Settlement

The findings from our subsurface exploration indicate that the site is generally underlain by firm to hard clay with thin layers of very loose to very dense sand. Static settlement may be a concern for structures supported on shallow footings where the sustained loads are moderate. Recommendations for shallow footings are provided along with recommendations for ground improvement to mitigate static settlement where a reduction in the estimated static settlement is desired. Alternative recommendations for mat foundations to mitigate static settlement are also provided.

7.4 Unsuitable Materials

Fill materials that were not placed and compacted under the observation of a geotechnical engineer, or fill materials lacking documentation of such observation, are considered undocumented fill. Undocumented fill is generally unsuitable as a bearing material below foundations due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill was encountered in the borings to depths that ranged between approximately 1½ feet (Boring B-3) and 6 feet (Boring B-5) below the ground surface. The depth of fill may vary within and beyond the observed range due to past grading activity. Recommendations for subgrade observation and remedial grading are provided to check foundation excavations for unsuitable materials and mitigate poor bearing conditions related to undocumented fill. Alternatively, ground improvement to mitigate static settlement under foundations can also mitigate poor or variable bearing conditions related to undocumented fill.

Soil containing roots or other organic matter are not suitable as fill or subgrade material below foundations, pavements, or engineered fill. Recommendations for clearing and grubbing to remove vegetative matter in soil during site preparation are provided.

7.5 Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, electrical resistivity, chloride content, and soluble sulfate content was performed on samples of the near surface soil. The results of the corrosivity tests are presented in Appendix C. California Department of Transportation (Caltrans) defines a corrosive environment for structural elements as an area where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.15 percent (1,500 ppm) or more, or pH of 5.5 or less (Caltrans, 2018). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 2. Based on these criteria and the results of our testing, the near-surface soil at the site meets the definition of a corrosive environment for structures, but the sulfate exposure to concrete is negligible, and the exposure classification for sulfate is S0. Recommendations to mitigate the impact of corrosive/deleterious soil on concrete structures are presented in Section 9.8.

Table 2 – Criteria for Deleterious Soil on Concrete		
Sulfate Content Percent by Weight	**Sulfate Exposure**	**Exposure Class**
0.0 to 0.1	Negligible	S0
0.1 to 0.2	Moderate	S1
0.2 to 2.0	Severe	S2
> 2.0	Very Severe	S3

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 (ACI, 2016)

7.6 Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing was performed on select samples of the near-surface soil to evaluate the expansion index. The tests were performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing indicate that the expansion index of two samples tested is 20 and 45, which is consistent with a low to very low expansion characteristic.

7.7 Excavation Considerations

We anticipate that the proposed project will involve excavations of up to approximately 9 feet deep for podium level excavation, foundation construction, and utility installation. The geologic materials encountered during our subsurface evaluation over this interval included fill and alluvium consisting of moist to wet, firm to very stiff clay with layers of very loose to loose clayey sand. The findings from our subsurface exploration indicate that the conditions encountered below this interval, if deeper excavations are needed for ground improvement, consisted of stiff to hard clay with layers of loose to very dense sand and clayey sand.

We anticipate that heavy earthmoving or drilling equipment in good working condition should be able to make the proposed excavations. Excavations in the fill may encounter obstructions consisting of debris, rubble, abandoned structures, or over-sized materials that may require special handling or demolition equipment for removal.

Near-vertical cuts in these deposits may not be stable particularly if the excavation encounters seepage or granular soil, extends below or near groundwater, or is exposed to rainfall/runoff.

Groundwater was encountered in the borings during drilling at depths that ranged between approximately 6½ feet (Boring B-2) and 8 feet (Boring B-4) below the ground surface. Based on pore pressure measurements collected during cone penetration testing, the depth to groundwater was estimated to range between approximately 4.7 feet (Sounding CPT-3) and 8.4 feet (Sounding CPT-4) below the ground surface at the time of testing. Variations in groundwater levels within and outside this range should be anticipated. Excavation subgrade that is near or below groundwater may be unstable under construction loading. Excavation subgrade may become unstable if exposed to wet conditions. Recommendations for excavation stabilization are presented. Excavated materials may also be wet and need to be dried out before reuse as fill.

7.8 Infiltration Characteristics

Ninyo & Moore performed percolation testing to evaluate the rate of infiltration on site for design of storm water management systems. The percolation test procedures utilized are presented in Appendix D. The test results, presented in Appendix D and summarized in Table 3, indicate that the infiltration rate of the near surface soil on site is relatively fast and consistent with Hydrologic Soil Group A. Due to the variability of subsurface materials encountered during our exploration, variability in subsurface infiltration should be anticipated.

Table 3 – Percolation Test Results

Test	Test Depth (feet)	Subsurface Conditions	Percolation Rate (minutes/inch)	Infiltration Rate[1] (inch/hour)
P-1	2	Clayey Sand	30	0.84

[1] Infiltration rate is percolation rate adjusted by a reduction factor to exclude percolation through sides of test hole.

8 CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into the design and construction of the subject project. The conclusions from our evaluation are as follows:

- The subsurface exploration for this study encountered fill and alluvium. The fill, as encountered, generally consisted of moist, firm to stiff, lean to sandy clay. The alluvium, as encountered, generally consisted of moist to wet, firm to hard, lean to sandy clay and fat clay with layers of very loose to very dense sand and clayey sand.

- The fill encountered in the borings extended to depths that ranged between approximately 1½ feet (Boring B-3) and 6 feet (Boring B-5) below the ground surface. The fill is undocumented. Recommendations for subgrade observation and remedial grading are provided to mitigate the potential for unsuitable materials and poor bearing conditions related to undocumented fill. Alternatively, poor or variable bearing conditions related to undocumented fill can also be mitigated by ground improvement under foundations.

- Groundwater was encountered in the borings during drilling at depths that ranged between approximately 6½ feet (Boring B-2) and 8 feet (Boring B-4) below the ground surface. Based on pore pressure measurements collected during cone penetration testing, the depth to groundwater was estimated to range between approximately 4.7 feet (Sounding CPT-3) and 8.4 feet (Sounding CPT-4) below the ground surface at the time of testing. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 6.4.

- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault.

- The results of our liquefaction analysis, presented in Appendix F, indicate that thin layers of sand and silty sand between approximately 25 and 38 feet below the ground surface will liquefy under the considered ground motion along with a few, very thin, scattered layers between 7 and 25 feet. Based on the distribution and relative thickness of the liquefiable layers, we do not regard reduction in foundation bearing capacity due to liquefaction as a design consideration for shallow foundations. Computed values of the Liquefaction Potential Index, presented in Appendix F, indicate that the potential for surface manifestation of liquefaction or sand boils is low.

- The results of our dynamic settlement analysis, presented in Appendix F, indicate that the free-field total dynamic settlement following the considered seismic event will be approximately 2 inches. Differential dynamic settlement is estimated to be about 1 inch over a horizontal distance of approximately 30 feet.

- Ground surface rupture due to faulting is not a design consideration based on the location of the project.

- Landslides and lateral spreading due to liquefaction are not design considerations based on the topographic conditions at the site.

- Static settlement may be a concern for structures supported on shallow footings where the sustained loads are moderate. Recommendations for footings are provided with ground improvement to mitigate static settlement where desirable. Alternative recommendations for mat foundations are also provided.

- Expansion Index testing indicates that the expansion characteristic of the near-surface soil on site has is low to very low.

- Our laboratory corrosion testing indicates that the near-surface site soils are considered corrosive to structures based on California Department of Transportation (Caltrans, 2018) corrosion guidelines. Recommendations for measures to mitigate the impact of corrosive/deleterious soil on concrete structures are presented.

- Percolation testing performed for this study indicate that the infiltration rate at the Test Hole (Figure 2) is relatively fast.

- Excavations that remain unsupported, encounter seepage or granular soil, extend below or near groundwater, or are exposed to water may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

- Excavations in the fill may encounter debris, rubble, oversize material, buried objects, or other potential obstructions.

9 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

9.1 Seismic Design Criteria

Ninyo & Moore performed a site-specific ground motion analysis in accordance with the procedure in Chapter 21 of ASCE Standard 7-16. The assumptions and models for this analysis are noted on Figure 4 and are listed in the references. Seismic Site Class C was selected based on the findings from our subsurface exploration presuming that the fundamental period of the proposed structure will not exceed ½ second. The design response spectrum based on the site-specific ground motion analysis is presented on Figure 5 and the corresponding seismic design criteria are summarized in Table 4. The spectral ordinates and seismic coefficients based on the mapped values of the risk-targeted spectral response acceleration, consistent with Section 11.4 of ASCE Standard 7-16, are also presented in the table (SEAOC & OSHPD, 2019). Either the site-specific or the general seismic criteria listed in Table 4 may be used for design as the site-specific ground motion analysis is optional for this site.

Table 4 – California Building Code Seismic Design Criteria

Seismic Design Parameter Evaluated for 37.4747° North Latitude, 122.1328°West Longitude	Site Specific	Section 11.4 ASCE 7-16
Site Class	C	C
Site Coefficient, Fa	---	1.2
Site Coefficient, Fv	---	1.4
Mapped Spectral Response Acceleration at 0.2-second period, S_S	---	1.500g
Mapped Spectral Response Acceleration at 1.0-second period, S_1	---	0.600g
Site-Adjusted Spectral Acceleration at 0.2-second period, S_{MS}	1.620g	1.800g
Site-Adjusted Spectral Acceleration at 1.0-second period, S_{M1}	1.302g	0.840g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.080g	1.200g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.868g	0.560g
Seismic Design Category for Risk Category I, II, or III	D	D

9.2 Foundation Recommendations

The proposed job training center may be supported on footings or mat foundations. Recommendations for footings and mat foundations are provided below. Ground improvement may be performed to reduce the degree of static settlement. Recommendations for ground improvement are provided in Section 9.4.

Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures. The foundation design parameters provided in the following sections are not intended to preclude differential movement of foundations. Minor cracking (considered tolerable) of foundations may occur.

9.2.1 Mat Foundations

The job training center may be supported on a mat foundation designed for a gross allowable bearing capacity of 1,500 pounds per square foot (psf). This allowable bearing capacity includes a factor of safety of more than 3 and may be increased by one-third when considering wind or seismic loading combinations.

Mat foundations should be designed for a total settlement of 1⅓-inch due to sustained loads and a differential settlement of ⅔-inch over a 20-foot span. The deflection of the mat due to applied loads may be evaluated using a modulus of subgrade reaction equivalent to 3 pounds per cubic inch for sustained loads. Mat foundations may undergo an additional 2 inches of total dynamic settlement following the seismic event considered with a differential dynamic settlement of approximately 1 inch over a horizontal distance of about 30 feet. Mat foundation subgrade should be prepared in accordance with the recommendations in Section 9.5.5. The geotechnical engineer should observe mat foundation subgrade to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

The mat slab should be no less than 10 inches thick and should be reinforced with deformed steel bars that have a nominal diameter of ½ inch or more. The mat slab and slab reinforcement should be designed and detailed by the structural engineer based on the anticipate loading and usage. Masonry briquettes or plastic chairs should be used to aid in the correct placement of slab reinforcement. Recommendations for concrete and concrete cover over reinforcing steel are presented in Section 9.8. Recommendations for a moisture vapor retarding system to reduce the potential for moisture vapor intrusion through the mat foundation are provided in Section 9.9.

A friction coefficient of 0.20 and an allowable lateral bearing pressure of 225 psf per foot of depth up to 2,250 psf may be used to evaluate foundation resistance to lateral loads with a safety factor of 2. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.2.2 Footings

Footings bearing on subgrade prepared per the recommendations in Section 9.5.5 may be designed using the criteria listed in Table 5. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Table 5 – Recommended Bearing Design Parameters for Footings

Footing[1]	Sustained Loads	Footing Widths	Bearing Depth[2]	Allowable Bearing Capacity[3]	Static Settlement
Wall Footing	10 kips/foot or less	12 inches or more	2 feet or more	2,000 psf	2 inches total 1 inch differential over 30 feet
Column Footing	200 kips or less	24 inches or more	2 feet or more	2,000 psf	2½ inches total 1¼ inch differential over 30 feet

Notes:
1 Podium floor within a foot or two of existing grade.
2 Below the lowest adjacent finish grade.
3 Net allowable bearing capacity in pounds per square foot with Safety Factor of 3 or more. Allowable bearing capacity may be increased by one-third for wind or seismic alternative basic load combinations.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 5 for sustained loads. Structures may undergo an additional 2 inches of total dynamic settlement following a significant earthquake with a differential dynamic settlement of about 1 inch over a lateral span of 30 feet. Footing settlement due to sustained static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction in pounds per cubic inch (pci) are provided in Table 6. The designer may interpolate between the values in the table for intermediate footing widths.

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle above the bottom edge of the footing. Footings should be deepened or excavation depths reduced as-needed. Footing bottoms should not be sloped more than 1-unit vertical to 10 units horizontal. Wall footings may be stepped provided that the bearing grade differential between adjacent steps does not exceed 18 inches and the slope of a series of such steps does not exceed 1-unit vertical to 2 units horizontal.

Table 6 – Footing Modulus of Subgrade Reaction

Footing	Footing Width				
	1 foot	2 feet	4 feet	7.5 feet	12.5 feet
Wall Footing	37 pci	17 pci	8 pci	4 pci	--
Column Footing	--	19 pci	10 pci	5.5 pci	3.5 pci

A friction coefficient of 0.35 and an allowable lateral bearing pressure of 225 psf per foot of depth up to 2,250 psf may be used to evaluate footing resistance to lateral loads with a safety factor of 2. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. The weight of the material above a plane rising up and away from the bottom edges of the footings at 20 degrees off plumb may be considered, along with the weight of the footing and the material over the footing, when evaluating footing resistance to uplift. A unit weight of 120 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation.

9.2.3 Slab-on-Grade Floors

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. Slabs subject to vehicular traffic should be no less than 6 inches thick for traffic consisting predominantly of passenger vehicles with periodic emergency vehicles or garbage trucks. Floor slabs should be reinforced with deformed steel bars with a nominal diameter of ⅜-inch or more. Masonry briquettes or plastic chairs should be used to maintain the position of slab reinforcement, during concrete placement, in the upper half of the slab with appropriate concrete cover over the reinforcing steel. Refer to Section 9.8 for the recommended concrete cover over reinforcing steel. Joints consistent with ACI guidelines (ACI, 2016) may be constructed at periodic intervals to reduce the potential for random cracking of the slab. Recommendations for a moisture vapor retarding system to reduce the potential for moisture vapor intrusion through the mat foundation are provided in Section 9.9. Where a vapor retarding system is not used, slabs should be constructed on 6 inches of compacted aggregate base conforming to Sections 9.5.4 and 9.5.6. Slab subgrade should be prepared in accordance with Section 9.5.5.

9.3 Foundations for Ancillary Improvements

Lightly-loaded ancillary improvements may be supported on foundations designed and constructed in accordance with the recommendations in this section.

9.3.1 Equipment Pads

The transformer and other mechanical equipment may be supported on mat foundations. Mat foundations for equipment pads should be not less than 8 inches thick with reinforcement consisting of one or more layers of deformed steel bars (nominal diameter of ½-inch or more) at a center-to-center spacing of not more than 18 inches in both directions. Mat foundations for equipment pads should be designed and detailed by a structural engineer for the anticipated loading and usage.

Mat foundations for equipment pads should be constructed over 6 inches of aggregate base compacted to 95 percent of the reference density as evaluated by ASTM D1557. Prior to placement of the aggregate base, foundation subgrade should be scarified to a depth of about 8 inches, moisture conditioned to near and above the optimum moisture content, then compacted to 90 percent of the reference density as evaluated by ASTM D1557.

Equipment pads up to 18 feet wide consistent with these recommendations may be designed for a net allowable bearing capacity of 1,000 pounds per square foot (psf). This allowable bearing capacity, which includes a safety factor of three or more, may be increased by one-third when considering wind or seismic loading combinations. The deflection of the mat due to applied loads may be evaluated using a modulus of subgrade reaction equivalent to 5 pounds per cubic inch for sustained loads. Mat foundations may undergo an additional 2 inches of total dynamic settlement following the seismic event considered with a differential dynamic settlement of approximately 1 inch over a horizontal distance of about 30 feet. A friction coefficient of 0.50 may be used to evaluate foundation resistance to lateral loads where the slab is underlain by aggregate base with no moisture vapor retarding system.

9.3.2 Minor Footings

Play area equipment, parking stackers, site walls, and other lightly-loaded ancillary improvements may be supported on footings. Footings 12- to 36-inches wide on level ground embedded 12 inches below the adjacent grade and bearing on firm or compact subgrade may be designed for a net allowable bearing capacity of 1,500 pounds per square foot. The allowable bearing capacity may be increased by one-third when considering wind or seismic load combinations.

Excavations for minor footings should be inspected. Debris, vegetation, or other deleterious matter should be removed and replaced with compacted fill per the recommendations in this report. Excavation subgrade that is loose, soft, or dry of optimum should be scarified and moisture conditioned, as needed, to achieve a moisture content near and above the optimum, before compaction, by mechanical means, to 90 percent of the reference density as evaluated by ASTM D1557.

Structures supported on footings consistent with these recommendations should be designed for a total and differential settlement due to sustained loads of approximately ½-inch and ¼ inch, respectively, over a horizontal distance of 30 feet. Minor footings may undergo an additional 2 inches of total dynamic settlement following the seismic event considered with a differential dynamic settlement of approximately 1 inch over a horizontal distance of about 30 feet.

The footings should be reinforced with deformed steel bars as detailed by the project structural engineer. A friction coefficient of 0.35 and an allowable lateral bearing pressure of 225 psf per foot of depth up to 2,250 psf may be used to evaluate footing resistance to lateral loads with a safety factor of 2. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.3 Drilled Piers

Play area equipment, parking stackers, light poles, and other lightly-loaded ancillary improvements may be supported on drilled piers as an alternative to footings. Drilled piers for ancillary improvements embedded up to 20 feet below grade may be designed for an allowable side friction of up to 500 pounds per square foot (psf) at 50 psf per foot of embedment depth to evaluate resistance to downward axial loads and up to 350 psf at 35 psf per foot depth for upward axial loads. The recommended values for allowable skin friction include a safety factor of 2 for downward loading and 3 for upward loading. The allowable side friction may be increased by one-third for alternative basic load combinations with loads of short duration such as wind or seismic loads. The spacing between adjacent piers should be equivalent to three pier diameters or more to mitigate reduction in axial resistance due to group effects. Structures supported on shallow pier foundations should be designed for a total settlement due to sustained loads of approximately ½ inch with a differential of approximately ¼ inch over a horizontal distance of 30 feet.

A lateral bearing pressure of 100 pounds per square foot (psf) per foot depth up to 1,500 psf may be used to evaluate resistance to lateral loads and overturning moments in accordance with Section 1807 of the California Building Code with a one-third increase for wind or seismic loading conditions. The allowable lateral bearing pressure may be increased by a factor of two for structures that can accommodate ½ inch of lateral deflection of the top of the pier foundation.

The spacing between adjacent piers should be equivalent to three pier diameters or more to avoid a reduction in lateral load resistance due to group effects for piers in a row perpendicular to the direction of lateral loading. For piers in a row parallel to the direction of lateral loading, the contribution of trailing piers to the lateral load resistance of the group should be neglected where the center to center spacing is less than eight pier diameters.

Drilled pier excavations should be cleaned of loose material prior to pouring concrete. Drilled pier excavations that encounter groundwater or cohesionless soil may be unstable and may need to be stabilized by temporary casing or use of drilling mud. Standing water should be removed from the pier excavation or the concrete should be delivered to the bottom of the excavation, below the water surface, by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.

9.4 Ground Improvement

Ground improvement may be performed to reduce the estimated potential settlement due to sustained static loads on foundations, mitigate concerns related to undocumented fill, and permit an increase in the allowable bearing capacity. The ground improvement program should be designed and constructed by a specialty contractor with experience utilizing the selected ground improvement technique on several previous projects with similar ground conditions. The ground improvement program should be designed to reduce the future building settlement under sustained loads to 1 inch (total) with a differential static settlement of ½ inch over a lateral distance of 20 feet. We anticipate that ground improvement by stone columns, aggregate piers, rigid inclusions, or drilled displacement grouting can achieve this objective. General recommendations and descriptions of these methods are provided in the following subsections.

9.4.1 Stone Columns and Aggregate Piers

Stone columns (or aggregate piers), consisting of crushed rock installed in a hole created by an auger, vibratory probe, or driven/pushed mandrel and compacted in lifts by a vibratory probe or rammer/tamper, may be used to reinforce the subgrade below footings and improve the average stiffness of the composite ground thereby reducing settlement and increasing the allowable bearing capacity for the footings. We anticipate that these methods can be designed to achieve an improved allowable bearing capacity of 4,000 pounds per square foot (psf). A pre-production test section should be constructed to demonstrate that the selected ground improvement technique and installation parameters can achieve the design criteria. Static load testing should be performed to evaluate the modulus of the constructed test columns/piers under loading conditions consistent with production work.

The ground improvement contractor should submit qualifications with resumes of key personal and descriptions of representative projects completed; a ground improvement design with shop drawings that describe the spacing, location, depth, and nominal diameter of the columns/piers; calculations to document the basis for the design; a work procedures plan outlining proposed means and methods for ground improvement; and a quality control plan that describes the measures and procedures to be implemented by the contractor to document that the ground improvement elements have been constructed in conformance with the work plan and shop drawings, and that the objective of the program has been achieved.

The quality control program should include a gradation analysis of the aggregate backfill material; monitoring, recording, and daily reporting of key parameters; and modulus testing of the constructed columns/piers. The key parameters for monitoring and reporting should include, as appropriate, start and finish time for column/pier installation; treatment depth; vibrator amperage draw or tamping duration per lift; and total quantity of backfill added per column or pier. The ground improvement and testing operations should be observed by the geotechnical engineer.

9.4.2 Rigid Inclusions and Drilled Displacement Grouting

Rigid inclusions (or drilled displacement grouting), where columns of grout or concrete are constructed by drilled-displacement or drilled-replacement methods, may also be used to reinforce the subgrade below footings and improve the average stiffness of the composite ground. The concrete or grout is typically placed through the hollow stem of the drilling tool as the tool is withdrawn from the ground. The grout/concrete columns formed by these techniques do not typically include steel reinforcement and are not structurally connected to the footings with an aggregate cushion or load transfer platform between the columns and the footing. We anticipate that rigid inclusions or drilled displacement grouting can be designed to achieve an improved allowable bearing capacity of 5,000 pounds per square foot (psf). A pre-production test section should be constructed to demonstrate that the selected ground improvement technique and installation parameters can achieve the design criteria. Static or dynamic load testing should be performed to evaluate resistance to axial loads.

The ground improvement contractor should submit qualifications with resumes of key personal and descriptions of representative projects completed; a ground improvement design with shop drawings that describe the spacing, location, depth, and nominal diameter of the columns; calculations to document the basis for the design; a work procedures plan outlining proposed means and methods for ground improvement; and a quality control plan that describes the measures and procedures to be implemented by the contractor to document that the ground improvement elements have been constructed in conformance with the work plan and shop drawings, and that the objective of the program has been achieved.

The quality control program should include sampling and compression testing of the grout/concrete; and monitoring, recording, and daily reporting of key parameters. The contractor should furnish equipment to automatically measure auger rotation, auger depth, penetration rate, torque delivered to the auger, crowd force, lifting rate, volume of grout placed, and pressure of the grout near the auger tip. These parameters should be automatically recorded as a function of auger depth at vertical intervals of 2 feet or less and submitted to the geotechnical engineer for review. To reduce the potential for soil mining due to over-rotation where continuous flight augers are used, the auger penetration rate should generally exceed the auger pitch in 1½ to 2 rotations for cohesionless soil and in 2 to 3 rotations for clay. The potential for soil mining and an appropriate penetration rate for the site conditions can be evaluated by pre-production test section. The target penetration rate should be selected by the ground improvement contractor based on the proposed equipment and experience on sites with similar ground conditions or based on the pre-production test section. To reduce the potential for defects in the column, the applied grouting pressure and the withdrawal rate should be maintained so that the grout pressure at the discharge point exceeds the overburden pressure. The volume of grout placed should exceed the theoretical volume of the column, typically by about 15 to 20 percent. The contractor should select a target grout volume factor based on the proposed equipment and experience on sites with similar ground conditions or based on a pre-production test section. The observed grout volume factor should be within 7½ percent of the target. The ground improvement and testing operations should be observed by the geotechnical engineer.

9.5 Earthwork

The earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction and the following recommendations. The geotechnical engineer should observe earthwork operations. Evaluations performed by the geotechnical engineer during the course of field operations may result in new recommendations, which could supersede the recommendations in this section.

9.5.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the grading recommendations presented in the report. The owner and/or their representative, the architect, the engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

9.5.2 Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, surface obstructions (e.g., pavements, aggregate base, curb/gutter, foundations), rubble and debris, and other deleterious materials from areas to be graded. Vegetation should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate landfill. Soils containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout. Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

9.5.3 Subgrade Observation and Remedial Grading

Prior to placement of fill, erection of forms or placement of reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical engineer in accordance with the recommendations in this section or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unless otherwise noted, unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed. Recommendations for clearing and grubbing to remove vegetation and other unsuitable materials are presented in Section 9.5.2.

Undocumented fill was encountered in the borings. The fill encountered in the borings extended to depths that ranged between approximately 1½ feet (Boring B-3) and 6 feet (Boring B-5) below the ground surface. To mitigate the potential for variable support characteristics of undocumented fill under mat foundations, ground improvement as described in Section 9.4 may be performed or the building pad should be overexcavated to a depth of 5 feet below the existing grade but not less than 2 feet below the future bearing elevation for the mat foundation. Where not obstructed by property limits or adjacent structures, removals should extend a lateral distance equivalent to 5 feet beyond the foundation. The exposed subgrade after remedial excavation should be scarified and moisture conditioned as needed to achieve a moisture content near and above the optimum before compaction to 90 percent of the reference density as evaluated by ASTM D1557. Remedial excavations should be backfilled with fill that conforms with the recommendations in Section 9.5.4 and is placed and compacted in accordance with the recommendations in Section 9.5.6. Undocumented fill that conforms with the criteria for general fill in Section 9.5.4, or can be processed to conform with the criteria for general fill, may be reused as fill.

