GEOTECHNICAL EVALUATION
HILLCREST RESIDENTIAL SUBDIVISION
WATSONVILLE, CALIFORNIA

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Project No. 1680.021

Prepared For:
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Attn: Mr. John Fry

CERTIFICATION

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MILLER PACIFIC ENGINEERING GROUP
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1.0 INTRODUCTION

This report summarizes our Geotechnical Evaluation for the planned Hillcrest Residential Subdivision located on an approximate 12-acre undeveloped lot located on Errington Road in southern Watsonville, California. A Site Location Map is shown on Figure 1. Our services have been provided in accordance with our Agreement dated November 5, 2020. The purpose of our services is to evaluate the site geologic conditions, significant geologic hazards which may affect the project, and provide geotechnical recommendations and design criteria for use in project planning, design and construction. The scope of our services is described in our proposal letter dated October September 28, 2020, and includes the following:

- A brief summary of the geologic setting and seismicity,
- A geologic hazards evaluation and recommended mitigation measures,
- Recommendations for grading (considering value engineering options), including cut slope inclinations, compaction criteria and soil engineering drainage,
- Recommended foundation system, including geotechnical design criteria for shallow and deep foundations,
- Recommendations for retaining walls, including mechanically stabilized earth walls,
- CBC/ASCE seismic design criteria,
- Trench backfill criteria, and
- Access road pavement section.

Phase 1 of our services included a geotechnical peer review of the original subdivision plans, produced by the previous owners, and geotechnical report for the project produced by Cornerstone Earth Group. The results of our review were summarized in a letter dated June 10, 2020, and our opinions / conclusions are summarized in Section 3.0 (below). This report concludes Phase 2 – Geotechnical Evaluation of our services. Future phases of work are anticipated to include a Geotechnical Consultation/Plan Review and Construction Observation and Testing.

2.0 PROJECT DESCRIPTION

Based on our review the preliminary project plans and discussions with you, we understand the existing plan is to construct 144-single-family residences, duplex and townhomes that will be constructed in 5 phases. Significant site grading will be required to develop building pads and allow access to the subdivision. Retaining walls up to 16-feet in height will be constructed along the north and eastern ends of the property to create level areas for a pedestrian trail and "rain-garden". New asphalt paved streets will be constructed to allow access to the residences. New site utilities will also be constructed to provide the subdivision with services. A site plan indicating the approximate extents of the planned improvements is shown on Figure 2.

3.0 DOCUMENT REVIEW

As part our peer review services, we reviewed the project documents that were developed for the previous property owners. This first phase of our work, repeated in the following sections of this report, included reviewing the following documents provided by you:

- Cornerstone Earth Group "Geotechnical Investigation Sunshine Vista Residential Development," February 10, 2017.

- Ifland Engineers, "Sunshine Vista," October 27, 2017, Sheets C2.0, C3.6, C5.0, C5.2, and C9.0.

3.1 Geotechnical Report Review

Previous subsurface exploration by Cornerstone Earth Group and Butano include a total of 22 borings, 11 Cone Penetration Tests (CPTs) and 11 exploratory trenches. Additionally, Cornerstone observed 58-test pits performed by the environmental engineering firm Trinity Source Group. Laboratory testing included moisture content, dry density, percent material passing the #200 sieve, plasticity index and triaxial compression. The boring, CPT and trench logs were provided; however, the logs from the 58-test pits performed by Trinity Source Group were not provided. The results of the previous subsurface explorations are presented in Appendix A.

The Cornerstone report indicates the project site is underlain by 0 to 10-feet of highly expansive fill intermixed with minor to significant amounts of debris consisting of tires, automobile parts, trash, concrete, wood, etc. Very stiff, highly expansive clay underly the fill followed by intermixed layers of very stiff to hard silts with variable amounts of sand, and medium dense to dense silty sands and poorly graded sands.

Cornerstone Earth Group provided various recommendations to develop the project site and construct the proposed improvements. Based on the review of the report, we made the following comments and recommendations:

Slope Stability Analysis – The geotechnical investigation report provides a preliminary slope stability analyses of three cross sections along the northwestern, northeastern, and southeastern corners of the property. The slope stability analyses were performed utilizing the program GSTABL7 and the “Bishop Method” of analysis under both static and pseudo-static conditions. The following recommendations that should be incorporated into the final slope stability analyses.