The impact of undocumented fill under footings can be mitigated by the ground improvement described in Section 9.4. Where ground improvement is not performed, the impact of undocumented fill under footings should be mitigated by overexcavating the footing locations to remove the undocumented fill. Ninyo & Moore should be retained to observe the remedial excavations to evaluate depth of removal to suitable materials. The exposed subgrade after remedial excavation should be scarified and moisture conditioned as needed to achieve a moisture content near and above the optimum before compaction to 90 percent of the reference density as evaluated by ASTM D1557. Remedial excavations should be backfilled with fill that conforms with the recommendations in Section 9.5.4 and is placed and compacted in accordance with the recommendations in Section 9.5.6. Undocumented fill that conforms with the criteria for general fill in Section 9.5.4, or can be processed to conform with the criteria for general fill, may be reused as fill. Alternatively, remedial excavations under footings may be backfilled with lean concrete or controlled low strength material (CLSM). Remedial excavations that are backfilled with general fill should extend a lateral distance beyond the footing edges equivalent to the depth of removal below the footing bearing elevation. Remedial excavations under footings that are backfilled with CLSM or lean concrete need not extend beyond the footing edges.

9.5.4 Material Recommendations

Materials used during earthwork operations should comply with the requirements listed in Table 7.

Table 7 – Recommended Material Requirements		
Material and Use	**Source**	**Requirements[1,2]**
General Fill: - for uses not otherwise specified	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
	On-site borrow	No additional requirements[1]
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02G
Permeable Aggregate	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size ¾ inch or less
Aggregate Base	Import	Class II; CSS[4] Section 26-1.02
Asphalt Concrete	Import	Type A; CSS[4] Section 39-2
Bedding and Pipe Zone Material -material below pipe invert to 12 inches above pipe	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - above bedding material	Import or on-site borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches

Notes:
1 In general, fill should not consist of pea-gravel and should be free of rocks or lumps in excess of 6-inches diameter, trash, debris, roots, vegetation or other deleterious material.
2 In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.
3 Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2018).
4 CSS is California Standard Specifications (Caltrans, 2015).

Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the consistency of import material brought to the site.

9.5.5 Subgrade Preparation

Subgrade should be prepared as per the recommendations in Table 8. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs.

Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements above.

A thin layer (approximately 3 inches) of lean concrete or controlled low strength material (CLSM) may be placed over prepared subgrade for footings or mat foundations to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

Table 8 – Subgrade Preparation Recommendations	
Subgrade Location	**Preparation Recommendations**
Below Footings	• Perform remedial grading or ground improvement as per Section 9.5.3 or Section 9.4, respectively. • Maintain compacted fill in moist condition by sprinkling water.
Below Mat Slabs	• Perform remedial grading or ground improvement as per Section 9.5.3 or Section 9.4, respectively. • Maintain compacted fill in moist condition by sprinkling water.
Below Fill and Flatwork	• Clear and grub per Section 9.5.2. • Check for unsuitable materials as per Section 9.5.3. • Scarify 8 inches then moisture condition and compact as per Section 9.5.6. • Keep in moist condition by sprinkling water.
Below Pavement	• Clear and grub per Section 9.5.2. • Check for unsuitable materials as per Section 9.5.3. • Scarify 8 inches then moisture condition and compact as per Section 9.5.6. • Proof roll compacted subgrade with loaded water truck under the observation of the geotechnical engineer. Mitigate yielding areas in accordance with the recommendations of the engineer. • Keep in moist condition by sprinkling water.
Utility Trenches	• Check for unsuitable materials per Section 9.5.2. • Remove or compact loose/soft material.

Remedial measures may be needed where the specified compaction cannot be achieved for footing and mat foundation subgrade due to shallow groundwater conditions. Where aeration, mixing, and recompaction cannot achieve the specified relative compaction, overexcavation and replacement with ¾-inch open-graded crushed rock that is compacted into the subgrade, may be needed to achieve a firm subgrade condition. The depth of overexcavation and replacement will be influenced by the conditions encountered and will be evaluated by the geotechnical engineer during construction.

9.5.6 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 9. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.

Table 9 – Compaction Recommendations

Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Aggregate Base	Pavement section or below hardscape	95 percent	Near Optimum
Subgrade	Below pavement with vehicular traffic	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Asphalt Concrete	Pavement section	91 percent	Not Applicable
Trench Backfill	Below pavement (within 2 feet of finished grade)	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe	90 percent	Near Optimum
General Fill	Below pavement (within 2 feet of finished grade)	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent

Notes:
1 Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete). The reference density of soil and aggregate should be evaluated by ASTM D 1557. The reference density of asphalt concrete should be evaluated by ASTM D 2041.
2 Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557.

9.5.7 Temporary Slopes and Excavation Stabilization

Trench excavations shall be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). Stabilization shall consist of shoring sidewalls or laying slopes back.

Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation. Table 10 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations (29 CFR, Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 10 may be used to design or select the internally-braced shoring system or trench shield. The recommendations listed in this table are based upon the limited subsurface data provided by our subsurface exploration and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse.

Table 10 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Fill & Alluvium (above groundwater)	Type C	1½h:1v (34°)	80×D + 72

Notes:

1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.
2 In layered soil, layers shall not be sloped steeper than the layer below.
3 Temporary excavations less than 4 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).
4 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical).

Excavation subgrade may become unstable and subject to pumping under heavy equipment loads if exposed to water or where excavations extend near or below the groundwater level. The contractor should be prepared to stabilize the bottom of excavations. In general, unstable bottom conditions may be mitigated by scarifying the subgrade and aerating the soil to achieve a moisture content near the optimum, dewatering to depress groundwater levels below the bottom of the excavation, overexcavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, or using geogrid to stabilize additional fill. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction.

9.5.8 Constructed Slopes

Fill slopes derived from on-site materials or cut slopes intended for long term stability may be constructed at an inclination of 2:1 (horizontal to vertical) or flatter. Constructed slopes taller than 15 feet should be re-evaluated by the geotechnical engineer.

Fill slopes, if utilized, should be constructed in accordance with the recommendations for subgrade preparation, fill placement, and other recommendations in this report. In addition, fill slopes should be over-built laterally by about 2 feet and cut back to expose compacted fill. Track-walking or wheel-rolling in lieu of overbuilding/trimming should not be permitted to mitigate the loose, uncontrolled outer surface of the fill slope. The geotechnical engineer should be consulted to provide additional recommendations for keyways, benches, and subdrains where fill slopes are constructed on grades steeper than 5:1 (horizontal to vertical).

Slopes that are not paved or otherwise armored, should be vegetated with drought-tolerant, deep-rooted plants to reduce the potential for erosion. Irrigation pipes should be anchored to the slope face rather than placed in trenches. Slope irrigation should be maintained at a level just sufficient to support plant growth. Leaking pipes should be promptly repaired.

9.5.9 Construction Dewatering

Water intrusion into the excavations may occur as a result of groundwater seepage or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

9.5.10 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with the recommendations in Section 9.5.7. Utility trenches should be backfilled with materials that conform to the recommendations in Section 9.5.4. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 9.5.6 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

To reduce potential for moisture intrusion into the building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of concrete or CLSM.

9.5.11 Rainy Weather Considerations

We recommend scheduling earthwork and foundation construction for the period between approximately April 15 and October 15 to avoid the rainy season. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed. A thin layer (approximately 3 inches) of lean concrete or CLSM may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

9.6 Retaining Walls and Vaults

Walls backfilled with imported fill or on-site soil meeting the criteria for general fill in Section 9.5.4 and retaining up to 10 feet of soil above the wall footing may be designed for active or at-rest equivalent fluid earth pressures of 86 or 96 psf per foot depth for undrained conditions with level backfill. Walls with drained backfill conditions may be designed for active or at-rest equivalent fluid earth pressures of 47 or 67 psf per foot depth with level backfill. Walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures. Vaults or other below grade walls that are restrained by framing, floor diaphragms, or abutting walls should be designed to resist at-rest earth pressures. For rising backfill conditions, the active or at-rest equivalent fluid earth pressures may be increased by 1 psf per foot depth per degree of inclination. Walls retaining broken back slopes may be evaluated by considering the slope height to be included as part of the wall height, or by considering the slope angle to be the slope of a plane extending from the toe of the slope at the back of the wall to the ground surface at a lateral distance behind the wall equivalent to twice the wall height. An additional equivalent fluid pressure of 32 psf per foot depth may be used to evaluate seismic earth pressure on retaining walls, as appropriate, for consideration with active earth pressures.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The lateral earth pressure due to a backfill surcharge of 240 psf should be a uniform horizontal surcharge of 94 psf for yielding conditions and 135 psf for at-rest conditions. An additional backfill surcharge and lateral earth pressure for adjacent footings should be considered, as applicable, where the adjacent footings bear above an imaginary plane that rises up and away from the bottom edge of the wall at a 2:1 (horizontal to vertical) gradient.

Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by weep holes or a subdrain at the base of the wall stem consisting of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the wall. Alternatively, geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used to supplement a smaller subdrain located near the base of the wall. Measures to reduce the rate of moisture or vapor intrusion through the wall may be advisable for walls where the discoloration resulting from moisture intrusion would be undesirable. Such measures might include use of concrete with a low water-to-cementitious-materials ratio, and/or the placement of an asphalt emulsion or 10-mil thick plastic membrane to the back surface of the wall.

Lateral forces may be resisted by friction at the base of the wall footing for gravity and semi-gravity walls, and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present, for semi-gravity and cantilever walls. Semi-gravity and cantilever walls on near level ground may be designed for a passive equivalent fluid lateral earth pressure of 225 psf per foot depth presuming a lateral deflection equivalent to 1 percent of the wall embedment depth to mobilize the passive condition. The passive earth pressure may be proportionally reduced for lower levels of lateral deflection as desired. The passive earth pressure for walls on ground sloping more than 5 percent should be reduced by 5 psf per foot depth per degree of inclination. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Gravity and semi-gravity walls may be designed for a coefficient of friction of 0.35 to resist lateral loads and a net allowable bearing capacity of 1,300 psf for a 12-inch footing width and 12 inches of embedment below the adjacent grade plus 200 psf per additional foot of width and 500 psf per additional foot of embedment up to 4,000 psf. The allowable bearing capacity may be increased by one-third for seismic load combinations. The coefficient of friction may be increased to 0.50 where the footing is constructed over 6 inches of aggregate base compacted to 95 percent of the reference density as evaluated by ASTM D1557.

Footing bottoms should not be sloped more than 1-unit vertical to 10 units horizontal. Wall footings may be stepped provided that the bearing grade differential between adjacent steps does not exceed 18 inches and the slope of a series of such steps does not exceed 1-unit vertical to 2 units horizontal. Walls should be designed to withstand a total static settlement of 1 inch with a differential of ½ inch over a 20-foot span.

9.7 Pavement and Flatwork

Recommendations for pavement and exterior flatwork are presented in the following sections. Recommendations for preparation of subgrade are presented in Section 9.5.5. Pavement sections were evaluated for a range of traffic indexes or loading conditions. The designer may interpolate between the values provided once a traffic index or loading condition has been selected.

9.7.1 Asphalt Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans, 2016). Alternative sections were evaluated. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 11.

Table 11 – Asphalt Concrete Pavement Structural Sections

Design R-Value	Traffic Index	Alternative 1	Alternative 2
5	5	3 inches AC 10 inches AB	6 inches AC 5 inches AB
5	6	3½ inches AC 13 inches AB	7 inches AC 5 inches AB
5	7	4 inches AC 16 inches AB	8 inches AC 7 inches AB

Notes:
1 AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2015).
2 AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2015).

Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted per Section 9.5.6. Asphalt concrete should be placed and compacted per Section 9.5.6. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over asphalt pavement should be discouraged. Cracks that form in the asphalt concrete surface should be periodically sealed to reduce moisture intrusion into the aggregate base section. Deep curbs that extend 6 inches below the aggregate base section may be used to reduce the potential moisture intrusion into the aggregate base section adjacent to landscaped areas or the bottom of slopes. Subdrains may be considered as a supplement or alternative means of the mitigating moisture in the aggregate base section. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

9.7.2 Exterior Flatwork

Concrete walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 4 inches of aggregate base. Concrete thickness should be increased to 6 inches over 6 inches of aggregate base at driveways for vehicular traffic up to periodic garbage trucks and emergency vehicles. The aggregate base should conform to and be compacted in accordance with our recommendations in Sections 9.5.4 and 9.5.6, respectively. Flatwork and driveway subgrade should be prepared in accordance with the recommendations in Section 9.5.5.

Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction should be detailed and constructed in accordance with the guidelines of ACI Committee 302 (ACI, 2016). The lateral spacing between contraction joints should be 8 feet or less for a 4-inch thick slab and 12 feet or less for a 6-inch thick slab. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs.

Flatwork may be reinforced with distributed steel to reduce the potential for differential slab movement where cracking occurs. The distributed reinforcing steel should be terminated about 6 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways. Slabs reinforced with distributed steel should be 6 inches thick (or more). To reduce the potential for differential slab movement across joints, the distributed steel may be extended through the joints. This improvement will be balanced by a reduction in the functionality of the contraction joint to encourage crack formation at joints. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of cover over the steel. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

9.8 Concrete

Laboratory testing indicated that the site soil may be considered a corrosive environment to structures per the Caltrans Corrosion Guidelines (2018) based on the concentration of chloride. Although the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested, due to the variability in the on-site soil, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, the concrete should have a water-to-cement ratio of not more than 0.40 and a 3-inch thick or thicker concrete cover should be maintained over reinforcing steel where concrete is cast-in-place against soil. Concrete cover over reinforcing steel for other exposure conditions should conform to ACI guidelines (ACI, 2016). A corrosion engineer should be consulted to further assess the potential for corrosion, review these mitigation measures, and provide recommendations for supplementary measures as-needed.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for slabs and flatwork should not contain large quantities of water or accelerating admixtures containing calcium chloride. Higher compressive strengths may be achieved by using larger aggregates in lieu of increasing the cement content and corresponding water demand. Additional workability, if desired, may be obtained by including water-reducing or air-entraining admixtures. Concrete should be placed in accordance with ACI Manual of Concrete Practice (MCP) and project specifications. Particular attention should be given to curing techniques and curing duration. Slabs that do not receive adequate curing have a more pronounced tendency to develop random shrinkage cracks and other defects.

9.9 Moisture Vapor Retarding System

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the ACI Manual of Concrete Practice (ACI, 2016), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient.

9.10 Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Drainage gradients should be 2 percent or more a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more a distance of 10 feet or more from the structure for pervious surfaces. Slope, pad, and roof drainage should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should be limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. Future alteration of the established drainage patterns may impact the constructed improvements.

9.11 Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

9.12 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in discrete borings and soundings. During construction, the geotechnical engineer should be retained to check and evaluate the exposed subsurface conditions for consistency with the findings in this report. During construction, the geotechnical engineer should be retained to:

- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill, aggregate base, and asphalt concrete.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe drilling and construction of soldier-pile-and-lagging walls if installed.
- Observe ground improvement operations if performed.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

10 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that this evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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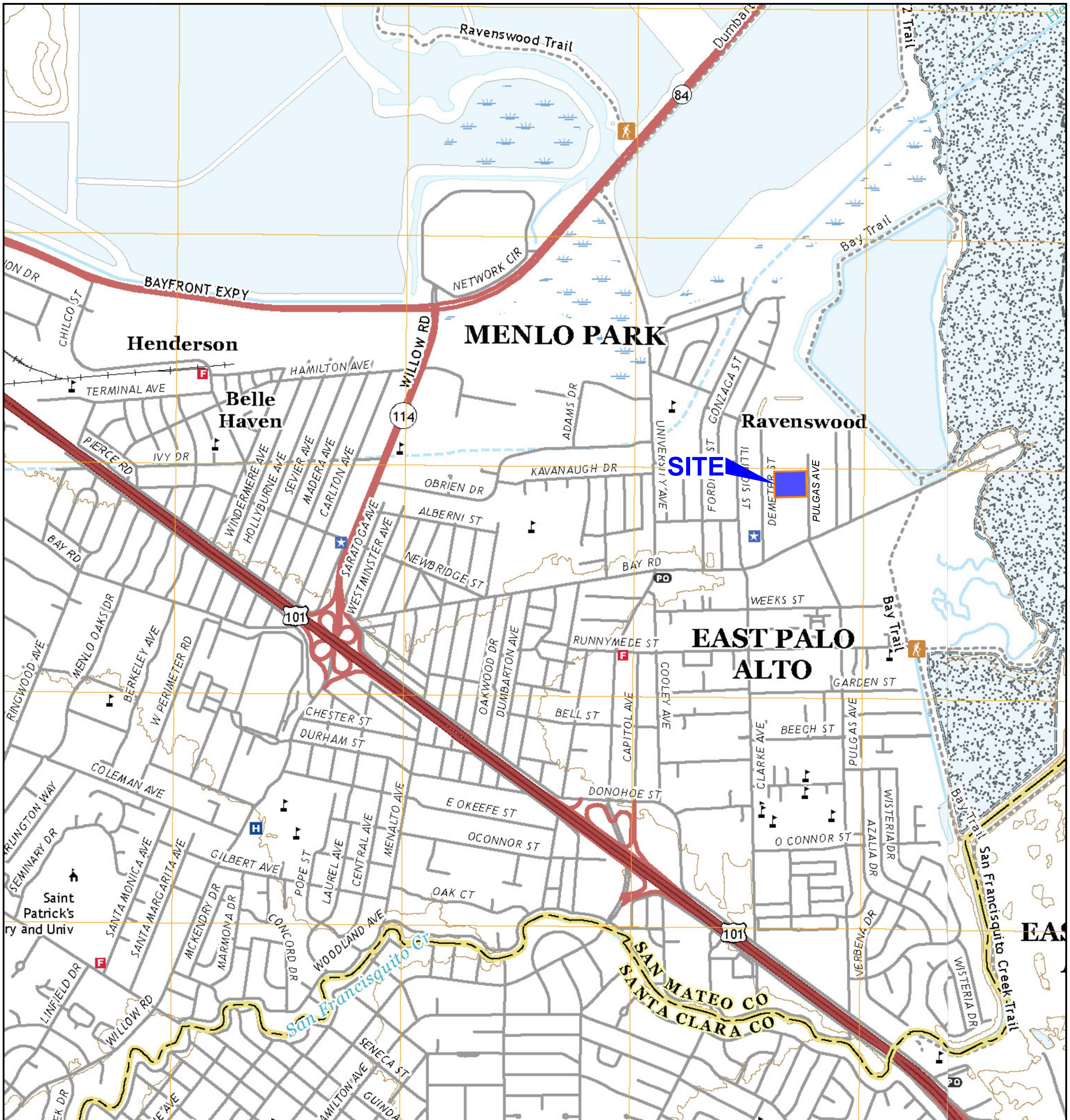
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2018

N

SCALE (FEET)

0 2,000 4,000

403645001.dwg 05/27/2020 AEK

FIGURE 1

SITE LOCATION

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA
403645001 I 05/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

LEGEND

SITE BOUNDARY

B-1 (50') SOIL BORING TOTAL DEPTH IN FEET

CPT-1 (65') CPT LOCATION TOTAL DEPTH, IN FEET

P-1 (2') PERCOLATION TEST LOCATION TOTAL DEPTH, IN FEET

MAM SURVEY LINE

N

SCALE (FEET)
0 100 200

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: GOOGLE EARTH, 2020

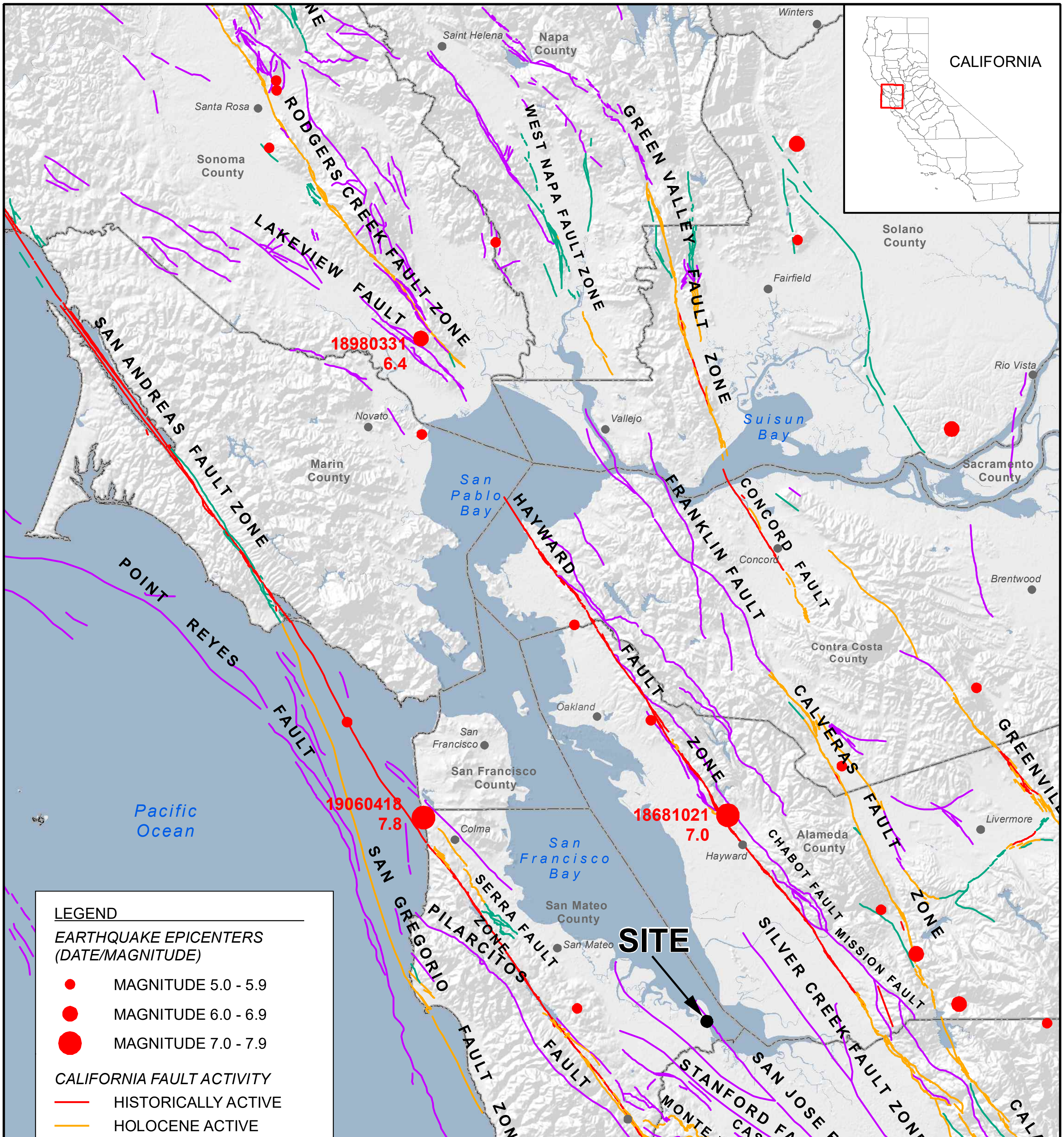
403645001.dwg 05/27/2020 AEK

FIGURE 2

EXPLORATION LOCATIONS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA
403645001 I 05/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA
CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

N

SCALE (MILES)
0 10 20

403645001_FL.pdf 05/27/2020 AEK

FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

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2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA
403645001 | 05/20

LEGEND

Qa	ALLUVIAL GRAVEL, SAND, SILT, & CLAY (HOLOCENE)
Qbm	ESTUARINE ORGANIC CLAY & SILTY CLAY (HOLOCENE)
Qa.1	ALLUVIAL FAN DEPOSITS (HOLOCENE)

THRUST FAULT

FAULT

GEOLOGIC CONTACT

STRIKE AND DIP OF BEDDING

N

SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: DIBBLEE, 2007

403645001.dwg 05/27/2020 AEK

FIGURE 4

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Geotechnical & Environmental Sciences Consultants

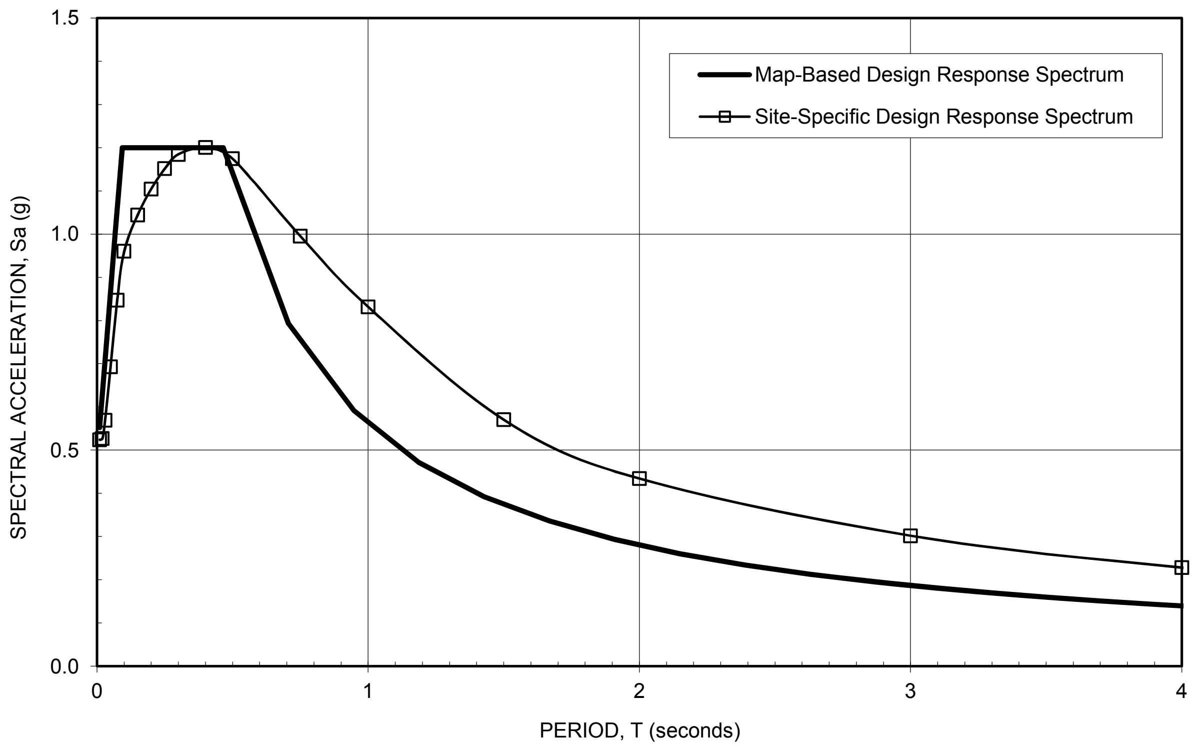
REGIONAL GEOLOGY

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 05/20

PERIOD (seconds)	MAP-BASED DESIGN RESPONSE SPECTRUM Sa, (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.010	0.557	0.524
0.050	0.866	0.693
0.100	1.200	0.960
0.150	1.200	1.044
0.200	1.200	1.104
0.250	1.200	1.151
0.300	1.200	1.184
0.400	1.200	1.200
0.500	1.120	1.174
0.750	0.747	0.995
1.000	0.560	0.831

S_{DS} = 1.080	S_{D1} = 0.868	S_{MS} = 1.620	S_{M1} = 1.302	PGA_M = 0.616



NOTES:

1 Site specific design response spectrum is two-thirds of the acceleration response spectrum that is associated with the maximum considered earthquake and is expected to achieve a 1% probability of collapse in a 50-year period (MCE_R). The MCE_R spectrum is computed as the lesser of probablistic and deterministic spectral response accelerations at each period per ASCE 7-16 Section 21.2.3. The site specific design response spectrum conforms with the lower bound limit in ASCE 7-16 Section 21.3.