1. An additional section should be analyzed based on the updated grading plans specifically, where significant fills are proposed. The cross section should extend down to the Watsonville Slough to verify adequate slope stability with the planned fill over native soils.
2. It appears the soil strength data of the native soils is based on two triaxial compression tests located at Boring 4, approximately 200-feet south and west of the Watsonville Slough. Additional strength data appears to be generated from Pocket Penetrometer tests performed during the exploration. We recommend performing additional subsurface exploration and laboratory strength testing in the lower portions of the property along the Watsonville Slough to identify the soil conditions and determine engineering properties for analyses. Shear strength based on the CPT data should be utilized to evaluate the shear strength of the underlying soils.
3. The “Bishop Method” only analyzes circular failures by moment equilibrium when determining the slope stability factor of safety, ignoring the horizontal force equilibrium. We utilized a method that analyzes both the moment and horizontal force equilibrium (i.e., Spencer and Morgenstern & Price methods) and checked for non-circular failure surfaces.

Removal of Existing Fills – The previous report recommends that all the existing fill, 0 to 10-feet thick, should be removed and replaced with compacted fill prior to placing new fill. Although we agree that the existing fill should be removed prior to placing fill if the proposed structures are highly sensitive to settlements, it is our opinion this conclusion is reasonable from a geotechnical standpoint if shallow foundations are utilized to support the structures. Removal and replacement of the existing fills may not be required if structures are supported on a drilled pier foundation system that extends through the fill, provided there is not an environmental health reason to remove the existing fills and fill is capped with at least three feet of clean soil .

Seismic Design Criteria – The report includes 2016 California Building Code (CBC) seismic design criteria. The 2016 CBC was the governing code at the time the report was published; however, the 2019 CBC was officially adopted in January 2020. 2019 CBC seismic design criteria are presented in Section 6.2.

Foundation Recommendations – The previous geotechnical investigation report recommends the proposed structures to be supported on deepened shallow foundations or post tensioned concrete mat slabs-on-grade. We agree that these two options are feasible. However, if at least the upper three feet of expansive soil is replaced, lime treated, or replaced with compacted non-expansive imported fill, the shallow foundations would not require deepening.

Deep foundations may be utilized to support the structures; however, deep foundations (i.e., drilled piers, auger-cast piles, helical anchors, torque down piles, etc.) may be difficult to construct if debris is encountered during construction. A site plan could be prepared based on historic stereo paired aerial photos and compared to current topographic maps to aid in determining the location of potential fill and debris.

If used, deep foundations should be interconnected with grade-beams formed on top of void boxes to prevent uplift pressures from expansive soils impacting the structure. If the existing fill and debris has been removed, the deep foundation system may consist of a helical anchor and grade-beam system.

Retaining Walls – The older plans indicate tiered retaining walls up to 13-feet in height will be constructed and backfilled on the north and eastern sides of the property to create level building pads. It is our opinion a mechanically stabilized earth retaining wall system (i.e., Versa-Lok, Keystone, etc.) would be the most cost-effective retaining wall type to support fills. Additionally, wall heights may be reduced by sloping the fills between the tiered walls to 2:1 (horizontal:vertical). Alternatively, retaining walls may be further reduced or eliminated if the residences are constructed partially on sloping ground or bi-level lots, provided they are supported on a deep foundation system.

Pavement Design – The report provides multiple pavement sections for Traffic Indices (T.I.) and assuming an R-Value of 5 for existing subgrade soils. We agree an R-Value of 5 for the existing highly expansive clayey soil condition is appropriate. However, it is our opinion the subgrade R-Value may be increased by either lime treating or removing the existing soils and replacing with imported non-expansive soils. Both options were recommended in the geotechnical investigation report. Additionally, the R-Value may be increased by installing a geotextile on the subgrade level prior to placing baserock. Pavement recommendations are presented in Section 6.8.

3.2 Civil Plan Review

Based on our review of the preliminary site and grading plans, significant site grading would be necessary to develop the site with anticipated cuts up to 10-feet and fills up to 25-feet. Retaining walls up to 13-feet were proposed to support fills along the northern and eastern sides of the property. The previously planned finished grades of the subdivision are fairly level with elevations between 65- and 52-feet above sea-level. A small extension of Loma Vista Drive is planned for access to the future subdivision at the western end of the property.

It is our opinion there could be significant cost savings if the grading plan is modified to allow the site to retain its overall general slope inclination downward from the southwest to the northeast. Moderate site grading would still be required to remove existing fill, if necessary, create roads, and prepare building pads. If this option is pursued the lots would “step-down” with the overall grade. The thick fills and tall retaining walls would not be necessary along the northern and eastern property lines. The residences located on the northern and eastern property lines may consist of split-level homes constructed on grade with some minor site grading required and possibly shorter retaining walls to create level backyard space. These residences may need to be supported on a drilled pier foundation system. Additionally, a sanitary sewer pump may be required to due to grade differences.

4.0 SITE CONDITIONS

We performed a site reconnaissance on December 18, 2020 to observe existing conditions at the site. The project site is located on a relatively level knoll. The area has recently been cleared with a majority of the surface now covered in exposed soil, low grasses, and large shrubs. Mature trees line the northern and eastern perimeter of property lining the bank of the Watsonville Slough that bounds the property along the northern and eastern sides of the site. Existing residential subdivisions are located along the western and southern property lines.

In the proposed building area surface elevations range between 50 to 70-feet above sea-level. The northern and eastern ends of the development area are set atop, up to an existing approximate 10- to 15-feet tall 2:1 (horizontal:vertical). A relatively level “bench”, approximately 20 to 50-feet wide, is located below this slope. An additional 10- to 20-foot tall 5:1 to 2:1 slope is located below this intermediate slope and terminates at the Watsonville Slough.

4.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

As shown on Figure 3, geologic mapping (USGS, 1997), indicates the site is underlain by Quaternary Watsonville Fluvial (map symbol Q_{wf}) and Basin (map symbol Q_b) deposits. Typically, fluvial deposits consist of poorly sorted, semi-consolidated, sand, silt, and gravels deposited by river or stream action. Basin deposits typically consist of highly plastic silty clay with interbedded layers of sands and gravels deposited at base of estuaries, lagoons, lakes, etc.

4.2 Seismicity

The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. An “active” fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no evidence of recent rupture.

4.2.1 Active Faults in the Region

The California Department of Conservation, Division of Mines and Geology has mapped various active and inactive faults in the region (CDMG, 1972 and 2000). These faults are shown in relation to the project site on the Active Fault Map, Figure 4. The Zayante Vergeles Fault is the nearest known active fault and is located approximately 4.3-kilometers (2.7-miles) northeast of the site (Google Earth, 2020).

4.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. A map showing the epicentral locations of significant earthquakes in the Bay Area between 1985 and 2016 is shown on Figure 5.

4.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the “Working Group on California Earthquake Probabilities” (USGS 2003, 2008; Field et al 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS (Aagaard, et. al., 2016) indicate the highest probability of a M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system, located approximately 67.0-kilometers (41.5-miles) northeast of the site, at 33%. The San Andreas Fault located approximately 8.1-km (5.0-miles) northeast of the site is assigned a 22% probability of rupture resulting in a M>6.7 or greater earthquake. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

4.3 Site History

Based on our review of readily available historic aerial photographs (Google Earth, 2021) the site was vacant in 1952. The next available aerial photo taken in 1968 indicates the site had been developed as an automobile salvage yard. The automobile salvage yard remained until 2017 when the automobiles and associated structures were removed from the site, leaving the area denuded of vegetation. Currently vegetation has begun to establish on the project site.

4.4 Subsurface Conditions and Groundwater

Based on our review of the geotechnical investigation, presented in Appendix A, the site is underlain by 0 to 10-feet of highly expansive fill intermixed with minor to significant amounts of debris consisting of tires, automobile parts, trash, concrete, wood, etc. Very stiff, highly expansive clay underly the fill, with intermixed layers of very stiff to hard silts with variable amounts of sand, and medium dense to dense silty sands and poorly graded sands to the maximum depth explored. Laboratory testing to determine soil permeability is beyond our current scope of work; however, the surficial soils consist of highly plastic clays. Typically, highly plastic clay soils exhibit very low infiltration rates and tend to hold and pond surface and subsurface water.