2 Probabilistic response spectrum is the 5% damped acceleration in the direction of maximum horizontal response associated with a ground motion having a 2% probability of exceedance in 50 years and adjusted for risk of collapse per Method 1 of ASCE 7-16 Section 21.2.1. Spectrum computed using UCERF3 single branch earthquake forecast and the CY14, CB14, and BSSA14 attenuation relationships.

3 Deterministic response spectrum is the 84th percentile, 5% damped spectral reponse acceleration in the maximum horizontal direction computed using CY14, CB14, and BSSA14 attenuation relationships and considering a Mw 8.0 event on the San Andreas fault about 13km from the site. Scaled to 1.5*Fa where appropriate.

4 Map-based design response spectrum is computed from mapped spectral ordinates, modified for Site Class C (Very dense and soft rock) conditions, in accordance with ASCE 7-16 Section 11.4. It is presented for comparison.

FIGURE 5

ACCELERATION RESPONSE SPECTRA

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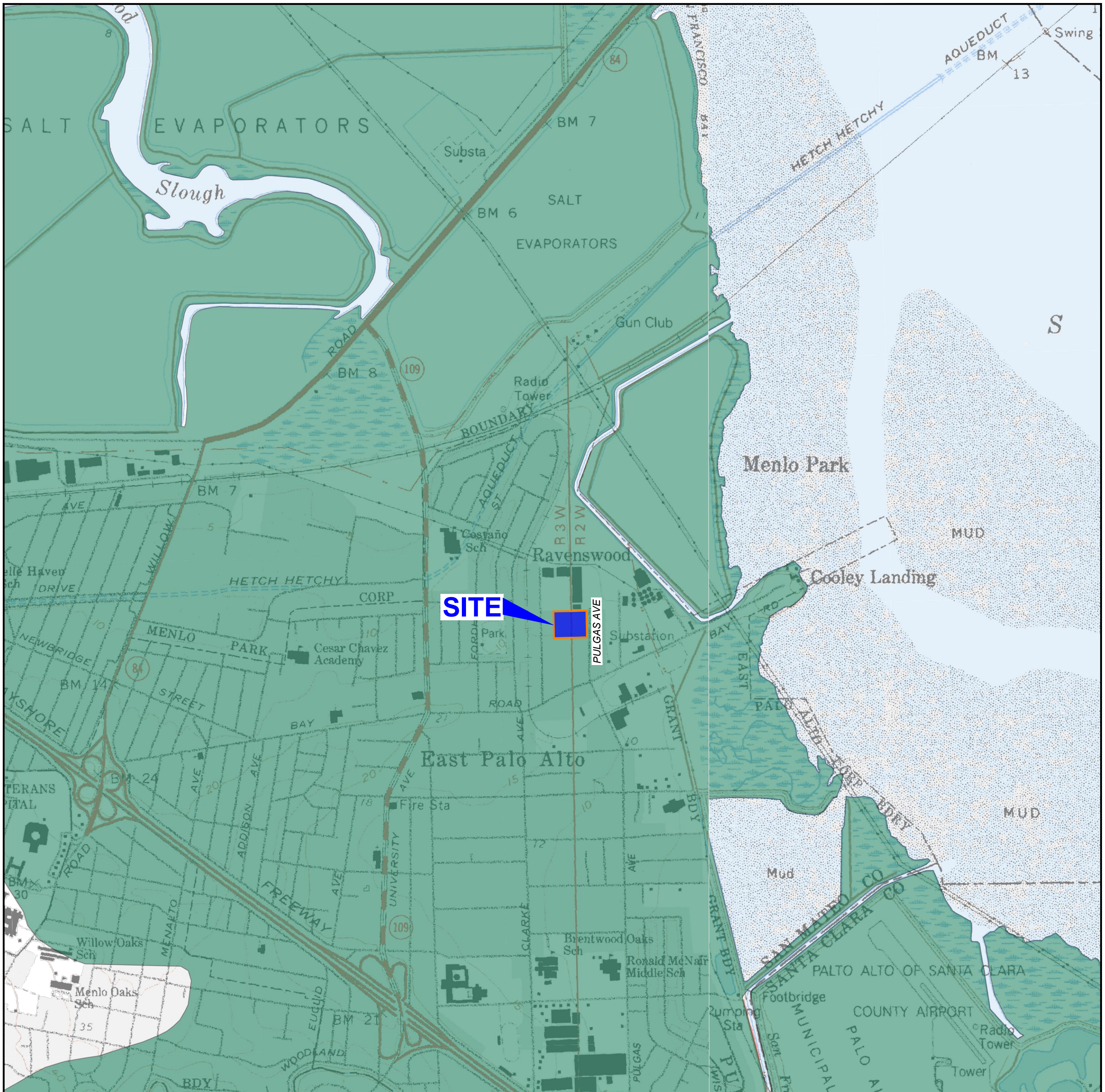

LEGEND

LIQUEFACTION ZONES:
Areas where historic occurrence of liquefaction, or local geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: CGS, 2006

N

SCALE (FEET)
0 2,000 4,000

403645001.dwg 05/27/2020 AEK

FIGURE 6

SEISMIC HAZARD ZONES

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
A disturbed soil sample was obtained in the field using the following method.

Bulk Sample
Bulk samples of representative earth materials were obtained from the borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with an inside diameter of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

The Shelby Tube Sampler
The Shelby tube sampler is a seamless, thin-walled, steel tube having an external diameter of 3.0 inches and a length of 30 inches. The tube was connected to the drill rod or hand tool and pushed into an undisturbed soil mass to obtain a relatively undisturbed sample of cohesive soil in general accordance with ASTM D 1587. When the tube was almost full (to avoid overpenetration), it was withdrawn from the boring, removed from the drill rod or hand tool, sealed at both ends, and transported to the laboratory for testing.

Field Testing
The following tests were performed in the field to evaluate soil properties.

Static Cone Penetrometer
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 1.5 square centimeters was manually pushed 6 inches into the soil. The penetrometer was instrumented to measure the Cone Penetration Index (Qc) computed as the peak force on the cone divided by the cone base area. The Cone Penetration Index is reported in kilograms per square centimeter (ksc) on the boring logs at the depth of the test as a measure of the relative density or consistency of the soil encountered.

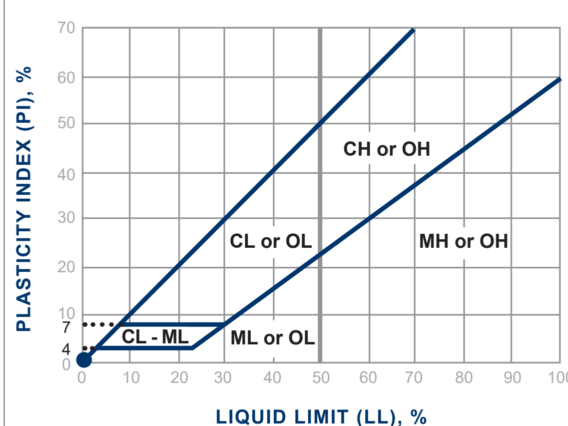
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0			Qc=17 Qc=20				CL	FILL: Brown to dark brown, moist, stiff, lean CLAY.
			22"/30"	17.2	94.2		CL	ALLUVIUM: Brown, moist, stiff, sandy lean CLAY.
			16	20.8	107.3			
10			11	20.5	106.2			Yellowish brown, wet, stiff.
			9				SP-SC	Brown, wet, loose, poorly graded SAND with clay.
								Medium Dense.
20			52				CL	Brown, wet, hard, lean CLAY.
			30	22.7	99.7			Very stiff.
30			30	21.0	104.2			
			38	22.1	102.6			
			21	23.7			SC	Yellowish brown, wet, dense, clayey SAND.
40			24				CL	Yellowish brown, wet, very stiff, lean CLAY.

DATE DRILLED 11/12/2019 BORING NO. B-1

GROUND ELEVATION 12' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'

DRIVE WEIGHT 140 lbs (wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40								ALLUVIUM:(continued) Yellowish brown, wet, very stiff, lean CLAY.
		■	42	25.4	90.6			Hard.
							SC	Yellowish brown, wet, medium dense, clayey SAND.
		■	60	19.6	102.2		CL	Yellowish brown, wet, hard, lean CLAY.
50								Total Depth = 50.0 feet. Backfilled with cement grout on 11/12/2019. Groundwater was encountered at a depth of approximately 7 feet below the round surface during drilling. Notes: The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
60								
70								
80								

DATE DRILLED 11/12/2019 BORING NO. B-1

GROUND ELEVATION 12' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'

DRIVE WEIGHT 140 lbs (wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DATE DRILLED	11/12/2019	BORING NO.	B-2
GROUND ELEVATION	12' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'		
DRIVE WEIGHT	140 lbs (wireline)	DROP	30 inches
SAMPLED BY GL	LOGGED BY GL	REVIEWED BY	PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					CL	FILL: Brown, dry to moist, firm, sandy lean CLAY.
	Bulk	Qc=15 Qc=10				SC	ALLUVIUM: Yellowish brown, moist, very loose, clayey SAND.
	Driven	Qc=18				CL	Brown, moist, firm, sandy lean CLAY.
	Driven	12				SC	Brown, moist, loose, clayey SAND. Yellowish brown.
10	Driven	12				CL	Brown, wet, firm, sandy lean CLAY.
	Driven	11					
	Driven	10	19.2			SC	Brown, wet, loose, clayey SAND.
						CL	Brown, wet, very stiff, lean CLAY.
20	Driven	28	25.2	99.2			
	Driven	23					Light brown.
30	Driven	22					Yellowish brown.
	Driven	31	20.4	104.3		SP-SC	Brown, wet, medium dense, poorly graded SAND with clay.
	Driven	51					Very dense.
40							Total Depth = 36.5 feet. Backfilled with cement grout on 11/12/2019. Groundwater was encountered at a depth of approximately 6.5 feet below the ground surface during drilling. Groundwater was measured at a depth of approximately 8 feet in borehole about 15 minutes after drilling. See notes on Boring B-1.

DATE DRILLED 11/12/2019 BORING NO. B-3

GROUND ELEVATION 13' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'

DRIVE WEIGHT 140 lbs (wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■		Qc=15				CL	FILL: Dark brown, dry to moist, firm, lean CLAY.
	■		Qc=11 Qc=16 Qc=18 Qc=19				SC	ALLUVIUM: Yellowish brown, moist, very loose, clayey SAND.
		■	17					Loose.
		■	11	20.1	103.3		CL	Brown, wet, firm, sandy lean CLAY. Trace gravel.
10								
		■	13	21.9	91.2		SC	Yellowish brown, wet, loose, clayey SAND.
20		■	26	26.5	93.7		CL	Brown, wet, very stiff, lean CLAY.
		■	14	23.5	100.5			Light brown, wet, stiff, sandy lean CLAY. Yellowish brown.
30		■	32	17.1	111.6			Very stiff.
		■	20	22.2	99.9			Reddish yellow, stiff.
								Total Depth = 35 feet. Backfilled with cement grout on 11/12/2019. Groundwater was encountered at a depth of approximately 7 feet below the ground surface during drilling. Groundwater was measured at a depth of approximately 7.5 feet in borehole about 15 minutes after drilling. See additional notes on Boring B-1.
40								

DATE DRILLED	11/11/2019	BORING NO.	B-4		
GROUND ELEVATION	12' ± (MSL)	SHEET	1	OF	1
METHOD OF DRILLING	6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'				
DRIVE WEIGHT	140 lbs(wireline)	DROP	30 inches		
SAMPLED BY	GL	LOGGED BY	GL	REVIEWED BY	PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE:Approximately 4.5 inches thick.
							GP-GM	AGGREGATE BASE:Approximately 2 inches thick. Gray to brown, dry to moist, medium dense, poorly graded GRAVEL with sand.
			Qc=11 Qc=14 Qc=14 Qc=17				CL	FILL: Olive gray to black, moist, firm, sandy lean CLAY.
								Stiff.
			21	20.5	103.3		CL	ALLUVIUM: Brown, moist, very stiff, lean CLAY.
				▽				
			10					Wet; firm; sandy.
10				23.2	104.8			
			11					Stiff, decrease in sand content.
				▼				
			8					Firm.
							SC	Yellowish brown, wet, very loose, clayey SAND.
							CL	Yellowish brown, wet, firm, sandy lean CLAY.
			22					Brown, very stiff.
20								
			50					Hard; trace sand.
			10					Light brown, firm; trace gravel.
			54	15.2	113.3		SW-SC	Brown, wet, medium dense, well-graded SAND with clay.
30								
			21					
								Total Depth = 31.5 feet. Backfilled with cement grout on 11/11/2019. Groundwater was encountered at a depth of approximately 8 feet below the ground surface during drilling. Groundwater was measured at a depth of approximately 12 feet in borehole about 15 minutes after drilling. See additional notes on Boring B-1.
40								

FIGURE B- 5

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DATE DRILLED 11/11/2019 BORING NO. B-5

GROUND ELEVATION 10' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5'

DRIVE WEIGHT 140 lbs(wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 4 inches thick.
						GP-GM	AGGREGATE BASE: Approximately 8 inches thick. Gray to brown, dry to moist, medium dense, poorly graded GRAVEL with sand.
	Bulk	Qc=12				CL	FILL: Olive gray to black, moist, firm, lean CLAY.
		Qc=11					
		24"/28"	22.1	104.2			
	Driven	20	23.0	101.9			
						CL	ALLUVIUM: Yellowish brown, moist, stiff, lean CLAY.
	Driven	11	30.1	62.7			Wet.
10							
	Driven	20	30.5	90.3			Stiff.
	Driven	40	25.7	97.3		CH	Brown, wet, hard, fat CLAY.
20							
	Driven	32	14.0	107.9			Light brown; very stiff.
						SW-SC	Brown, wet, medium dense, well-graded SAND with clay.
	Driven	45	18.4	107.4			
30							
	Driven	63					Very dense.
	Driven	23					Medium dense.
	Driven	24	18.6			SP	Brown, wet, medium dense, poorly graded SAND.Approximately 2 inches thick.
40							

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EAST PALO ALTO, CALIFORNIA
403645001 | 5/20

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 11/11/2019 BORING NO. B-5 GROUND ELEVATION 10' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 6" Hollow Stem Auger, Mobile B-61, Hand Auger Top 5' DRIVE WEIGHT 140 lbs(wireline) DROP 30 inches SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC **DESCRIPTION/INTERPRETATION**
40							CL	ALLUVIUM:(continued) Yellowish brown, wet, hard, sandy lean CLAY.
		■	50	30.7	98.6			
		■	52	23.4	98.2			
50								Total Depth = 50 feet.Backfilled with cement grout on 11/11/2019. Groundwater was encountered at a depth of approximately 7 feet below the ground surface during drilling. Groundwater was measured at a depth of approximately 10 feet in borehole about 15 minutes after drilling. See additional notes on Boring B-1.
60								
70								
80								

APPENDIX B

Cone Penetration Testing

APPENDIX B

CONE PENETRATION TESTING

Field Procedure for Cone Penetration Testing

A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 15 square centimeters was hydraulically pushed through the soil using the reaction mass of a 30 ton rig at a constant rate of about 20 millimeter per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the force on the conical point required to penetrate the soil, the force on a friction sleeve behind the cone tip as the penetrometer was advanced, and the pore pressure on a transducer behind the cone tip. Penetration data was collected and recorded electronically at intervals of about 2-inches. Cone resistance corrected for pore pressure was calculated by dividing the measured force of penetration by the cone base area and adding a fraction of the recorded pore pressure. Friction sleeve resistance was calculated by dividing the measured force on the friction sleeve by the surface area of the sleeve. The friction ratio was calculated as the ratio of the tip resistance to the sleeve friction. A graph of the computed values of cone resistance (tip) and friction ratio are presented on the logs in the following pages. The tip resistance and friction ratio were used to classify the soil behavior type encountered using the method by Robertson (2009). A graph of the encountered soil types are also presented on the logs in the following pages.

CONETEC

Job No: 19-56172
Client: Ninyo & Moore
Project: Project Thunder
Start Date: 11-Nov-2019
End Date: 26-Nov-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting[2] (m)	Elevation[3] (ft)	Refer to Notation Number
CPT-01	19-56172_CP01	26-Nov-2019	383:T1500F15U500	8.2	64.96	4147814	576660	13	
CPT-02	19-56172_CP02	26-Nov-2019	383:T1500F15U500	7.4	100.97	4147844	576734	12	
CPT-03	19-56172_CP03	11-Nov-2019	443:T1500F15U500	4.7	75.54	4147938	576730	9	
CPT-04	19-56172_CP04	26-Nov-2019	383:T1500F15U500	8.4	75.46	4147851	576631	13	
CPT-05	19-56172_CP05	11-Nov-2019	443:T1500F15U500	7.0	65.12	4147961	576641	12	
CPT-06	19-56172_CP06	11-Nov-2019	443:T1500F15U500	7.3	65.12	4147903	576661	12	

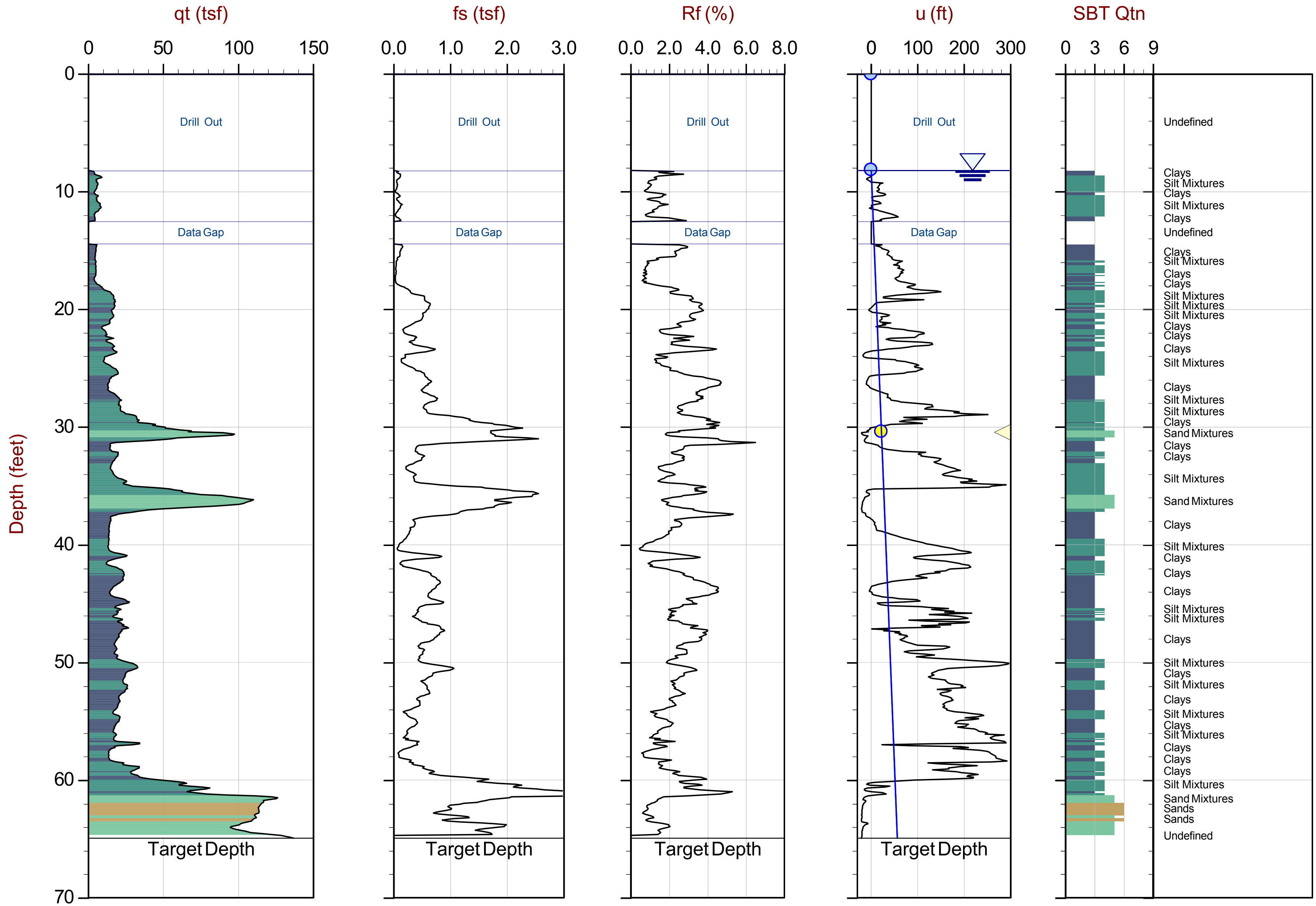
1. The assumed phreatic surface was based on the results of the shallowest pore pressure dissipation test performed within the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. Elevations are refrenced to the ground surface and are derived from Google Earth Elevation for the recorded coordinates.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-26 09:40
Site: Project Thunder

Sounding: CPT-01
Cone: 383:T1500F15U500



Max Depth: 19.800 m / 64.96 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147814m E: 576660m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

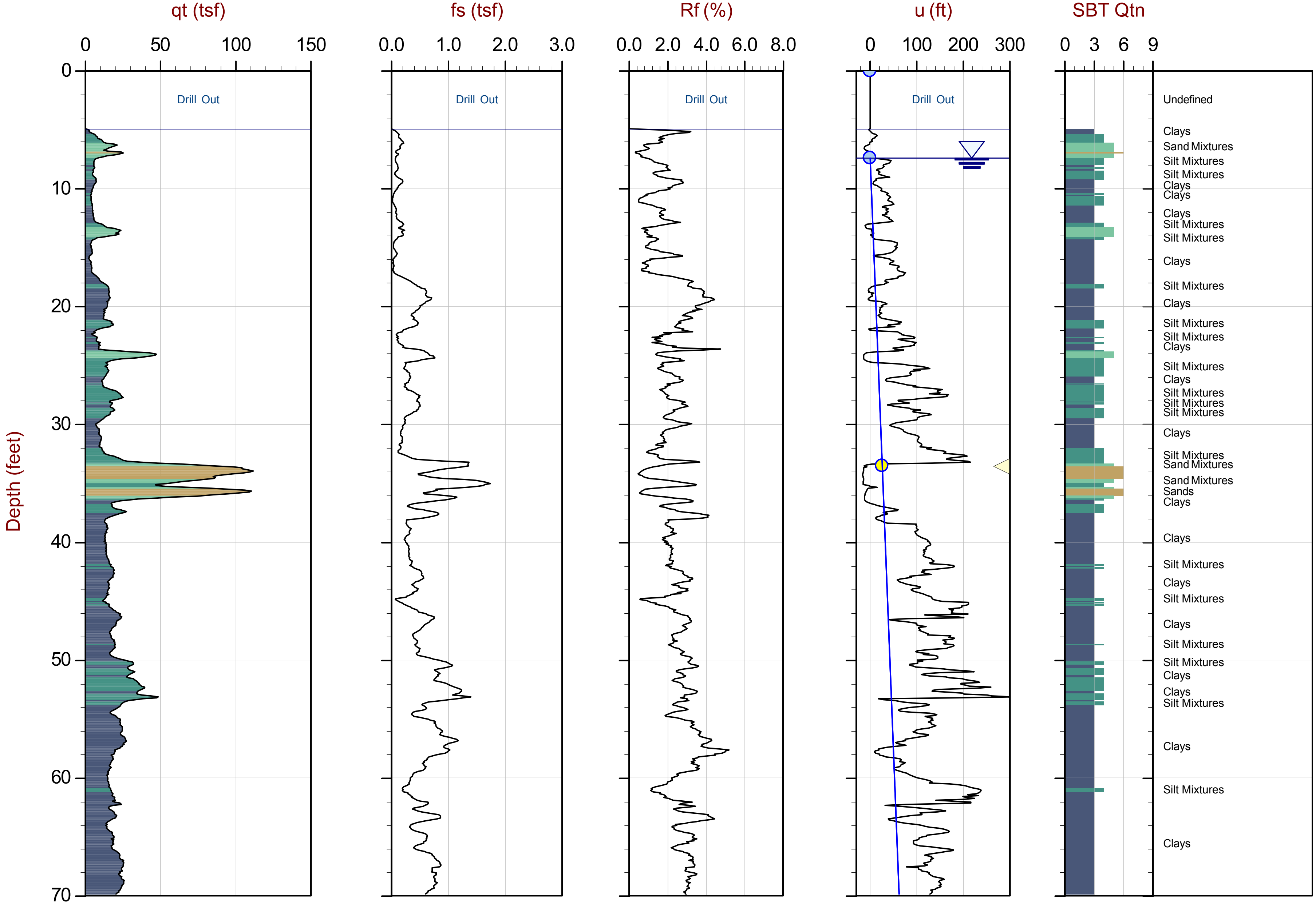
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-26 08:06
Site: Project Thunder

Sounding: CPT-02
Cone: 383:T1500F15U500



Max Depth: 30.775 m / 100.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147844m E: 576734m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

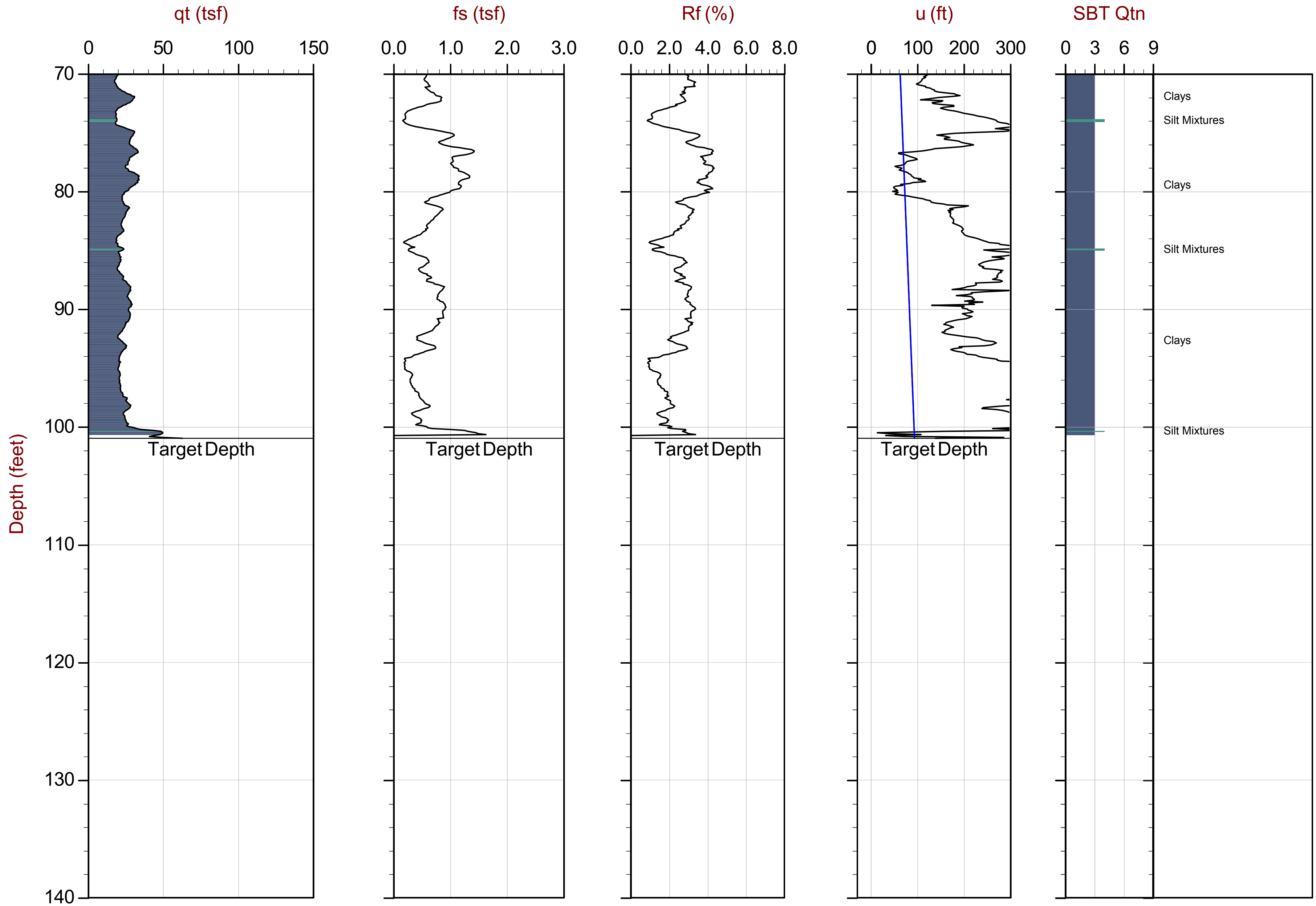
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-26 08:06
Site: Project Thunder

Sounding: CPT-02
Cone: 383:T1500F15U500



Max Depth: 30.775 m / 100.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147844m E: 576734m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

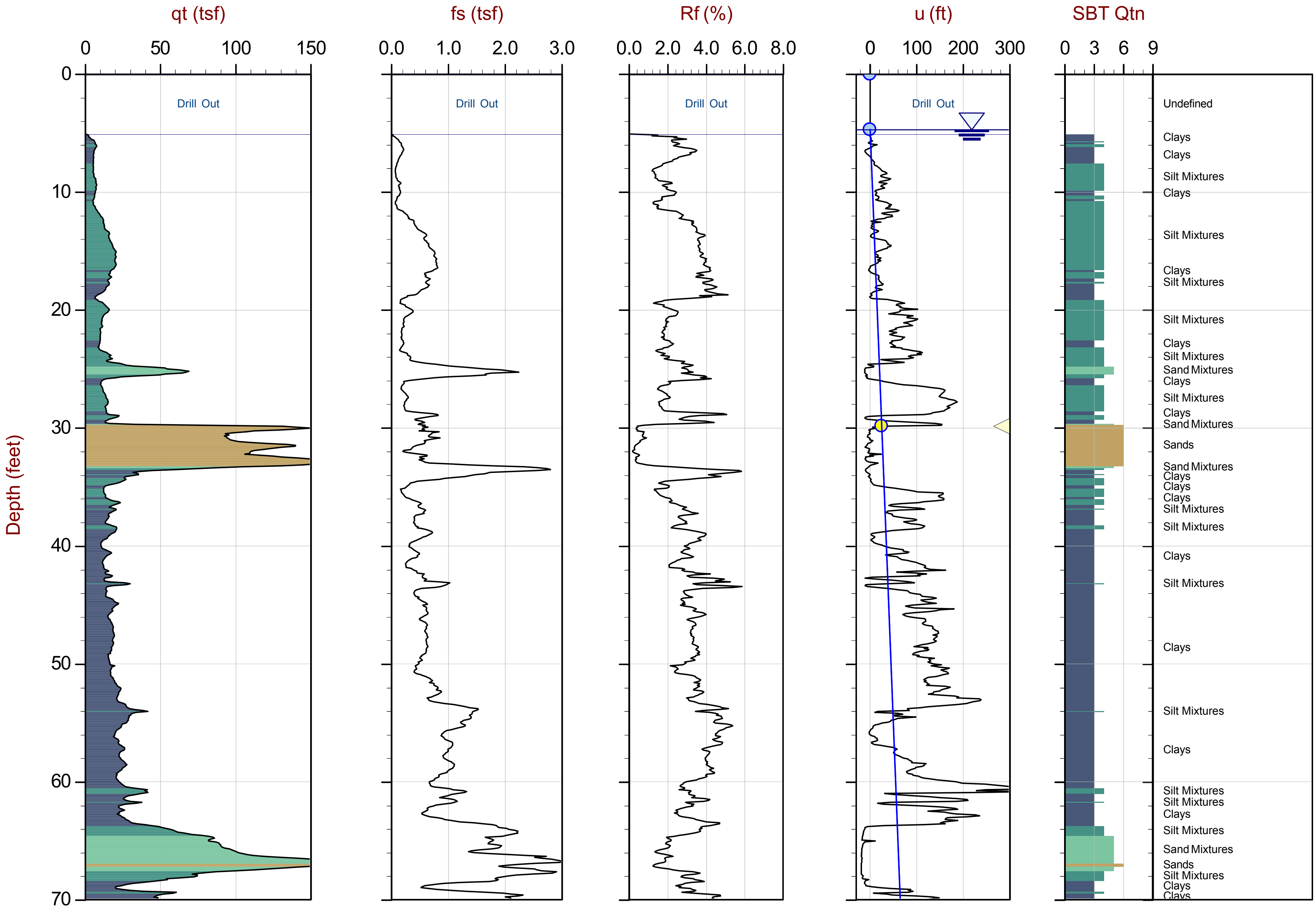
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-11 07:51
Site: Project Thunder

Sounding: CPT-03
Cone: 443:T1500F15U500



Max Depth: 23.025 m / 75.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147938m E: 576730m

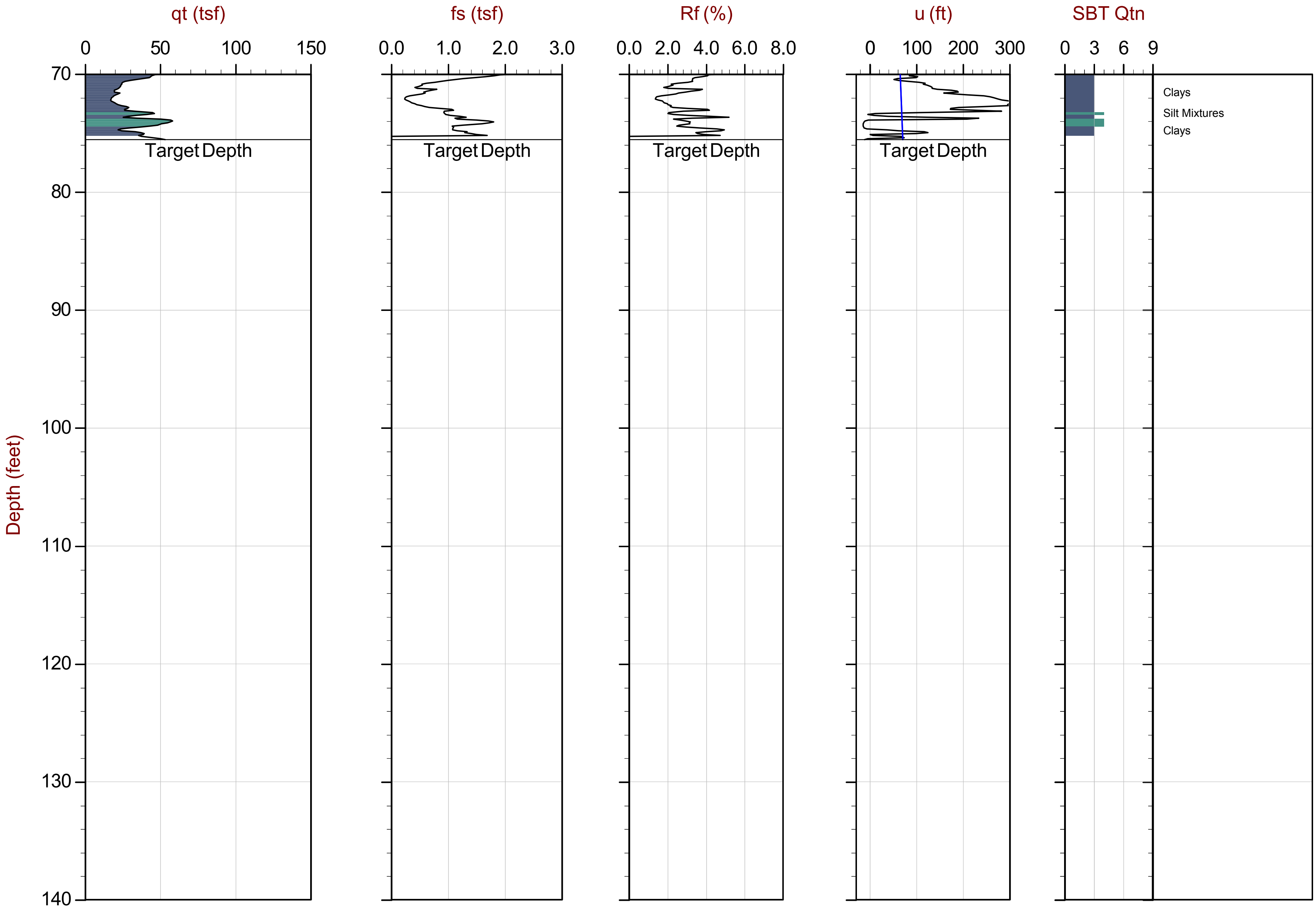
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-11 07:51
Site: Project Thunder

Sounding: CPT-03
Cone: 443:T1500F15U500



Max Depth: 23.025 m / 75.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147938m E: 576730m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

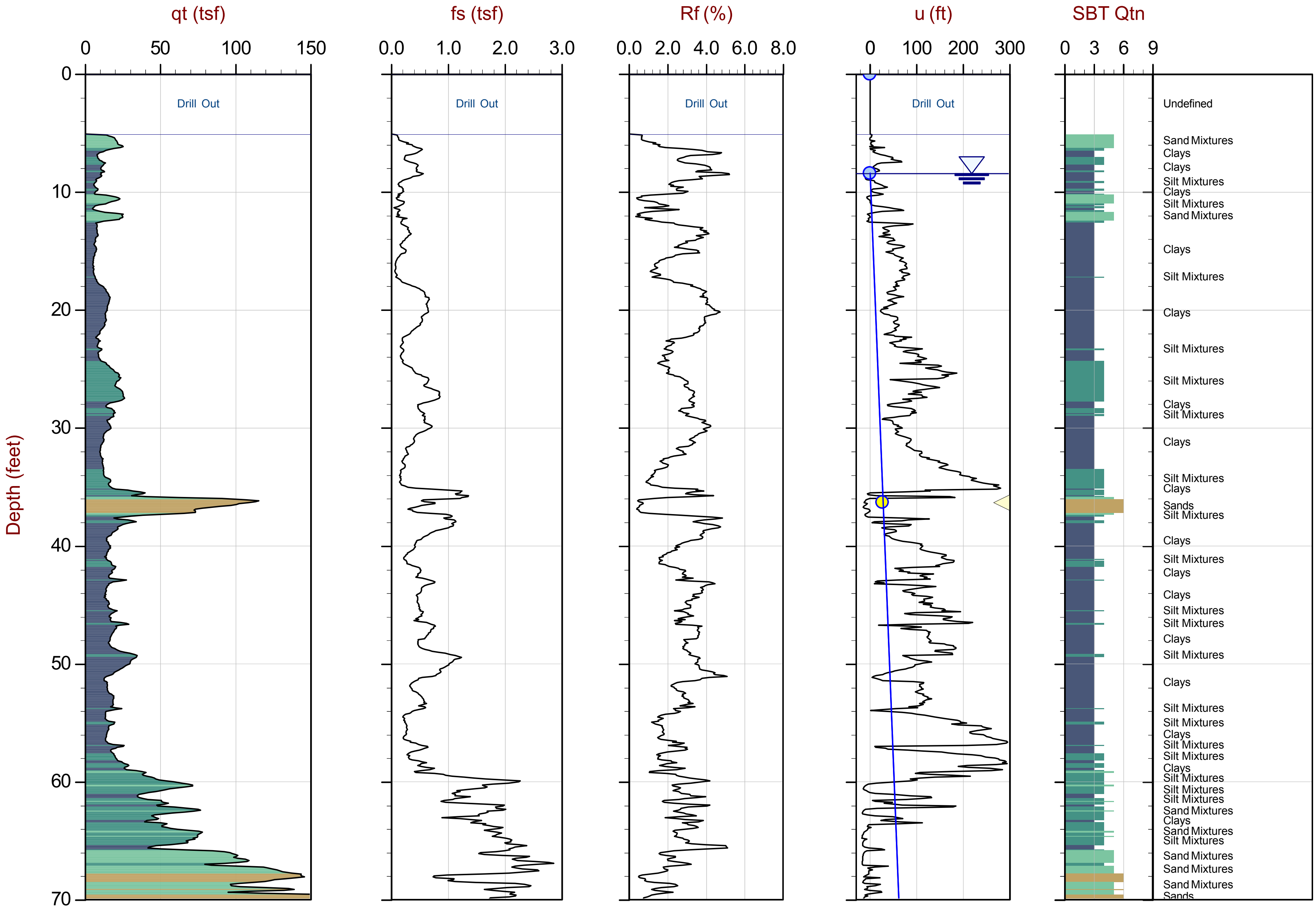
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-26 10:31
Site: Project Thunder

Sounding: CPT-04
Cone: 383:T1500F15U500



Max Depth: 23.000 m / 75.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147851m E: 576631m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

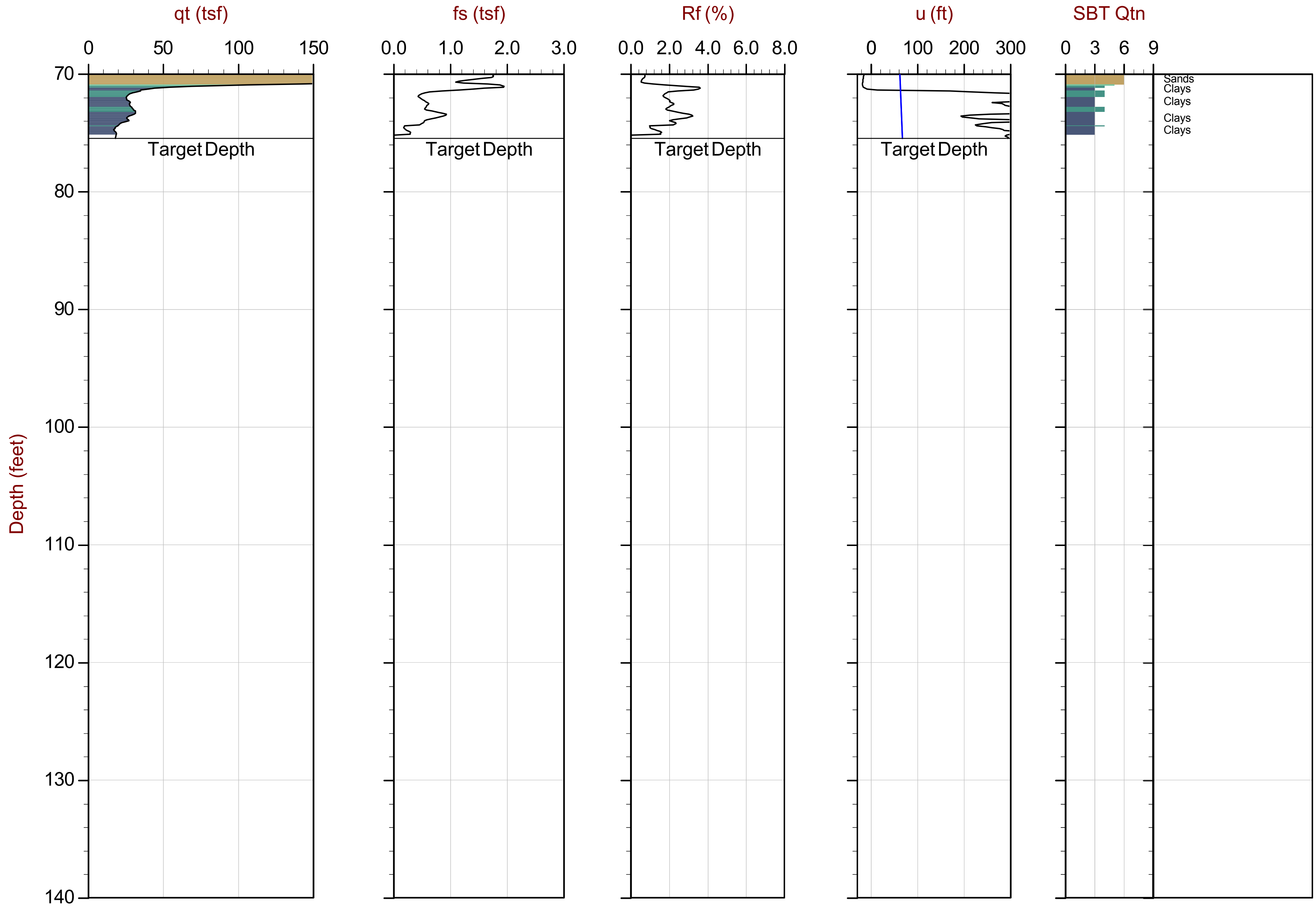
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-26 10:31
Site: Project Thunder

Sounding: CPT-04
Cone: 383:T1500F15U500



Max Depth: 23.000 m / 75.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147851m E: 576631m

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

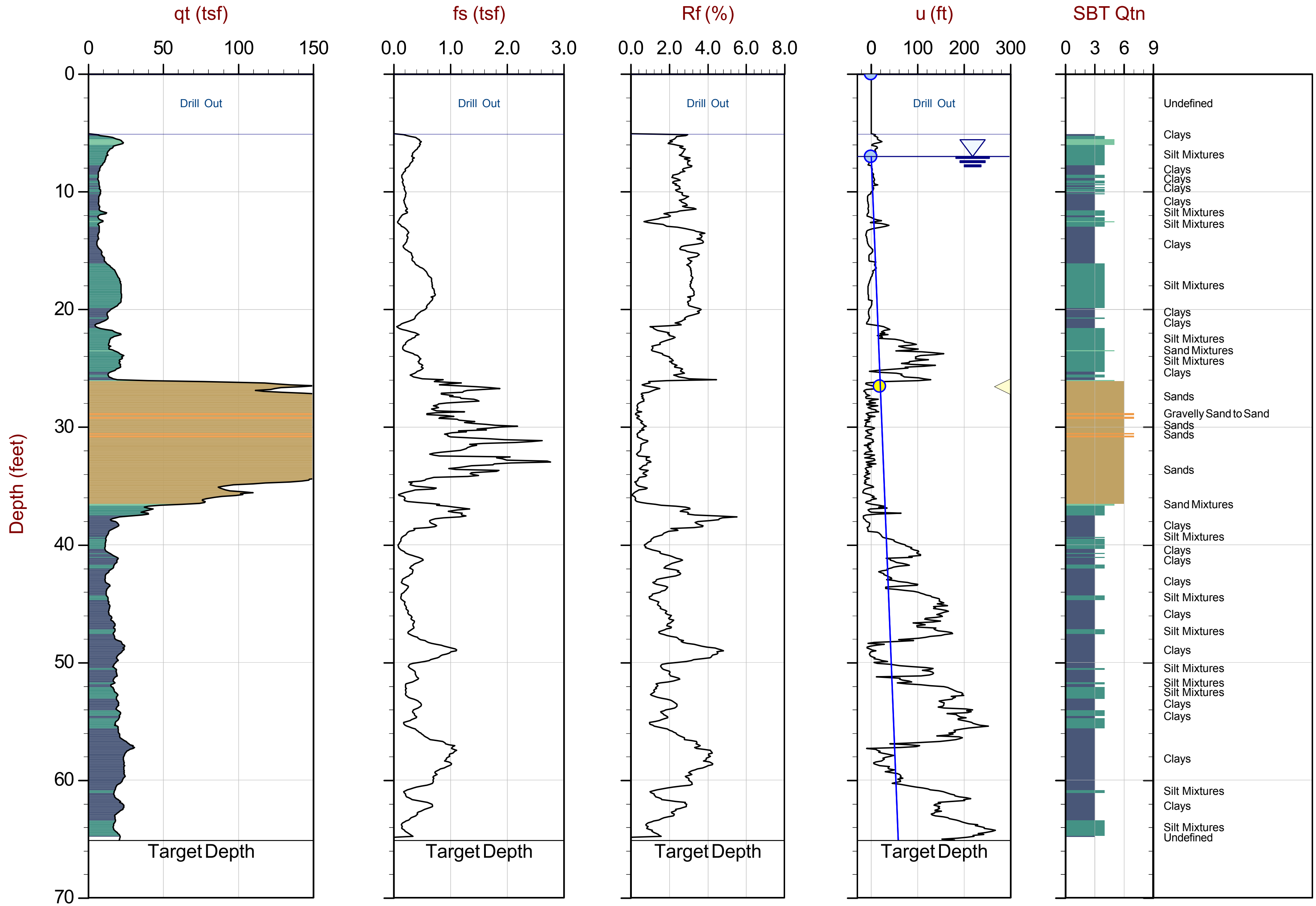
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-11 09:17
Site: Project Thunder

Sounding: CPT-05
Cone: 443:T1500F15U500



Max Depth: 19.850 m / 65.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147961m E: 576641m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

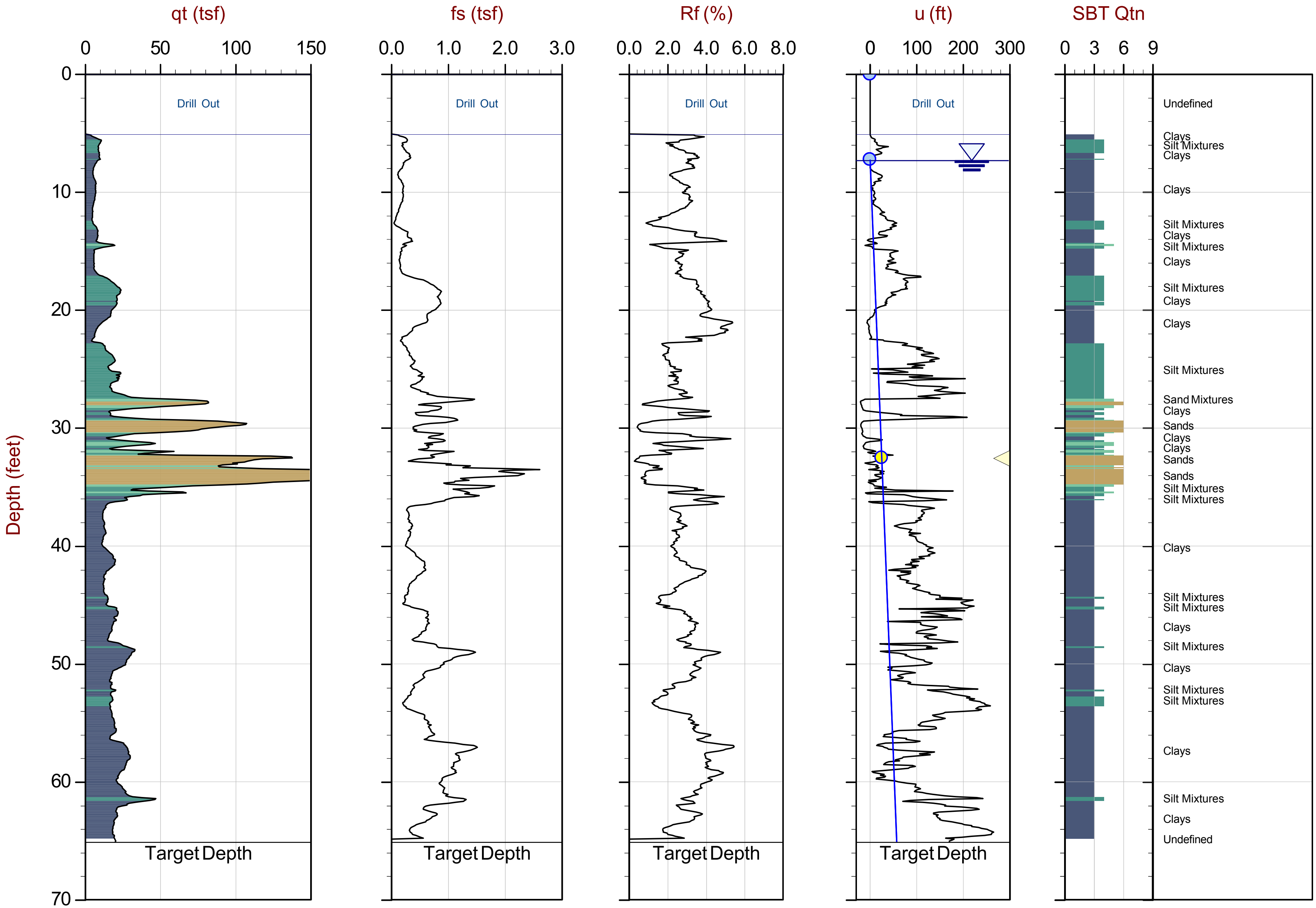
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56172
Date: 2019-11-11 10:24
Site: Project Thunder

Sounding: CPT-06
Cone: 443:T1500F15U500



Max Depth: 19.850 m / 65.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56172_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4147903m E: 576661m

Equilibrium Pore Pressure (Ueq)   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC	Job No:	19-56172
	Client:	Ninyo & Moore
	Project:	Project Thunder
	Start Date:	11-Nov-2019
	End Date:	26-Nov-2019

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-01	19-56172_CP01	15	300	30.43	22.3	8.2
CPT-02	19-56172_CP02	15	200	33.55	26.1	7.4
CPT-03	19-56172_CP03	15	305	29.86	25.1	4.7
CPT-04	19-56172_CP04	15	180	36.33	27.9	8.4
CPT-05	19-56172_CP05	15	335	26.57	19.5	7.0
CPT-06	19-56172_CP06	15	405	32.56	25.3	7.3

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the boring logs in Appendix A.