Groundwater was observed during the previous subsurface exploration at a depth of about 30 feet, which corresponds to an elevation of about +0 feet. Groundwater levels typically fluctuate with the seasons with higher levels anticipated during the winter months. Cornerstone Earth Group anticipates the groundwater levels will be dependent on the water level of the adjacent Watsonville Slough with a historic high groundwater level at an elevation of +11 feet.

5.0 GEOLOGIC HAZARDS EVALUATION

The principal geologic hazards which could potentially affect the project site include strong seismic shaking, lurching, slope instability and erosion. Other hazards, such as fault surface rupture, and liquefaction are not considered highly significant at the site. More detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

5.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. The project site is not located with an Alquist-Priolo Earthquake Fault Zone, and the nearest known active fault to the site, the Zayante-Vergales, lies approximately 3.8-kilometers (2.4-miles) to the northeast. Therefore, we judge the risk of fault surface rupture at the site is low.

Evaluation: *No significant impact.*
Recommendations: *No mitigation measures are anticipated.*

5.2 Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site.

5.2.1 Deterministic Seismic Hazard Analysis

Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson, Silva & Kamai, Boore, Stewart, Seyhan & Atkinson, Campbell & Borzognia, and Chiou & Youngs, (2014)), for a weathered rock subsurface condition, were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake, probable median accelerations and plus one standard deviation ($+1\sigma$), peak ground accelerations (PGA) for earthquakes on faults near the site are shown in Table A.

TABLE A
DETERMINISTIC PEAK GROUND ACCELERATION
Hillcrest Residential Subdivision
Watsonville, California

Fault	Fault Distance[1]	Moment Magnitude[1]	Median PGA[1,2,3,4]	$+1\sigma$ PGA[4]
Zayante-Vergales	3.8 km	6.9	0.44 g	0.74 g
San Andreas	8.1 km	8.0	0.40 g	0.68 g
Calaveras	8.5 km	6.9	0.33 g	0.56 g
Sargent	14.2 km	6.7	0.23 g	0.40 g
San Gregorio	22.7 km	7.4	0.22 g	0.37 g
Monterey Bay	23.7 km	7.2	0.20 g	0.34 g

Reference:
1. Google Earth (2020)
2. Abrahamson, Silva and Kamai (2014)
3. Boore, Stewart, Seyhan and Atkinson (2014)
4. Campbell and Borzognia (2014)
5. Chiou and Youngs (2014)
6. Values determined using Vs_{30} = 760 m/s for Site Class “B”

5.2.2 Probabilistic Seismic Hazard Analysis

Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475-year statistical return period) and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the online USGS Unified Hazard Tool (USGS, 2019). The results of the probabilistic analyses are presented below in Table B.

TABLE B
PROBABILISTIC SEISMIC HAZARD ANALYSES
Hillcrest Residential Subdivision
Watsonville, California

	Statistical Return Period	Magnitude	PGA
2% in 50 years	2,475 years	6.9	0.36 g
10% in 50 years	475 years	7.0	0.21 g

Reference: USGS Unified Hazard Tool, accessed 2020

The potential for strong seismic shaking at the project site is high. Due to its close proximity, the San Andreas Fault (approximately 8.1-kilometers northeast) presents the highest potential for strong ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Minimum mitigation measures should include designing the structures and foundations in accordance with the most recent version of the California Building Code. Recommended seismic coefficients are provided in Section 6.2 of this report.*

5.3 Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 35 to 50%, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity less than 7. The previous subsurface explorations did not encounter loose, granular soils below the groundwater level that would be prone to liquefaction or other liquefaction related phenomena. Additionally, the assumed historic high groundwater level is approximately 50- to 40-feet below the ground surface (elev. +11) in the project development area. Cornerstone Earth Group performed a liquefaction analysis utilizing the CPT data with the results indicating liquefiable soils do not underlie the project site.

Evaluation: *No significant impact.*
Recommendations: *No mitigation measures are anticipated.*

5.4 Seismically-Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Loose granular soils were not observed in the near-surface soils; therefore, we judge the risk of seismically-induced settlement at the site is low.