Moisture Content
The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The test results are presented on the boring logs in Appendix A.

In-Place Density Tests
The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

200 Wash Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The test results are presented on Figure C-1.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-2 through C-12. The test results were utilized in evaluating the soil classification in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits
Tests were performed on selected representative soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure C-13.

Consolidation Test
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures C-14 through C-17.

Direct Shear Tests
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figure C-18 and C-19.

Expansion Index Test
The expansion index of selected materials were evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and inundated with tap

water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure C-20.

Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative samples in general accordance with California Test (CT) 643. The soluble sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-21.

Unconsolidated Undrained Triaxial Compression Tests

A triaxial compression test was performed on selected relatively undisturbed samples in general accordance with ASTM D 2850. The specimens were sheared under compression without drainage at a constant rate of strain shortly after application of a confining stress in a triaxial cell. The test results are presented on Figure C-22.

Unconfined Compression Tests

Unconfined compression tests were performed on relatively undisturbed samples in general accordance with ASTM D 2216. The test results are presented on Figure C-23.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-3	14.5-15.0	Yellowish brown clayey SAND.	90	48	SC
B-4	24.5-25.0	Light brown sandy CLAY.	93	68	CL
B-5	9.5-10.0	Yellowish brown lean CLAY.	100	78	CL
B-5	19.5-20.0	Yellowish brown lean CLAY.	100	75	CL

PERFORMED IN ACCORDANCE WITH ASTM D 1140

FIGURE C-1

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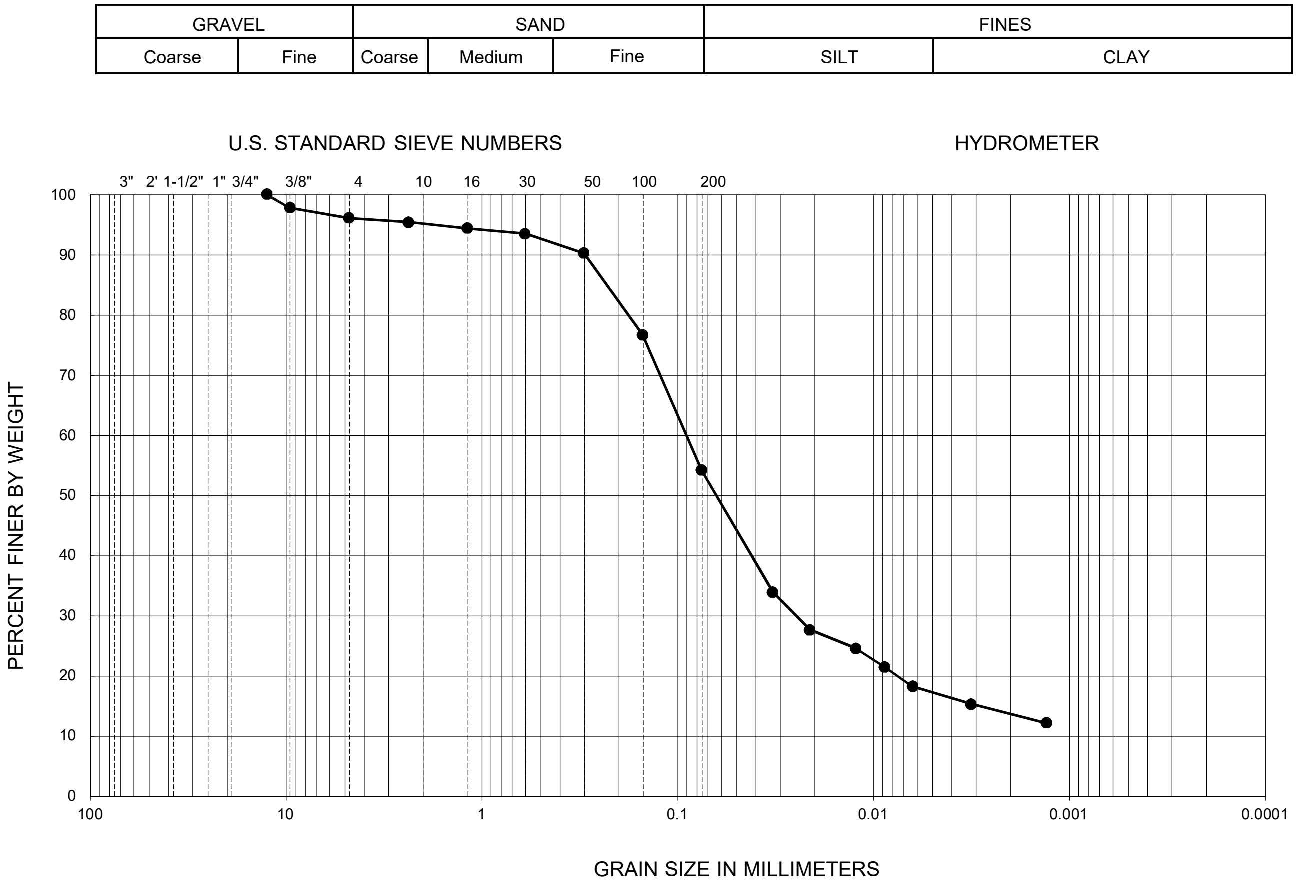
NO. 200 SIEVE ANALYSIS TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	6.0-6.5	28	16	12	--	0.025	0.095	--	--	54	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422

FIGURE C-2

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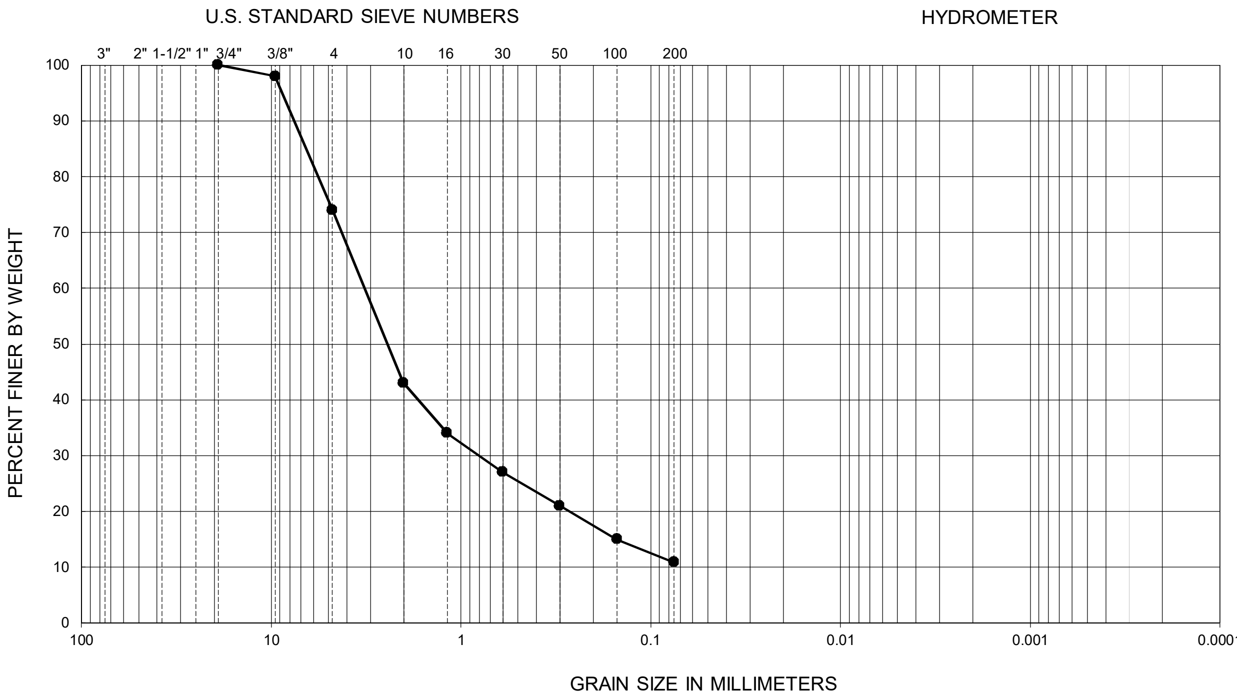
GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	14.5-15.0	--	--	--	--	0.85	3.51	--	--	11	SP-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-3

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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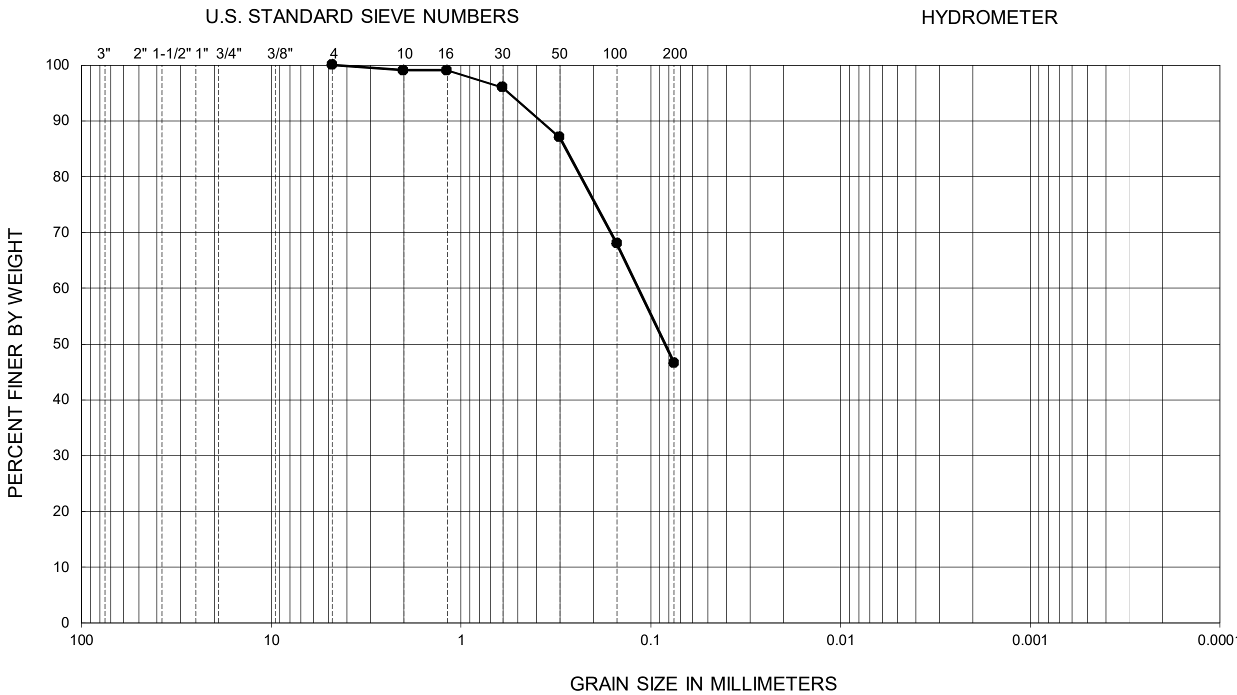
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	35.0-36.5	--	--	--	--	--	0.12	--	--	47	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-4

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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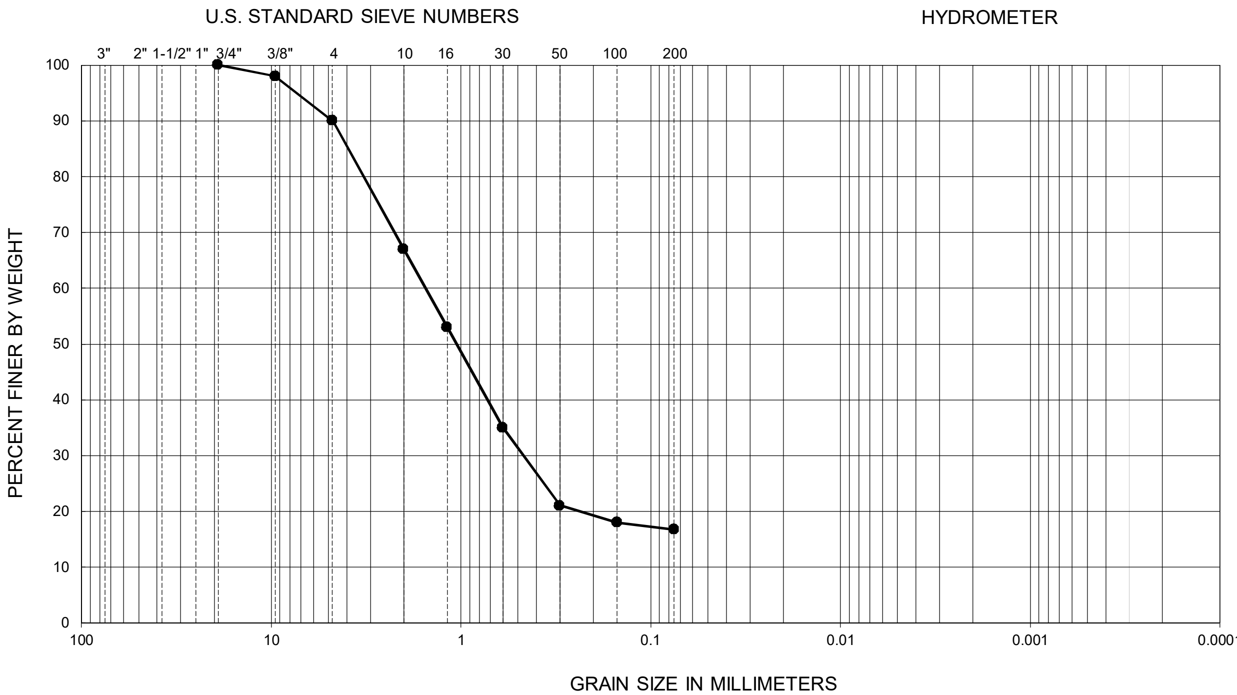
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	15.0-16.5	--	--	--	--	0.49	1.59	--	--	17	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-5

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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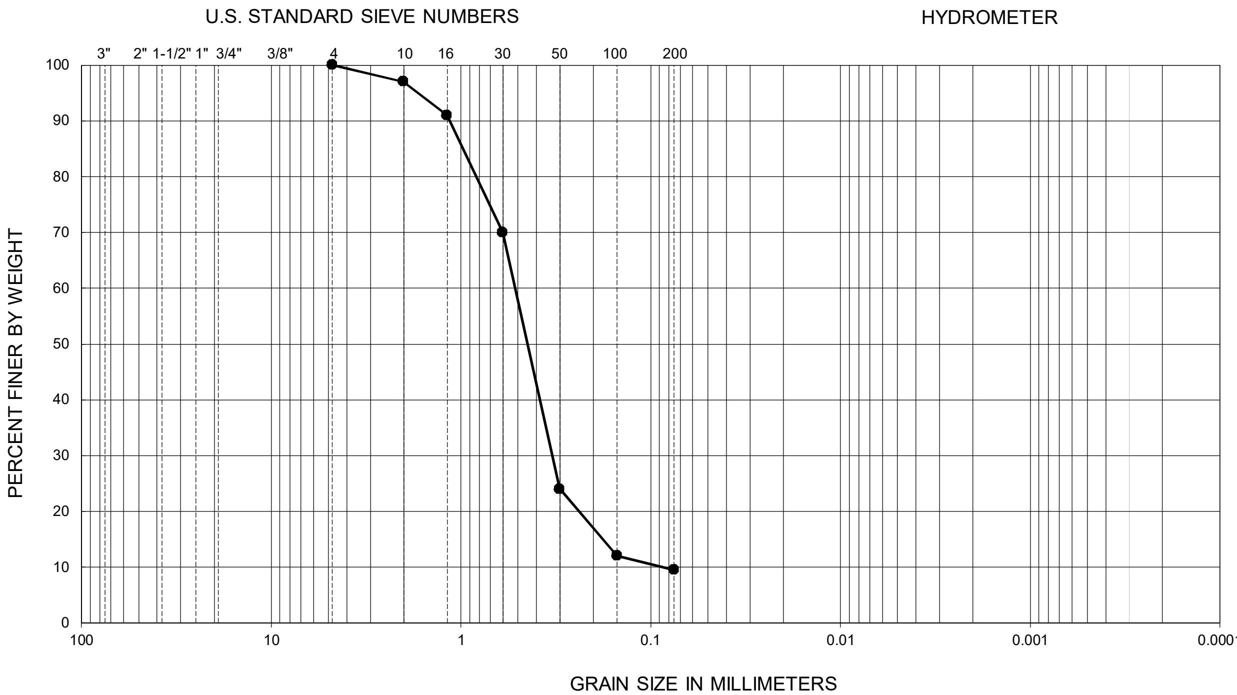
Geotechnical & Environmental Sciences Consultants

403645001 | 5/20

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	34.5-35.0	--	--	--	0.09	0.34	0.53	5.9	2.4	10	SP-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

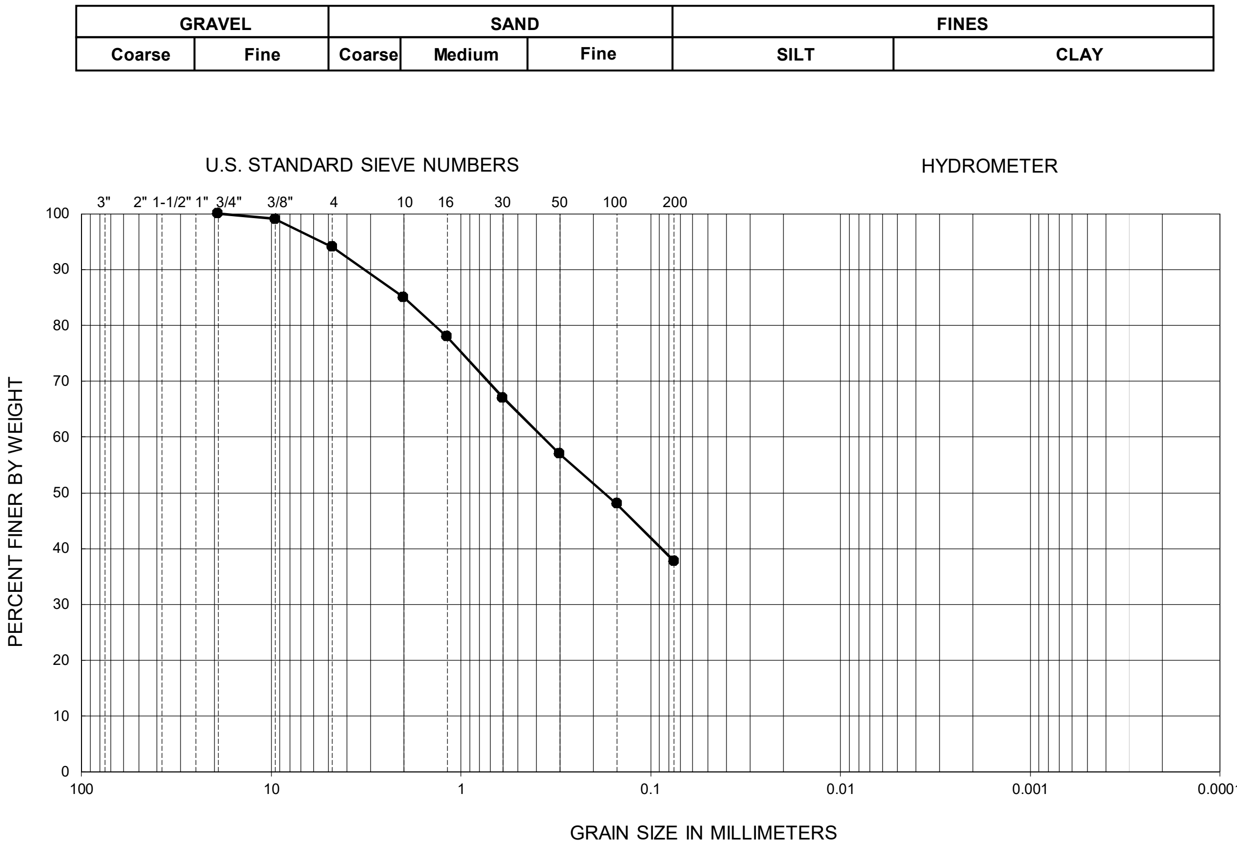
FIGURE C-6

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

403645001 | 5/20



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	6.0-6.5	--	--	--	--	--	0.39	--	--	38	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-7

GRADATION TEST RESULTS

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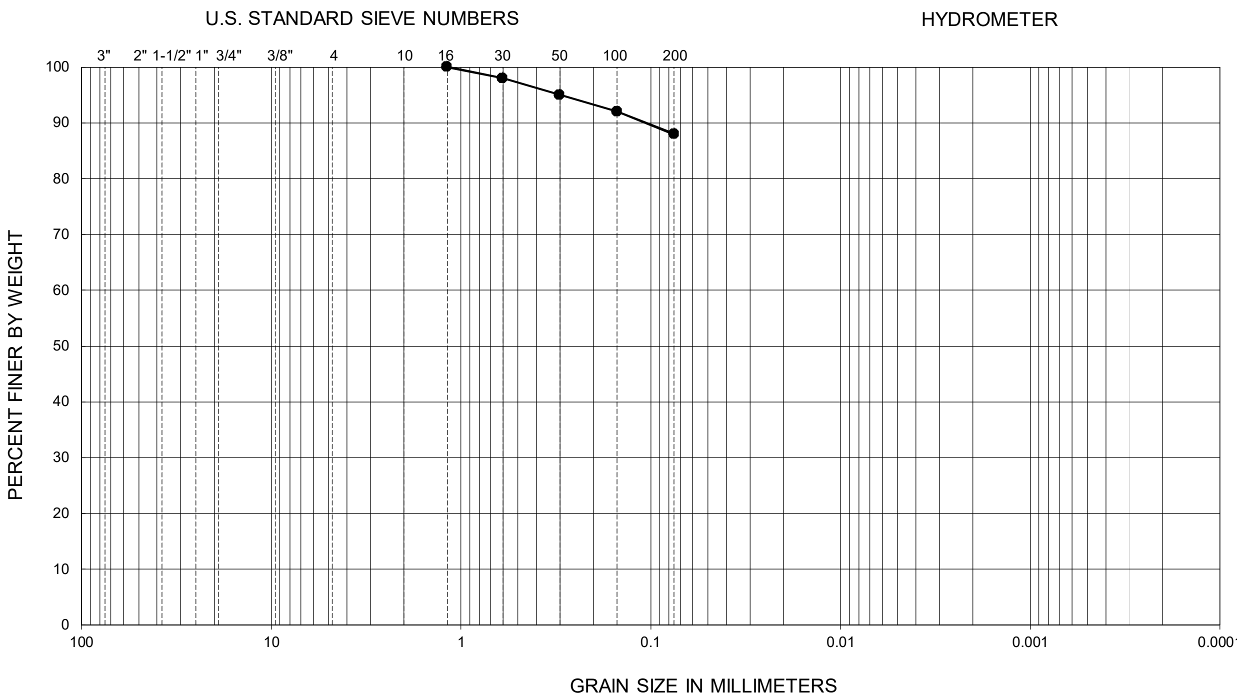
2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	19.5-20.0	--	--	--	--	--	--	--	--	88	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-8

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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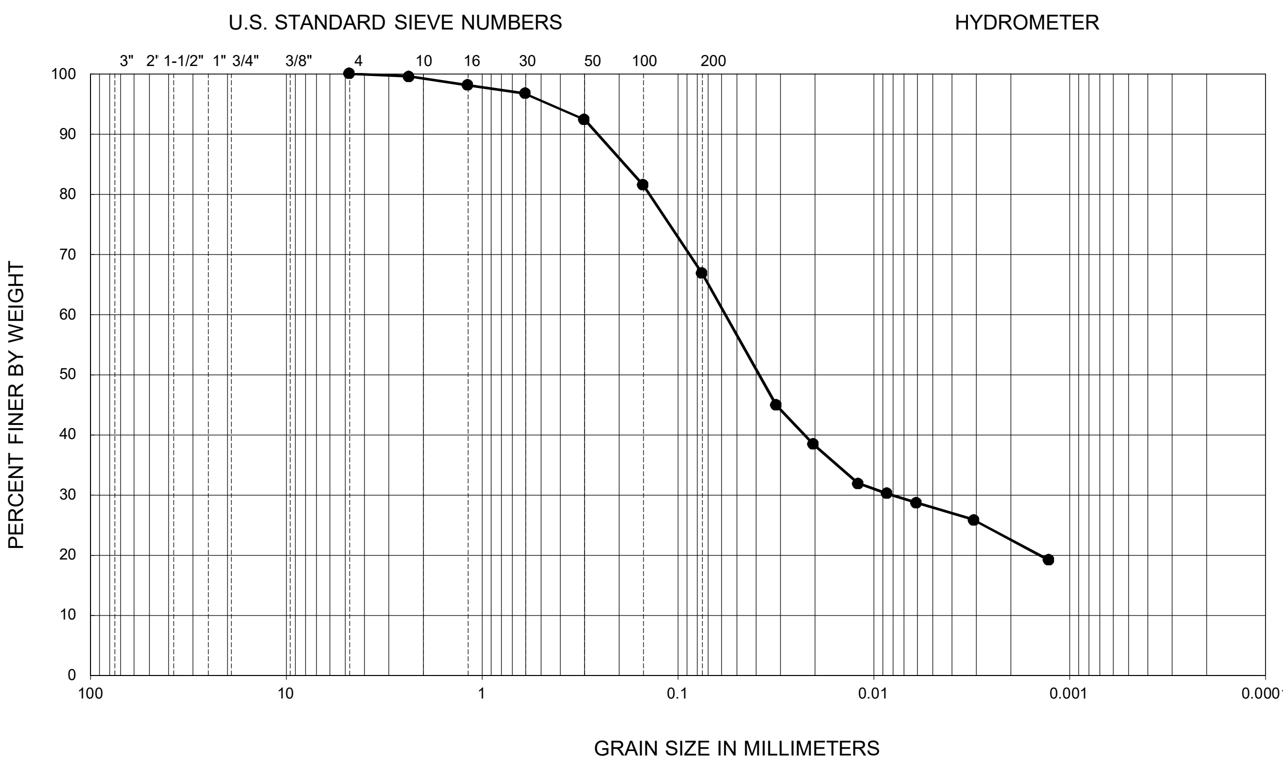
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-4	6.0-6.5	33	15	18	--	0.01	0.06	--	--	67	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422

FIGURE C-9

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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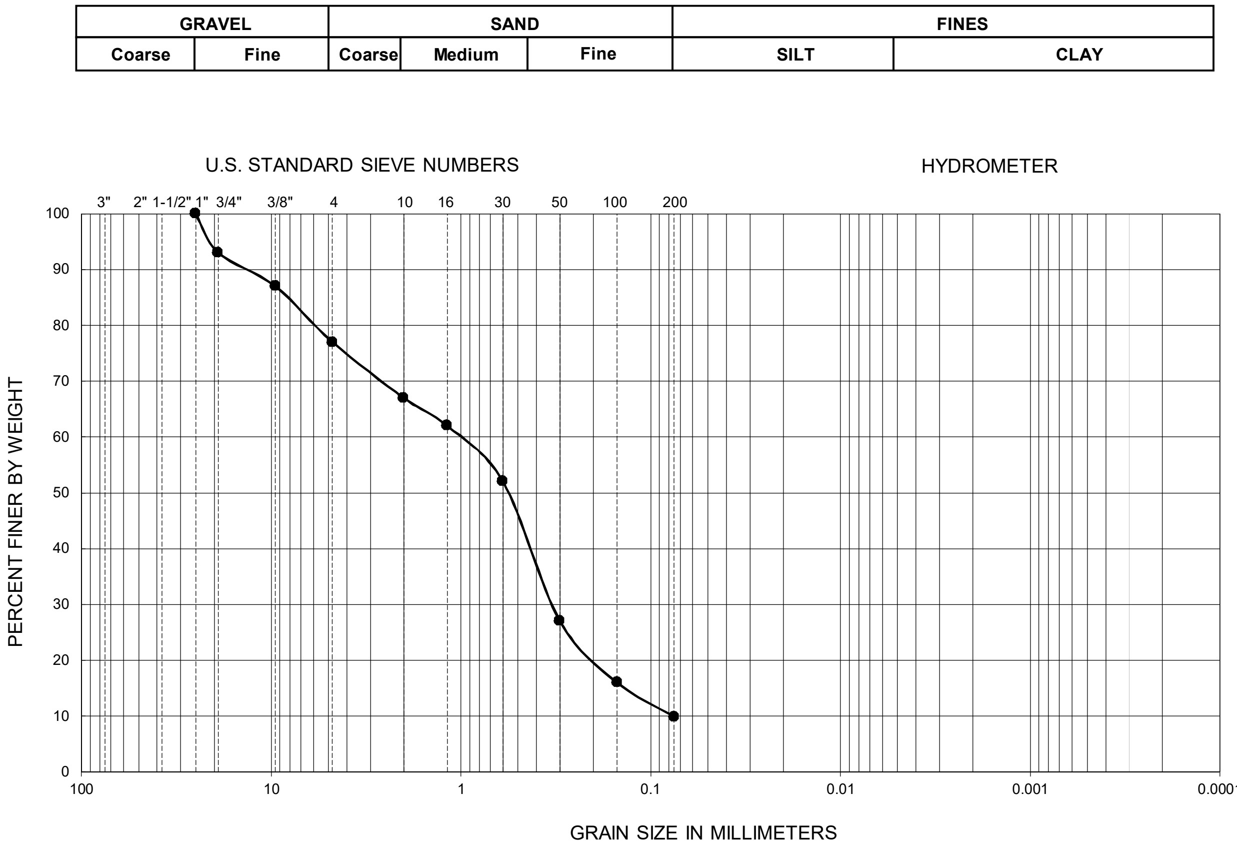

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-4	29.5-30.0	--	--	--	0.08	0.34	1.06	13.8	1.4	10	SW-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-10

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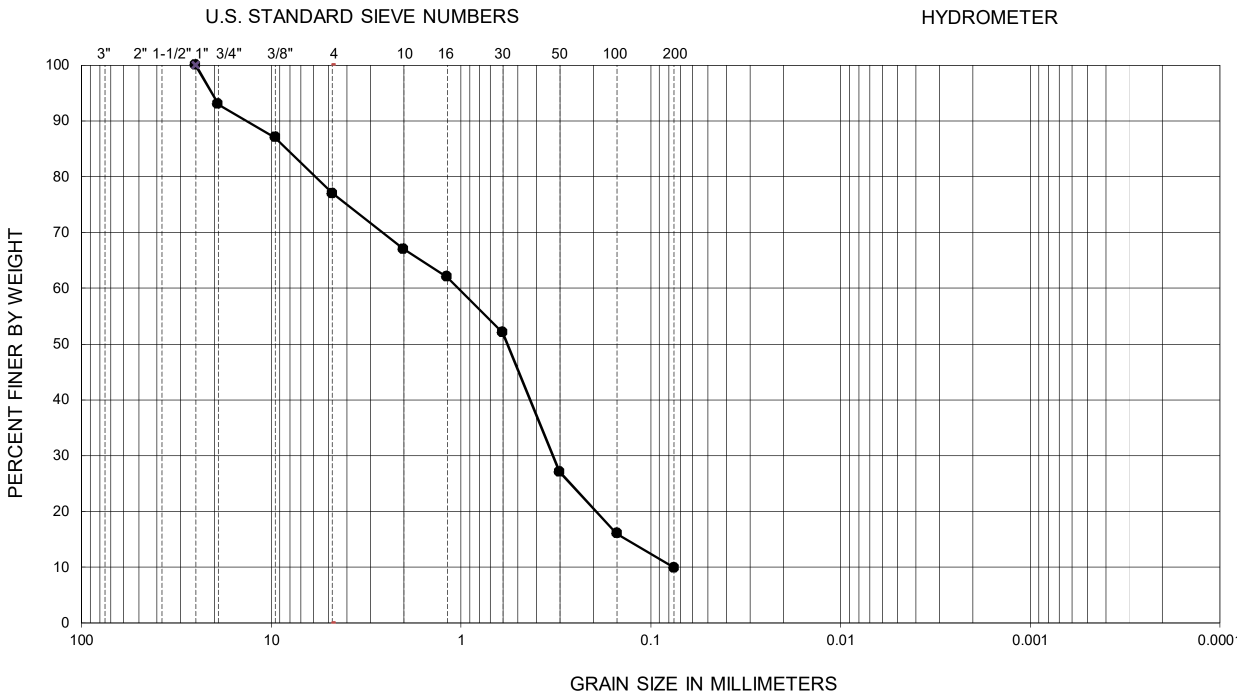
GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-5	29.5-30.0	--	--	--	0.08	0.34	1.06	13.8	1.4	10	SW-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-11

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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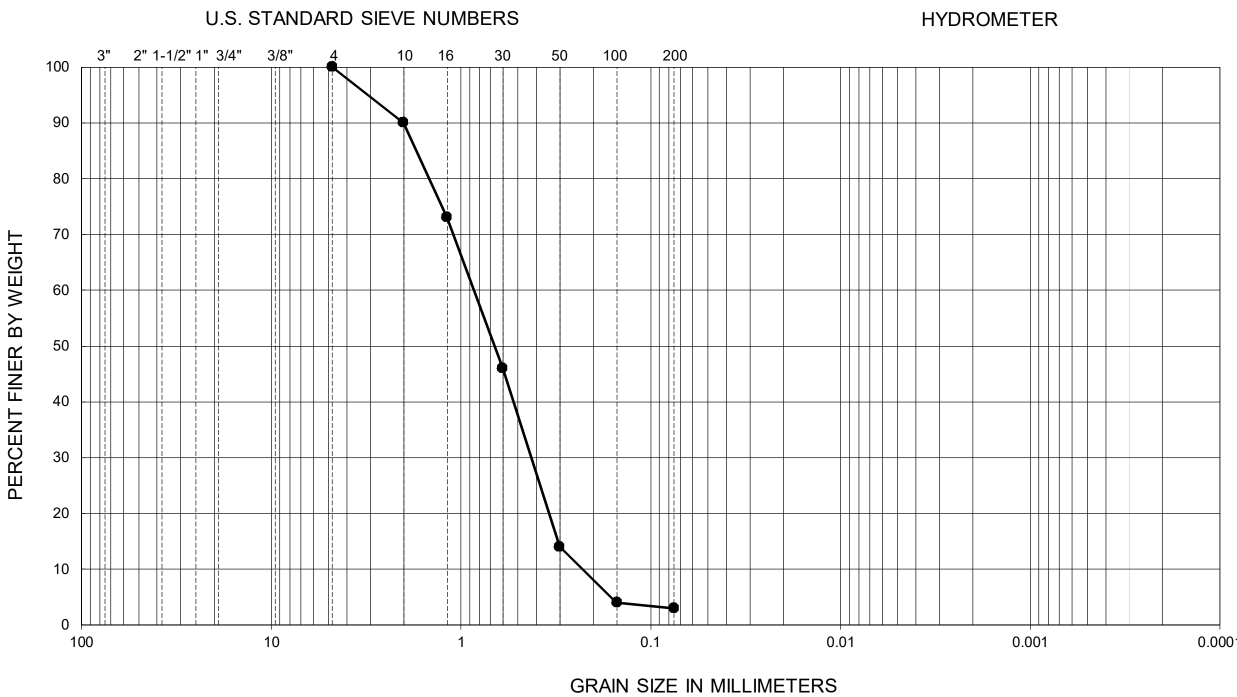
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-5	38.5-39.5	--	--	--	0.24	0.45	0.90	3.8	0.9	3	SP

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-12

GRADATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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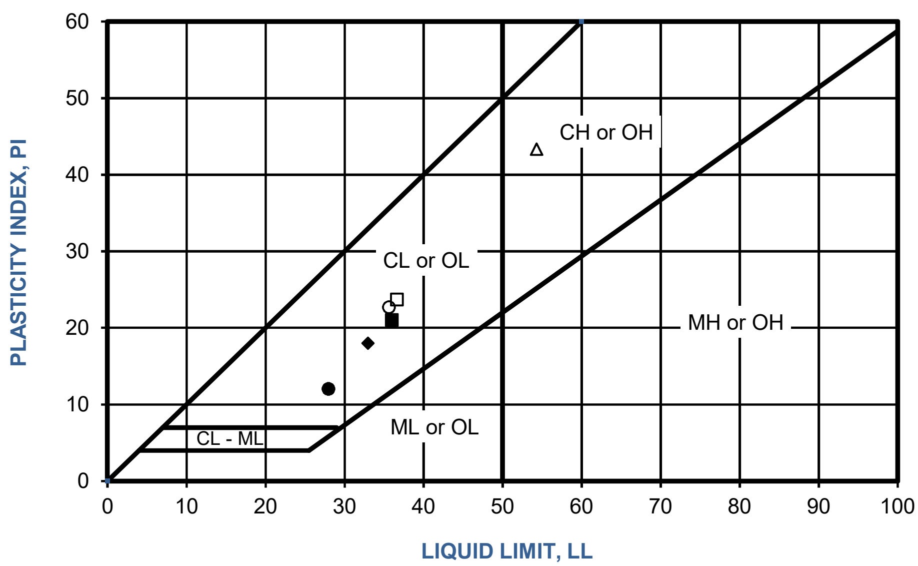
Geotechnical & Environmental Sciences Consultants

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	6.0-6.5	28	16	12	CL	CL
■	B-3	14.5-15.0	36	15	21	CL	CL
◆	B-4	6.0-6.5	33	15	18	CL	CL
○	B-4	24.5-25.0	36	13	23	CL	CL
□	B-5	9.5-10.0	37	13	24	CL	CL
Δ	B-5	19.5-20.0	54	11	43	CH	CH



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE C-13

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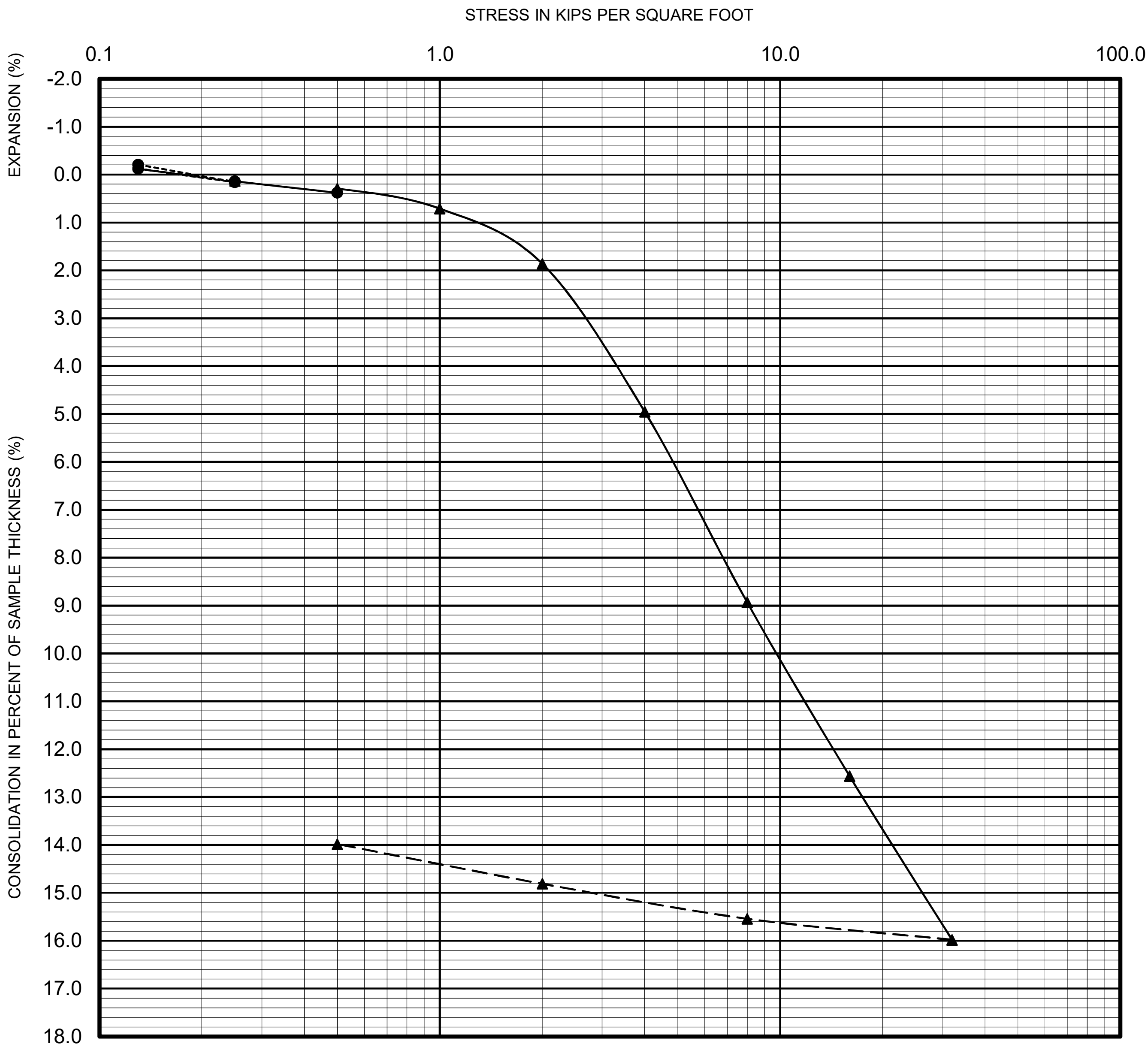
Geotechnical & Environmental Sciences Consultants

ATTERBERG LIMITS TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20



Seating Cycle		Sample Location	B-1
Loading Prior to Inundation		Depth (ft)	2.5-4.33
Loading After Inundation		Soil Type	CL
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-14

CONSOLIDATION TEST RESULTS

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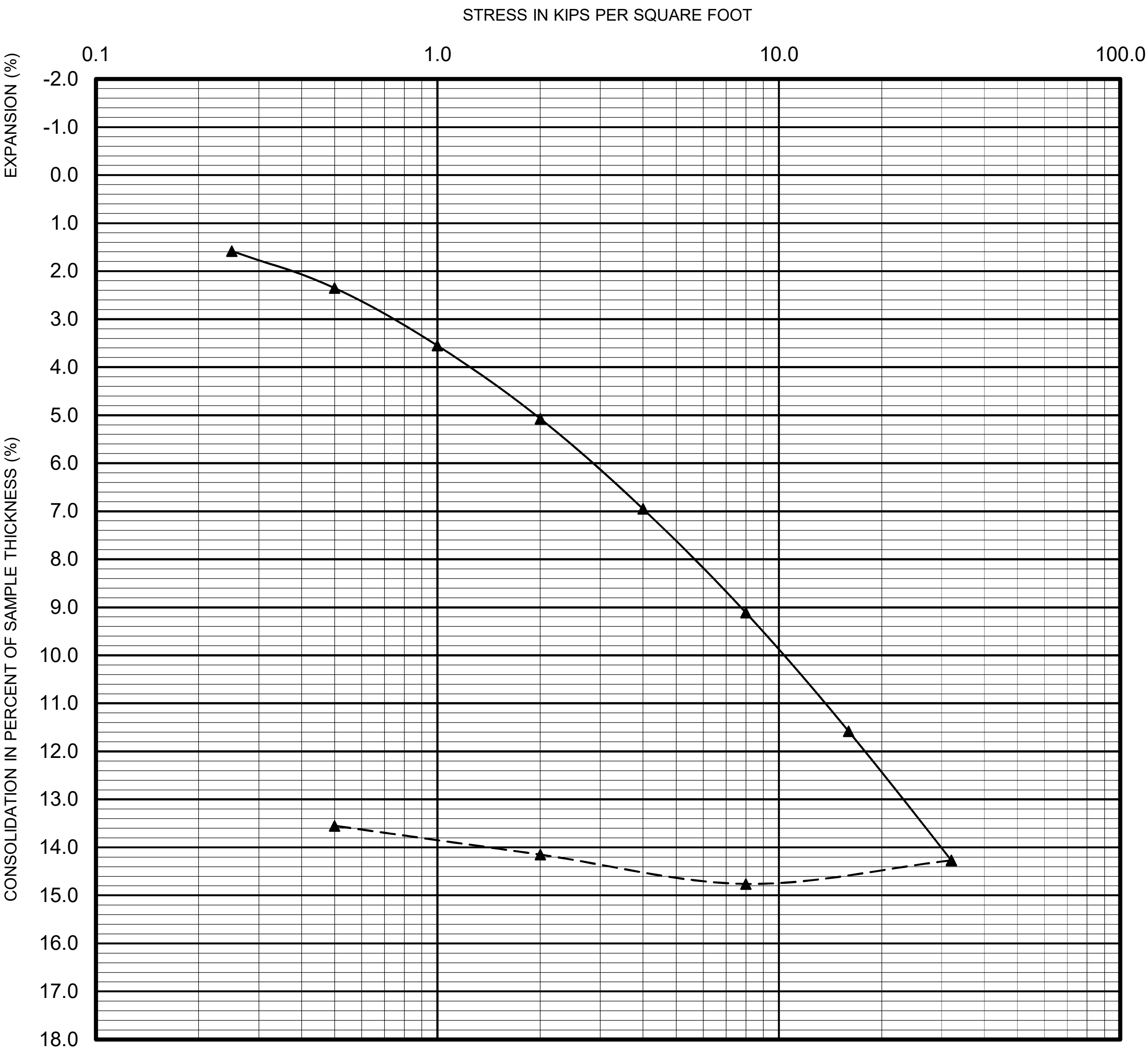
2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20



Seating Cycle		Sample Location	B-4
Loading Prior to Inundation		Depth (ft)	9.5-10.0
Loading After Inundation		Soil Type	CL
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-15

CONSOLIDATION TEST RESULTS

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2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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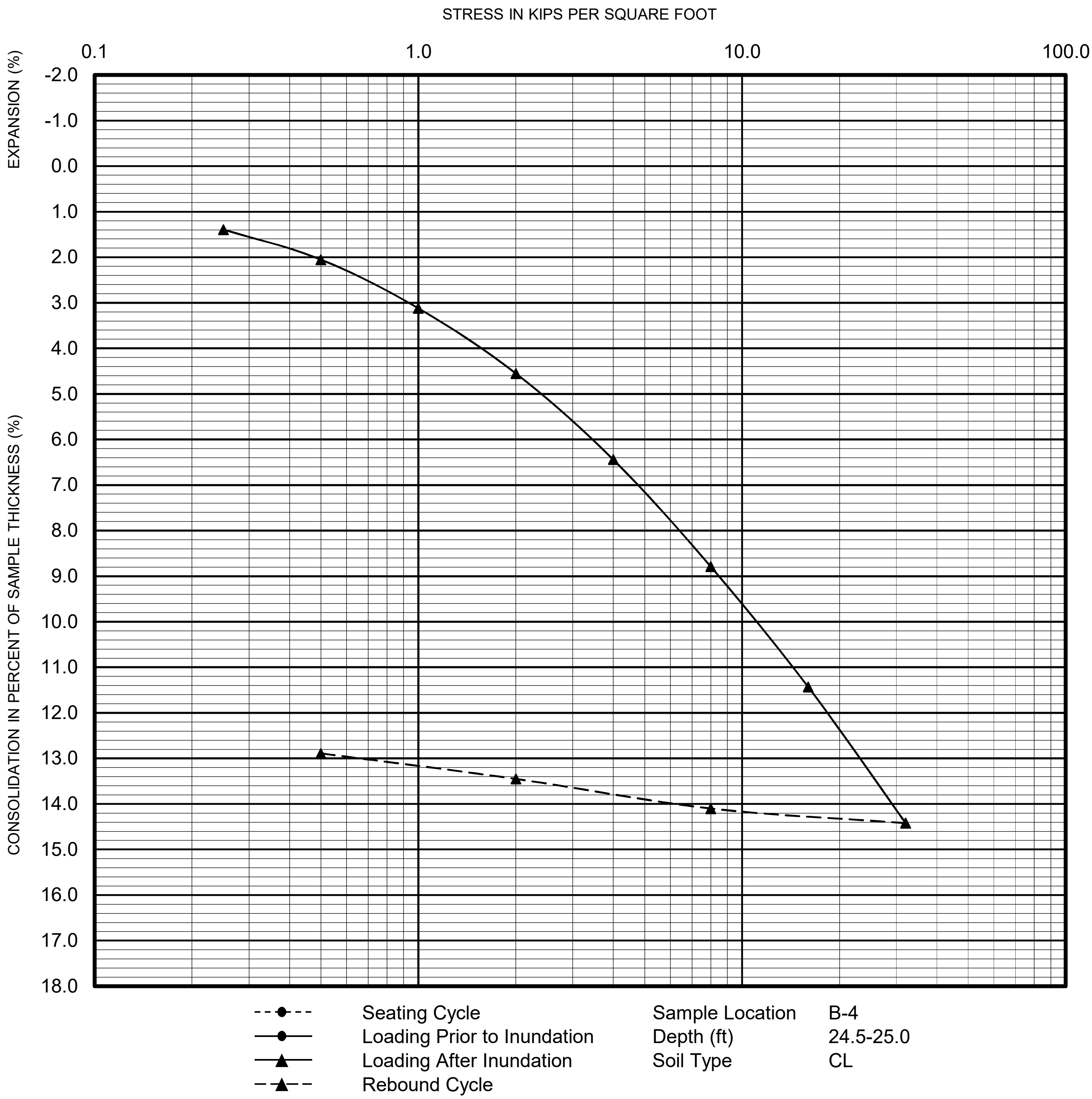