Evaluation: *Less than significant.*
Recommendations: *No mitigation measures are anticipated.*

5.5 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking generally along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. As previously discussed, an approximate 5-foot tall 2:1 slopes are proposed as part of the grading plan. As with all slopes located in seismically active areas, there is some risk of ground cracks forming along the crest of these slopes during a strong seismic event. Therefore, we judge lurching and ground cracking is a low moderate geologic hazard at the project site.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Mitigation measures include following the setback guidelines outlined in the California Building Code. Per the CBC the bottom elevation of the proposed structural foundations structures should be setback at least 7-feet from any slope face.*

5.6 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. While the building sites are located on relatively level ground, the northern and eastern slopes are prone to erosion due to excess surface runoff and concentrated flow. Therefore, the risk of damage due to erosion is generally moderate to high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Special engineering measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system. The project Civil Engineer is responsible for designing the site drainage system and, an erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook.*

5.7 Seiche and Tsunami

Seiche and tsunami are short duration earthquake-generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is located within 250-feet of the Watsonville Slough; however, the proposed residential subdivision will be constructed on top of a knoll at elevations between 50- to 70-feet above sea level, well above tsunami and seiche inundation elevations. Therefore, the risk of inundation by seiche or tsunami is low.

Evaluation: *Less than significant.*
Recommendations: *No mitigation measures are anticipated.*

5.8 Flooding

The residential subdivision is positioned at a relatively high elevation, approximately 50 to 70-feet above sea level. FEMA Flood Maps indicate the lower elevations of the property, immediately adjacent to the Watsonville Slough, are prone to flooding. However, these flood areas are more than 100-feet away from proposed improvements. Therefore, we judge widespread flooding is not a significant hazard at the project site. However, whenever new development is performed, localized changes to the existing grades may result in localized flooding.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Careful attention should be paid to site grading and drainage design to minimize the effects of potential flooding. The Project Civil Engineer should consider the potential for localized ponding of water and small-scale flooding during the maximum credible rainfall event to design site grades and drainage systems.*

5.9 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures.

Based on the subsurface exploration performed by Cornerstone Earth Group, highly plastic and expansive soils were observed on the property near the ground surface. Additionally, swell pressure tests were performed during the previous geotechnical investigation. The results of the swell testing indicate the expansive soils may exert 1,200 to 3,500 psf on the proposed improvements. Therefore, the risk of expansive soils impacting the project site is high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Foundations should be designed to withstand uplift pressure and seasonal movement from soil swelling and shrinkage. Foundations may consist of shallow foundations, rigid mat slabs, or deeper drilled pier foundations. If drilled piers are used, include void boxes to prevent uplift pressures on grade beams, and extend piers well below the zone of significant moisture fluctuation.*

Alternatively, at least 3-feet of expansive soils should be removed from the structural areas and replaced with select fill (sandy, low plasticity or lime treated clayey on-site soils) and traditional shallow foundations used. The site grading and foundation design recommendations are outlined in Sections 6.1 and 6.3, respectively.

5.10 Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites due to consolidation of soft compressible clays (i.e., Bay Mud) or compression of loose granular soils. Differential settlement may occur where structures span cut/fill transitions or other variable support conditions. Soft clayey soils were not observed during the previous subsurface exploration performed by Cornerstone Group. However, significant fills, up to 16-feet are planned as part of the residential subdivision which will exert significant stress on the underlying stiff clayey soils. These loads can cause the underlying clay layers to consolidate resulting in settlements at the ground surface. We utilized the available laboratory consolidation data performed by Cornerstone and the computer software Settle3D produced by Rocscience to predict the amount of settlement that may occur over time. A graph indicating the predicted settlement based on fill height is presented on Figure 6.

The predicted settlements indicated on Figure 6 be considered approximate to a degree of accuracy of 25%. The laboratory testing performed by Cornerstone did not include a coefficient of consolidation to determine the time rate of settlement and therefore we cannot provide a time rate estimate; however, we anticipate the predicted settlements will take some time to occur.

In addition to the fill placement causing the underlying soils to consolidate under the large fill loads, the fills over 5-feet in height will consolidate, resulting in surface settlements. A general range of fill settlement is approximately 0.5% to 1% of the fill height. As an example, a 15-foot-tall fill should settle 0.08- to 0.15-feet or 1.0- to 1.8-inches. This settlement typically occurs within 5- to 10-years after the fill has been placed.