Sample Location	B-4
Depth (ft)	24.5-25.0
Soil Type	CL

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FIGURE C-16

CONSOLIDATION TEST RESULTS

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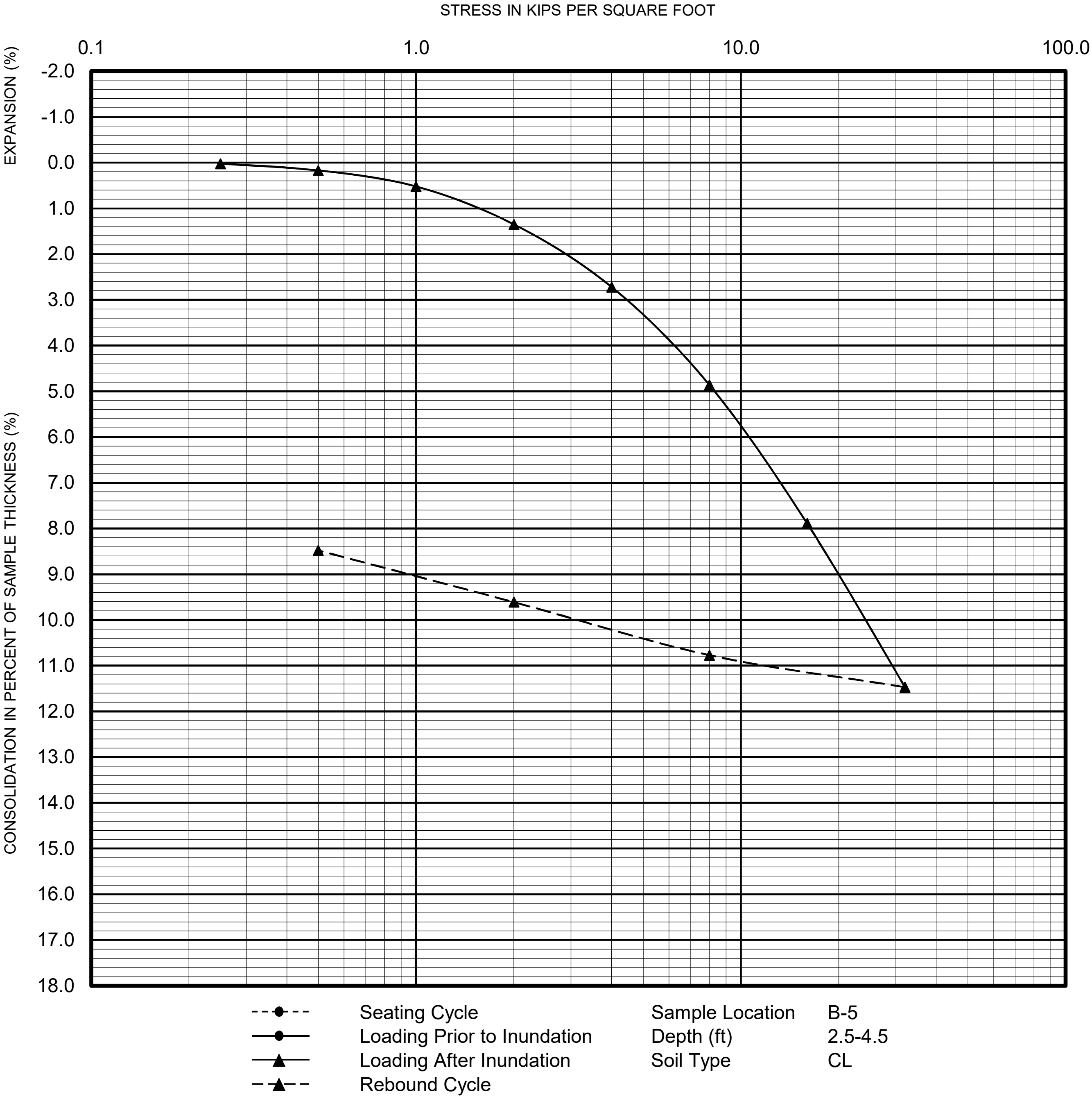

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FIGURE C-17

CONSOLIDATION TEST RESULTS

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EAST PALO ALTO, CALIFORNIA

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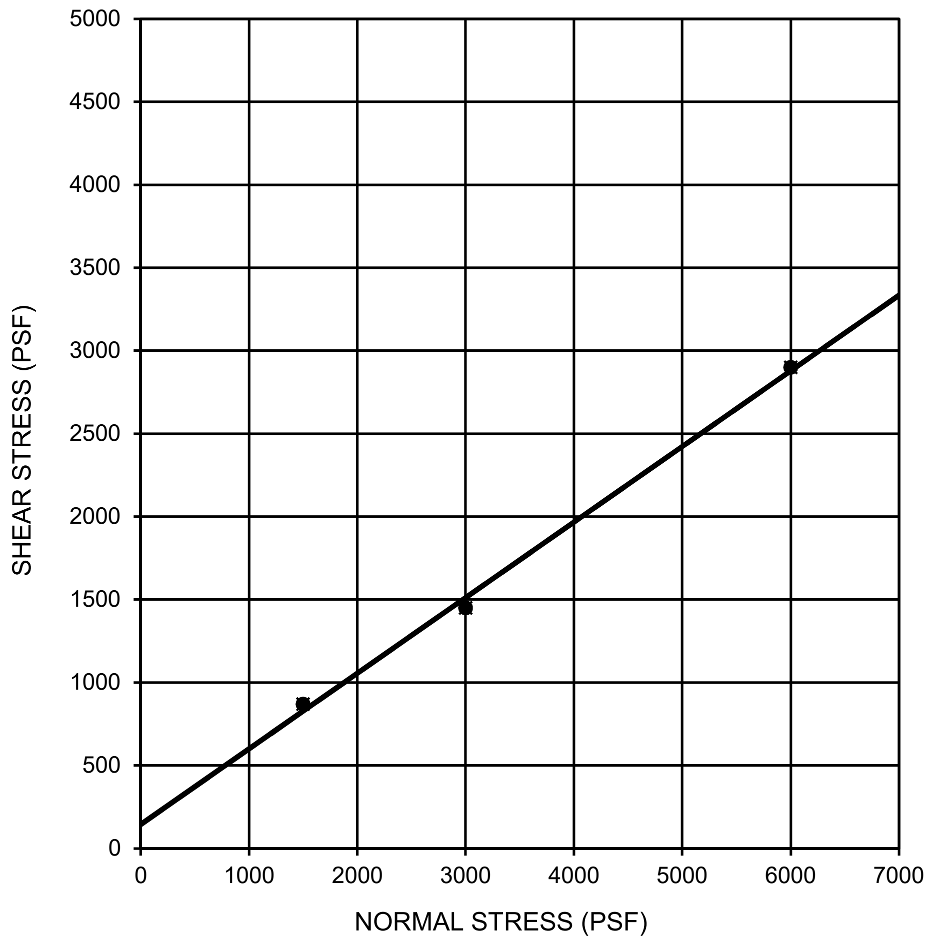

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Clayey SAND with clay	——●——	B-1	14.5-15.0	Peak	145	24	SP-SC
Clayey SAND with clay	– – X – –	B-1	14.5-15.0	Ultimate	145	24	SP-SC

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FIGURE C-18

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DIRECT SHEAR TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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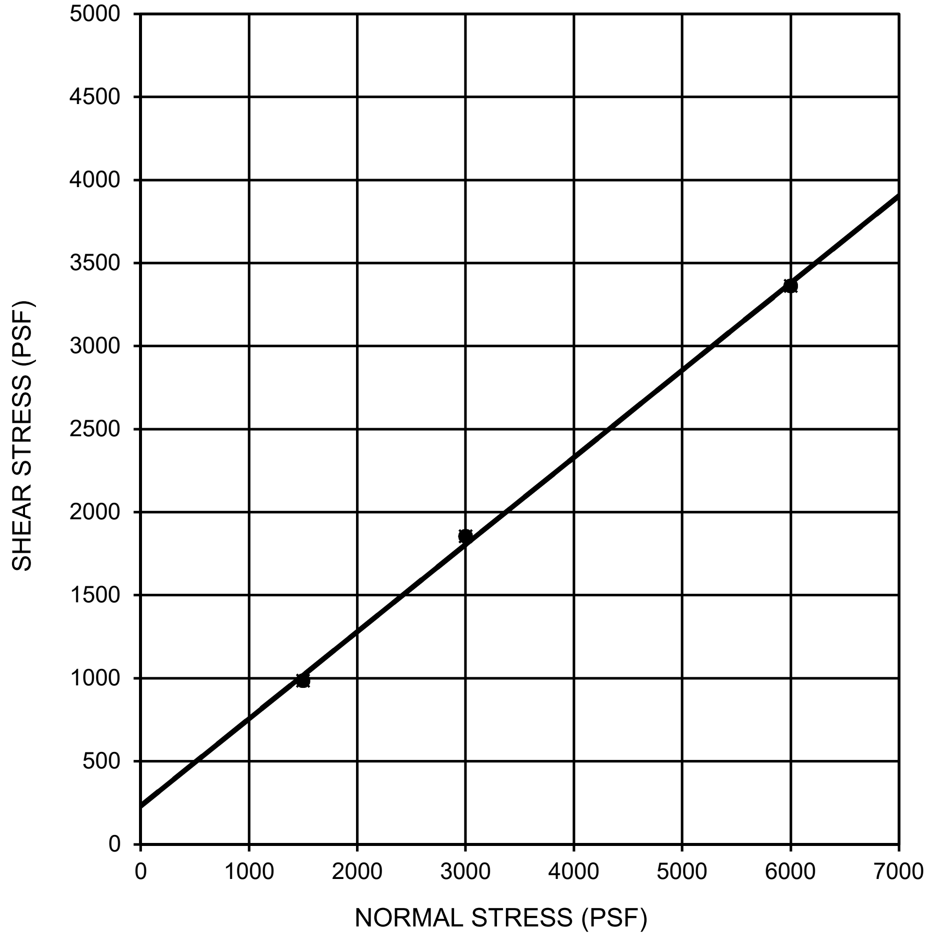

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Clayey SAND	——●——	B-3	6.0-6.5	Peak	232	28	SC
Clayey SAND	- - X - -	B-3	6.0-6.5	Ultimate	232	28	SC

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FIGURE C-19

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DIRECT SHEAR TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	1.0-2.5	12.1	100.7	23.4	0.045	45	Low
B-4	1.0-5.0	9.7	110.6	17.8	0.020	20	Very Low

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FIGURE C-20

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EXPANSION INDEX TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	5.0-6.0	7.2	1,200	240	0.024	780
B-4	1.0-5.0	7.0	950	600	0.060	3300

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE C-21

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CORROSIVITY TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

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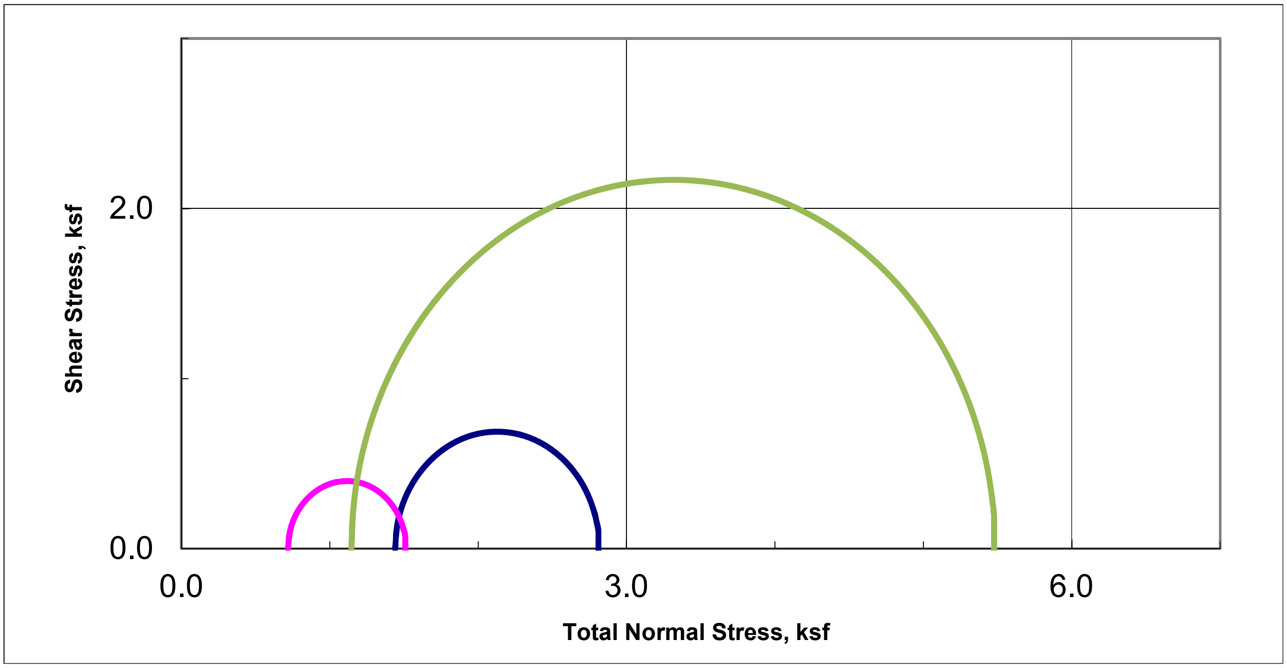

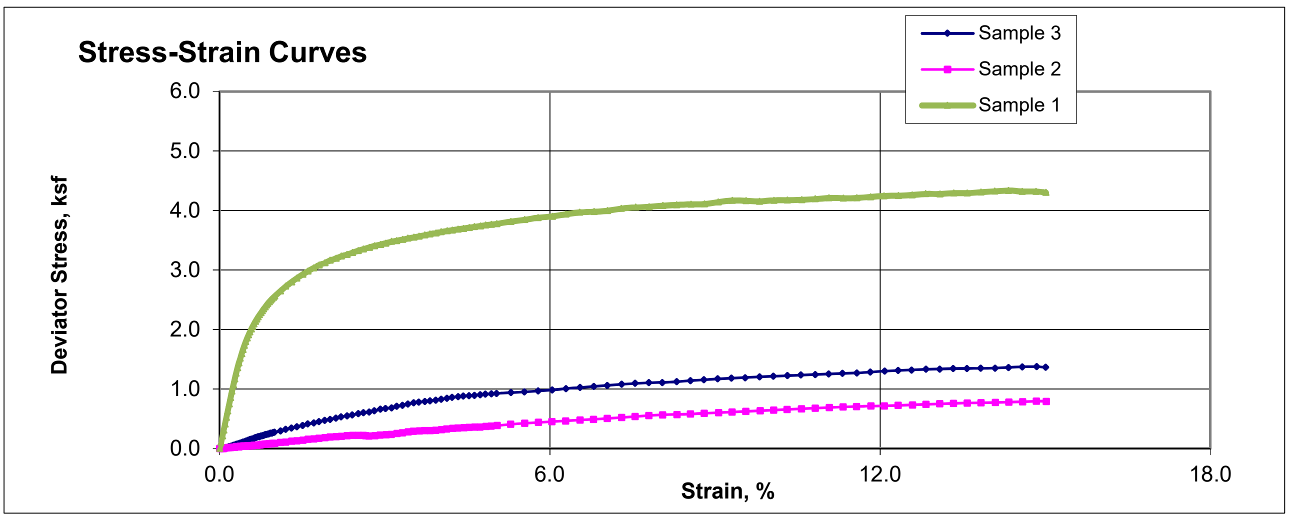

SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH (ksf)
◆	Brown lean CLAY	CL	B-2	19.5-20.0	25.2	99.2	1.15	2.17
■	Brown sandy lean CLAY	CL	B-4	9.5-10.0	23.2	104.8	0.72	0.40
◆	Yellowish brown sandy lean CLAY	CL	B-4	24.5-25.0	--	--	1.44	0.69

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STRAIN RATE: 1.0%/MIN

FIGURE C-22

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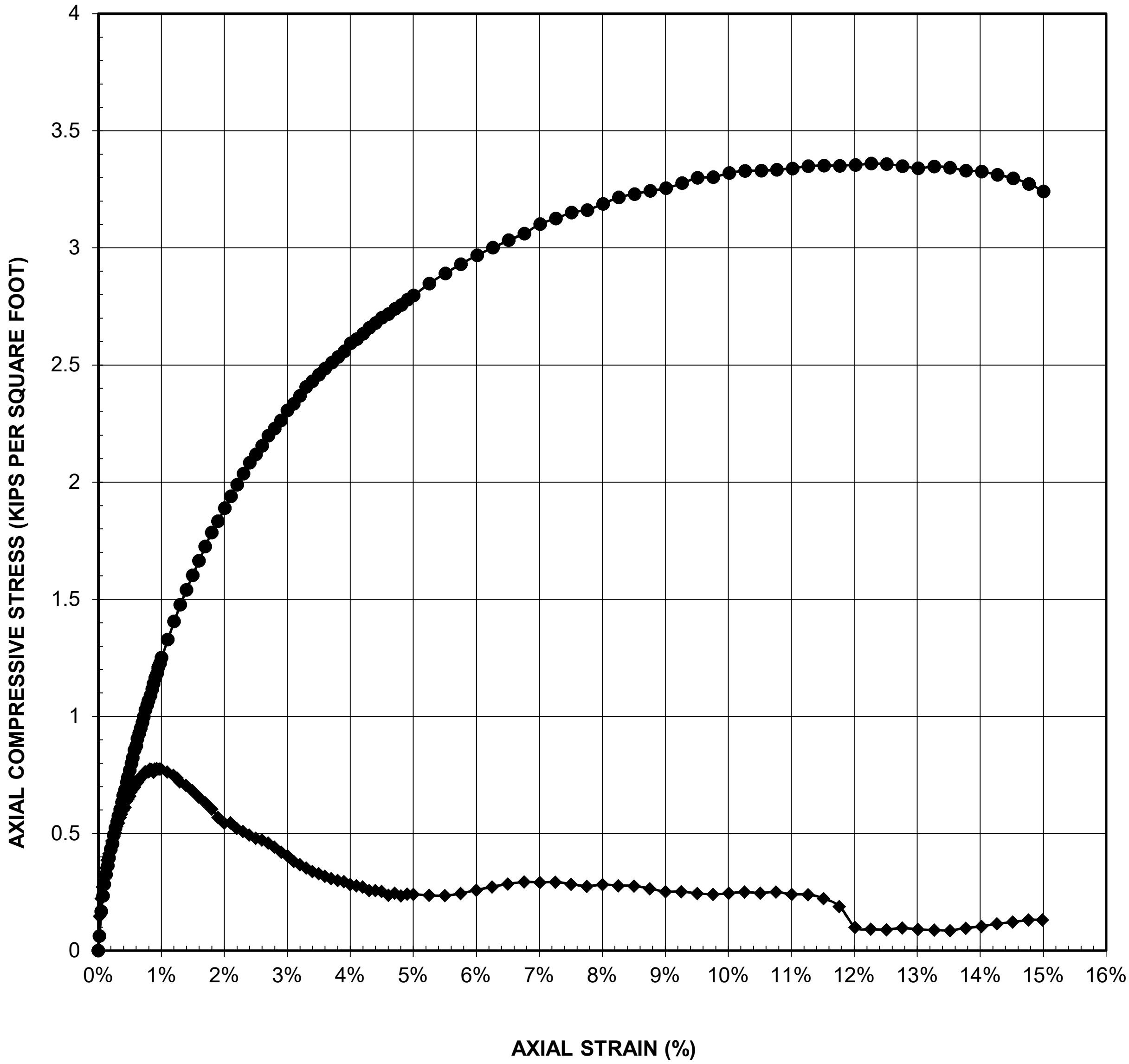
UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY gd, (pcf)	STRAIN RATE (%/min.)	UNCONFINED COMPRSSIVE STRENGTH (ksf)
◆	Brown sandy lean CLAY	CL	B-1	2.5-4.3	17.2	94.2	1.00	0.78
●	Olive gray to black lean CLAY	CL	B-5	2.5-4.5	22.1	104.2	1.00	3.36

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FIGURE C-23

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UNCONFINED COMPRESSION TESTS RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

APPENDIX D

Percolation Testing

APPENDIX D

PERCOLATION TESTING

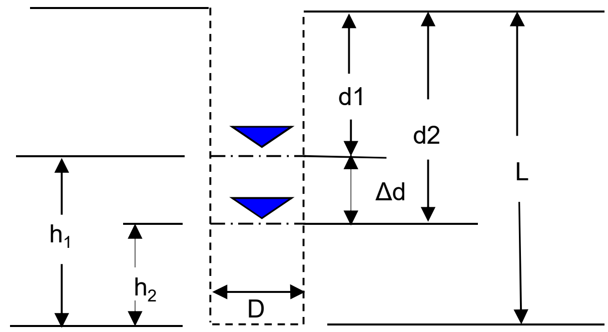
Field Procedure for Percolation Testing

The infiltration characteristics of the site soil were evaluated by field percolation testing. The location of the field percolation test holes is noted on Figure 2. The test hole was excavated with hand tools to a depth of approximately 2 feet, with a diameter of about 8 inches. The subsurface conditions encountered in the test hole consisted of clayey sand. After cleaning the excavation of loose material, water was added to the test hole to achieve a water level approximately 6 inches above the bottom of the test hole. The drop in the water level was recorded over periodic intervals. Water was added to the test hole between measurement intervals to maintain sufficient water levels in the hole for percolation. The percolation rate reported is the percolation rate over the last measurement interval. The infiltration rate is the percolation rate adjusted by a reduction factor to exclude exfiltration occurring through the sidewalls of the test hole. The results of the percolation testing are presented on Figure D-1.

Project =	2519 & 2535 PULGAS AVENUE
Project No. =	403645001
Depth of Boring, L (ft) =	2.0
Diameter of Boring, D (in) =	8.0
Diameter of Pipe (in) =	8.0
Initial Depth to Water, d1 (in), (Final Period) =	18.00
Initial Height of Water, h1 (in), (Final Period) =	6.00
Water Level Drop, Δd (in), (Final Period) =	1.00
Reduction factor, Rf =	2.4

h1 = L - d1 (in inches)

Rf = ((2h1 - Δd)/DIA) +1



Test No. (Hole No.)	Time (hr:min)	Elapsed Time (min)	Depth to Water, d (in)	Water Level, h (in)	Change in Water Level, Δd (in)	Time Interval (hour)	Percolation Rate (inch/hour)	Adjusted Percolation Rate (inch/hour)
P-1	12:00 12:30	30	18.00 20.00	6.00 4.00	2.00	0.50	4.0	1.68
	12:30 1:00	30	18.00 19.00	6.00 5.00	1.00	0.50	2.0	0.84
	1:00 1:30	30	18.00 19.00	6.00 5.00	1.00	0.50	2.0	0.84
	2:00 2:30	30	18.00 19.00	6.00 5.00	1.00	0.50	2.0	0.84
	2:30 3:00	30	18.00 19.00	6.00 5.00	1.00	0.50	2.0	0.84
	3:00 3:30	30	18.00 19.00	6.00 5.00	1.00	0.50	2.0	0.84

FIGURE D-1

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PERCOLATION TEST RESULTS

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

APPENDIX E

Geophysical Survey

APPENDIX E

GEOPHYSICAL SURVEY

Scope

A seismic survey using passive surface wave techniques was performed at the site on November 22, 2019. The survey was performed along one line using passive techniques. The survey line location is noted on Figure 2 of the report. The purpose of the survey was to evaluate the characteristic shear-wave velocity for seismic site classification and to provide a profile of shear wave velocitie with depth at the survey location.

Passive Surface Wave Techniques

The passive surface wave method provided a shear wave velocity model to a depth of approximately 100 feet below the ground surface (bgs) and V_s100 for seismic site classification (CBC, 2019). The passive seismic method carried out included Micro-tremor Array Measurements (MAM) and consisted of one linear profile of seismic data collection. The following sections provide a summary of the methods and analyses used in our study. The seismic model results are provided on Figure E-1.

Field Methods

A Geode 24–Channel Seismograph (Geometrics Inc., San Jose, California) was used for the MAM survey. Twenty four 4.5 Hertz (Hz) vertical component geophone were placed at intervals 15 feet for a total profile length of 345 feet. Approximately twenty-five to thirty records were collected, with a record length of 30 seconds (s) and a 2 millisecond (ms) sampling interval. The field data were digitally recorded in SEG2 format, reviewed in the field for data quality, saved to a hard disk, and documented.

Data Processing and Modeling

The MAM seismic data were processed using SeisImager (Geometrics Inc., San Jose, California) seismic processing software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (longer-period and lower-frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (shorter-period and higher-frequency) surface waves travel relatively shallow within the earth and thus contain more information about velocity closer to the surface. The dispersion is dependent on the material properties, such as surface wave velocity, relative material densities, and Poisson's ratio. An inversion is performed on the collected passive seismic shear wave records within SeisImager to produce a model of the variation in shear wave velocities with depth. The following data processing flow was used to calculate Average Shear-wave Velocities (AVS) to a depth of approximately 100 feet (Vs100).

- Collated records into list file and edited any bad channels or records,
- Applied 2D Spatial Auto Correlation (SPAC); using a linear array and 24 geophones at 15 feet spacing,
- Phase velocity frequency transformation from 2 to 25 Hz
- Automated velocity picks of raw phase velocity were calculated and updated manually,
- Created an initial model and carried out a non-linear Least Squares Method (LSM) inversion to produce a final shear wave velocity model; convergence of the inversion was judged whether the model achieved an RMS <5% within 5-7 iterations,
- Calculated V_s100 using final shear wave velocity model.

Results

Shear wave data resolution generally decreases with depth, due to the loss of sensitivity of the dispersion curve to changes in shear wave velocity as depth increases. Our MAM seismic modeling results are provided on Figure E-1. The scaled figures indicate our interpretation of the approximate changes in shear wave velocity with depth across the surveyed location.

The model results indicate Vs100 values of 1246 feet/sec for MAM-1 (Figure 2). Accordingly, the site is interpreted to have a Seismic Site Classification of Class C.

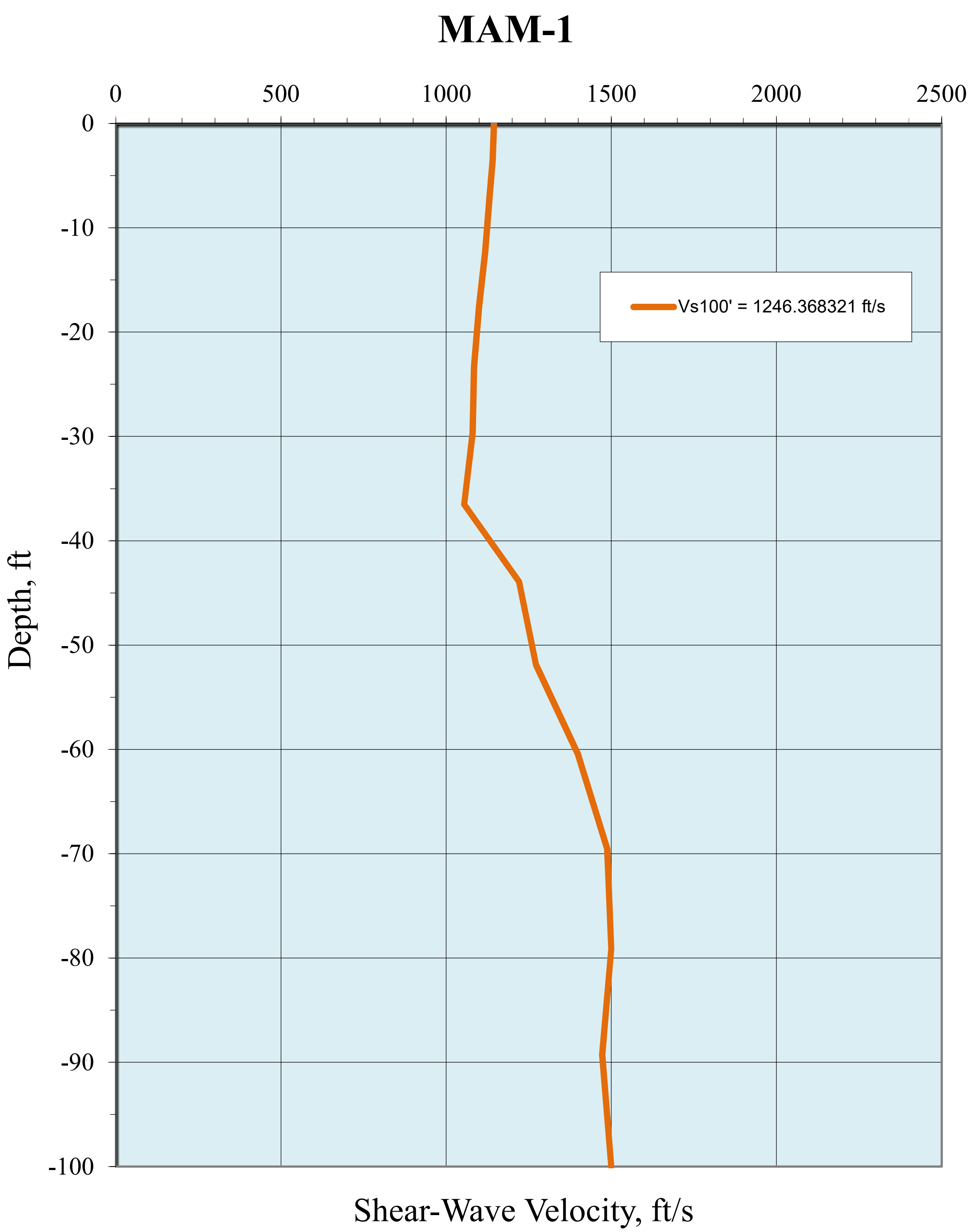

FIGURE E-1

MAM SHEAR WAVE VELOCITY PROFILE

2535 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

403645001 | 5/20

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APPENDIX F

Calculations

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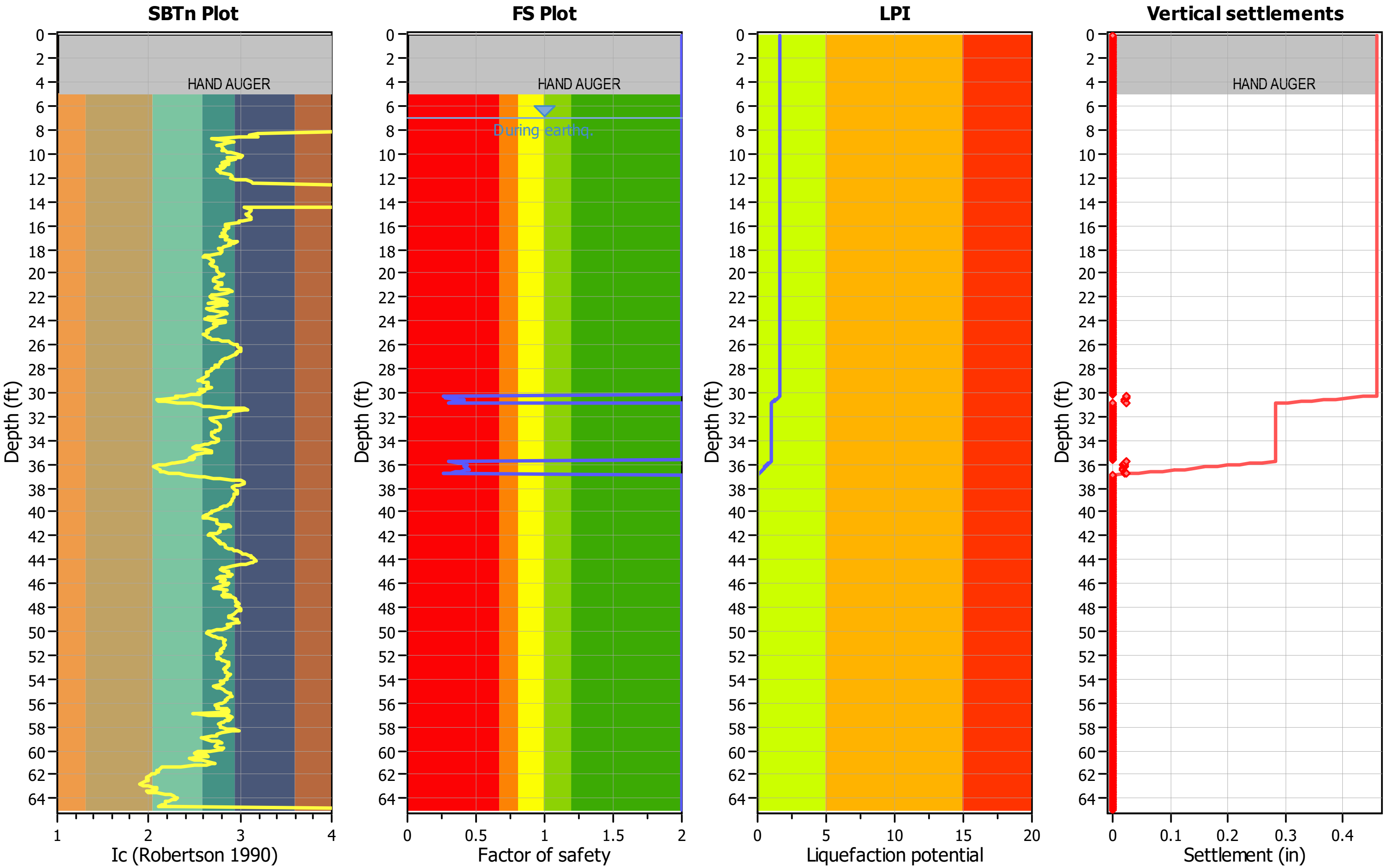
Project: **403645001 - 2519 & 2535 PULGAS AVENUE**
Location: **EAST PALO ALTO, CALIFORNIA**

CPT: CPT-1
Total depth: 64.96 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.20 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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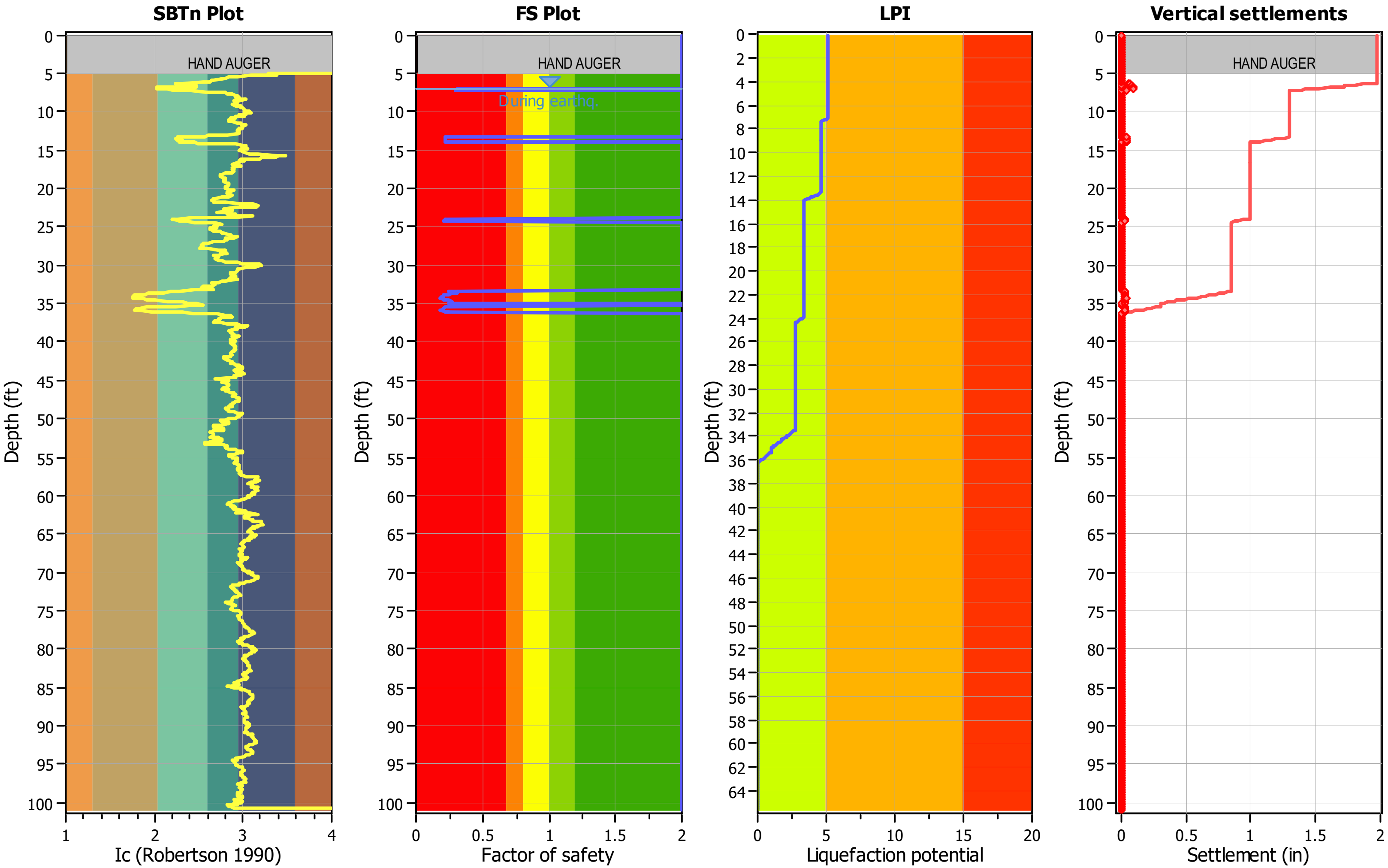
Project: **403645001 - 2519 & 2535 PULGAS AVENUE**
Location: **EAST PALO ALTO, CALIFORNIA**

CPT: CPT-2
Total depth: 100.97 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.40 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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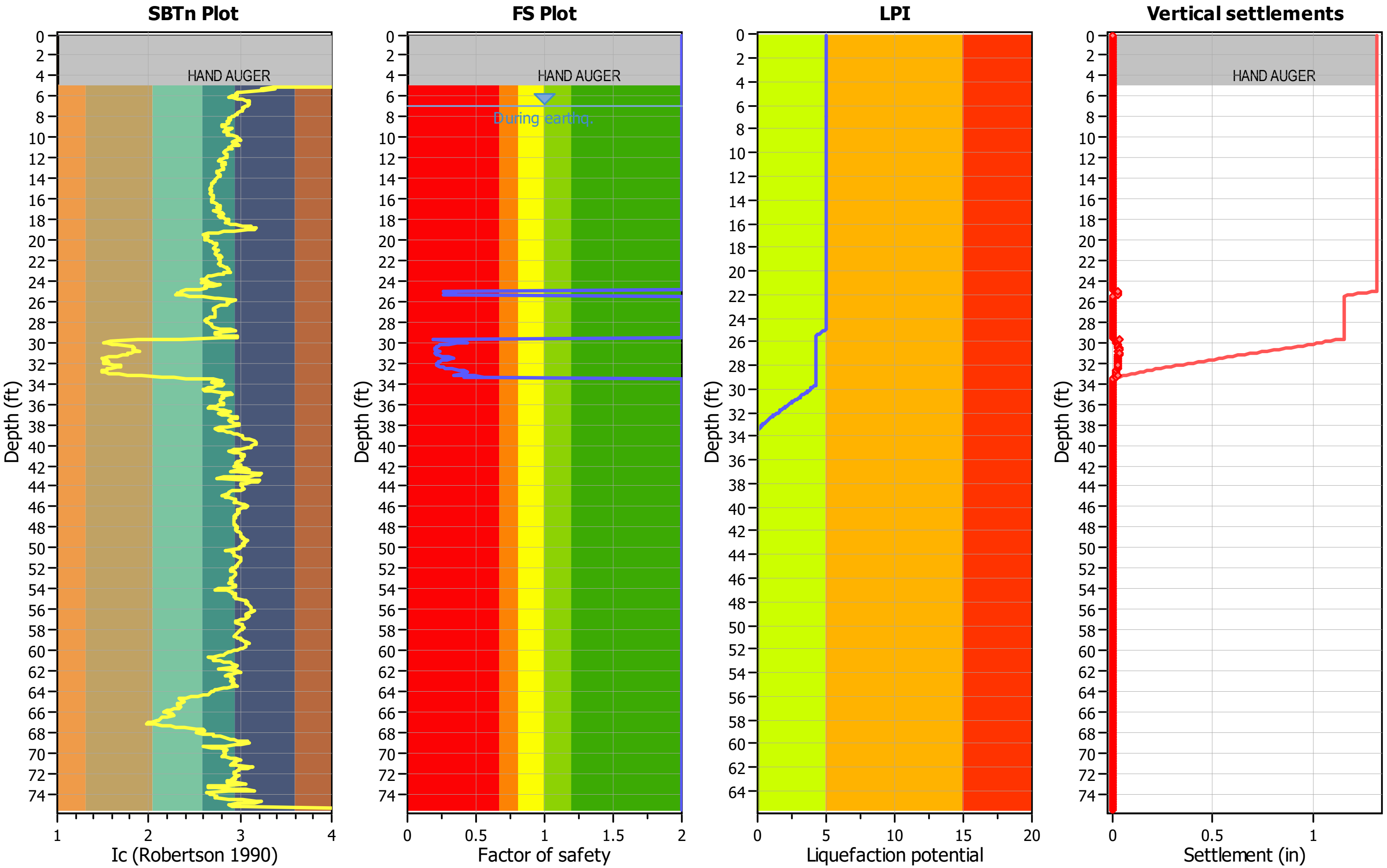
Project: 403645001 - 2519 & 2535 PULGAS AVENUE — **CPT: CPT-3**

Location: EAST PALO ALTO, CALIFORNIA — Total depth: 75.54 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.70 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	Limit depth:	50.00 ft
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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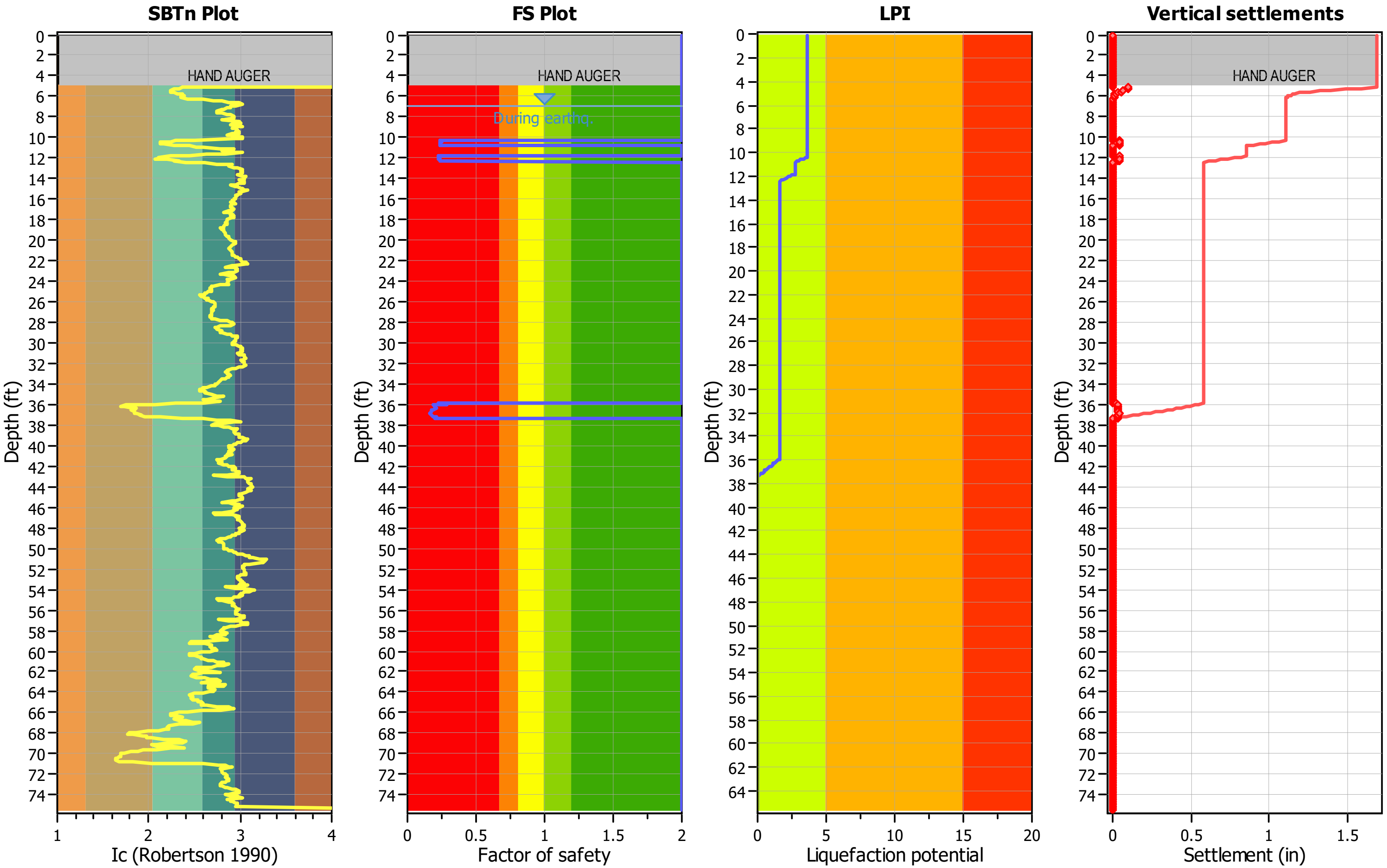
Project: 403645001 - 2519 & 2535 PULGAS AVENUE
Location: EAST PALO ALTO, CALIFORNIA

CPT: CPT-4
Total depth: 75.46 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.40 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	Limit depth:	50.00 ft
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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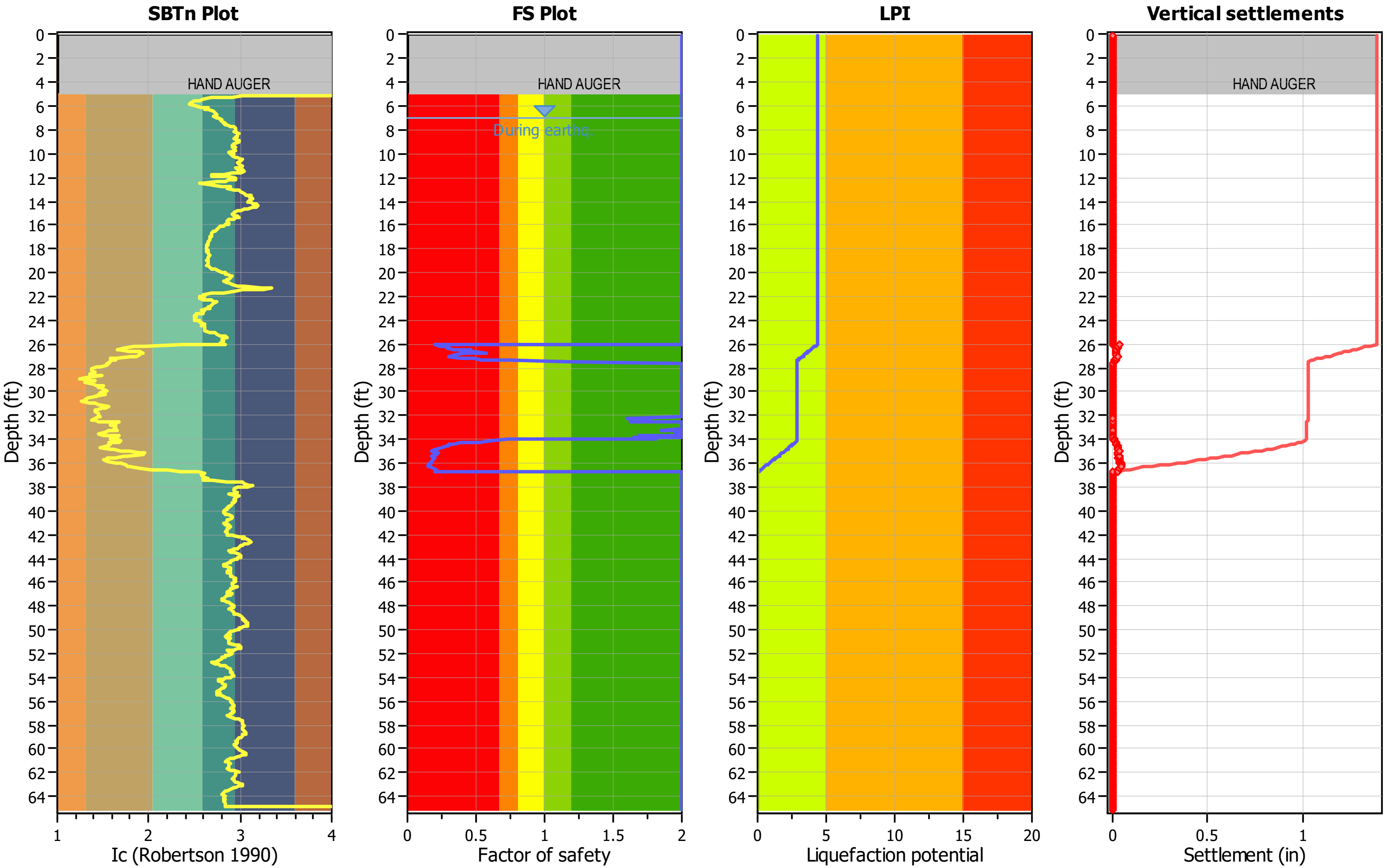
Project: **403645001 - 2519 & 2535 PULGAS AVENUE**
Location: **EAST PALO ALTO, CALIFORNIA**

CPT: CPT-5
Total depth: 65.12 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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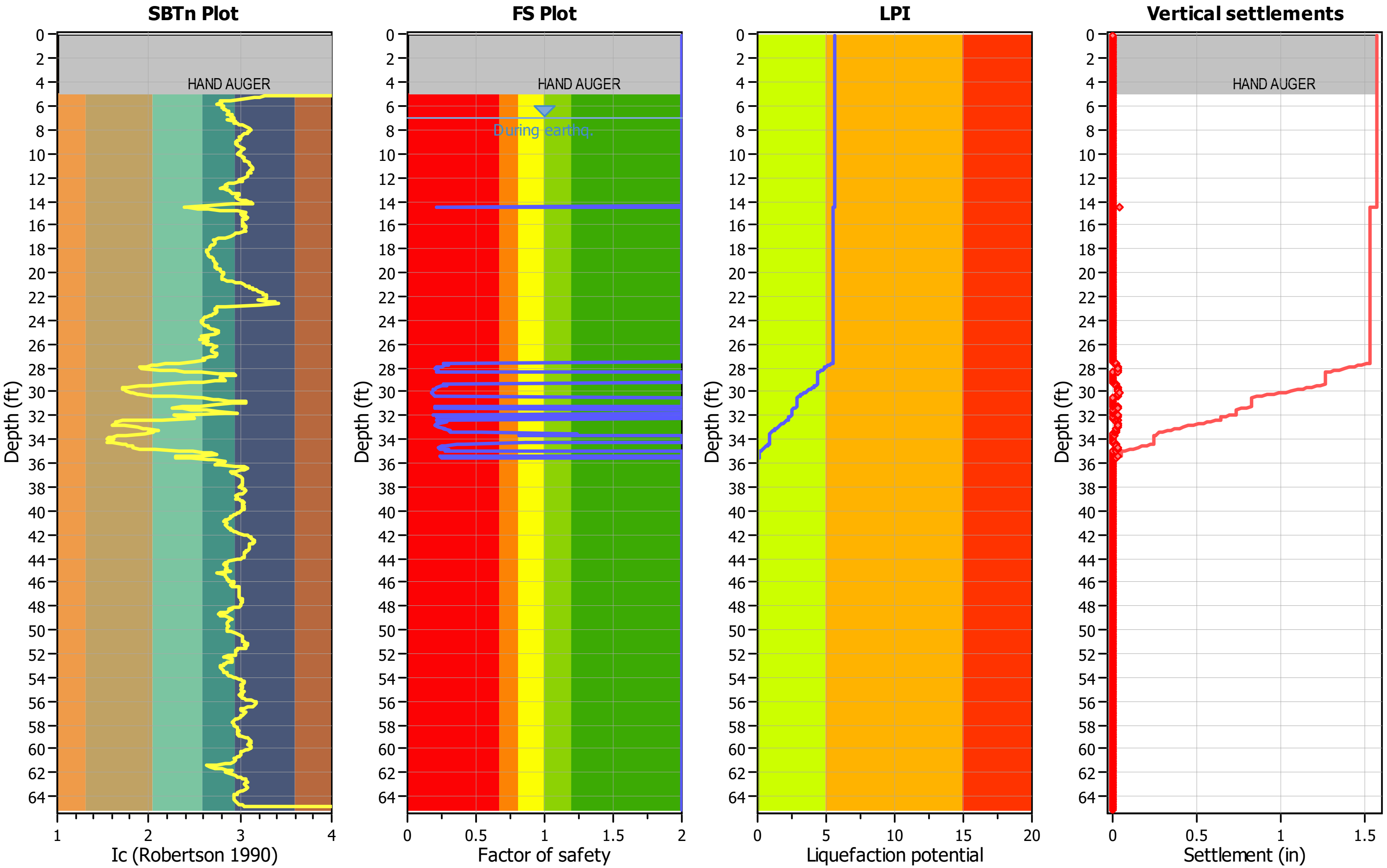
Project: **403645001 - 2519 & 2535 PULGAS AVENUE**
Location: **EAST PALO ALTO, CALIFORNIA**

CPT: CPT-6
Total depth: 65.12 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.30 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.40	Trans. detect. applied:	No	Limit depth:	50.00 ft
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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