Based on our settlement analysis and the anticipated fill heights we judge the risk of site settlement to the residential structures to be low. Hardscaped site improvements overlying the deeper fills, i.e. asphalt streets and parking areas, will experience some additional cracking and additional maintenance should be anticipated.

Evaluation:	*Less than significant with mitigation.*
Recommendations:	*Fills should be prepared and compacted as outlined in the Site Grading section of this report. Additional maintenance may also be required to repair cracks that may appear in the proposed overlying hardscape.*

5.11 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The project site has experienced previous landslides. As previously discussed, an approximate 10- to 15-foot tall 2:1 slope is located along the northern and eastern ends of the proposed subdivision. We understand this slope will be reduced in height to no greater than 5-feet in height with the addition of level pedestrian areas, rain gardens and tiered site retaining walls. Additionally, a new retaining wall up to 16-feet in height is proposed to bury on-site contaminated soils and create additional level space for recreational and parking space. The weight of this new fill may reduce the stability of the lower areas. Slope stability analysis was performed by Cornerstone Earth Group identified placing additional fill would reduce the overall slope stability.

We performed an updated slope stability analysis on various cross sections generated from the current grading plan and utilizing the stability software SLIDE developed by Rocscience. The “Spencer” slope stability analysis method was utilized to analyze the cross sections. Two of the five sections analyzed, the MSE Wall Section and Section B, indicate placing significant fill on the existing grades. The remaining sections either involved cutting soil away from the slopes or adding minor amounts of fill. Therefore, we performed slope stability analyses on the MSE Sections and Section B. Section B also includes a 2:1 (horizontal:vertical) above the five foot terraced retaining walls. Therefore, 2 stability analyses were performed on this section, a global analysis and an analysis focused on the 2:1 slope.

For static slope stability analyses, a factor of safety against soil movement above 1.5 is considered appropriate. Under seismic conditions, a factor of safety less than 1.0 indicates some movement may be observed during a strong seismic event. We utilized the procedures outlined by Bray and Travasarou, 2007 to determine the amount of deformation during a strong seismic event. Two deformation result values are determined in the analysis. The smaller value refers to a higher probability of occurrence during a strong seismic event while the larger of the numbers has a lower probability of occurrence. Additionally, the predicted deformations are more likely to occur in smaller amounts throughout the mass that add up to the predicted values rather than the predicted deformation occurring in one location within the mass. The results of our slope stability and slope deformation analyses are presented in Appendix B and summarized below on Table C.

TABLE C
Slope Stability Results
Hillcrest Residential Subdivision
Watsonville, California

	Static F.S.	Seismic F.S.	Seismic Deformation[1]
Section B (Global)	2.00	0.75	~4 – 8-in
Section B (2:1)	3.37	1.30	0.0-in (F.S. > 1.0)
Section MSE Wall	1.75	0.66	~6 – 12-in

Notes:
1. Predicted deformations are distributed throughout the landslide mass.

Evaluation:	*Less than significant with mitigation.*
Recommendations:	*Structures should be setback from the slope crests as outlined in the California Building Code. Structures may be constructed within the setback zone provided they are supported on a deep foundation system as described in the Foundation section of this report.*

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of reference material and the previous subsurface exploration and laboratory testing, we conclude that the proposed subdivision is feasible from a geotechnical perspective. The primary geotechnical issues to address in design of the project are providing adequate seismic design, expansive soils, protecting the residential subdivision from potential slope instability and providing uniform foundation support. Specific recommendations and criteria to address these and other geotechnical project facets are presented in the following sections.

6.1 Site Preparation and Grading

Preliminary plans indicate moderate to significant site grading will be performed to develop the project site. Site grading is expected to include creating building pads, creating new streets to allow site access, and constructing new pedestrian paths. The grading recommendations presented below are appropriate for construction in the late spring through fall months. From winter through the early spring months, on-site soils may be saturated due to rainfall and may be difficult to compact without drying by aeration or the addition of lime and/or cement (or a similar product) to dry the soils. Site preparation and grading should conform to the recommendations and criteria outlined below. General recommendations for wintertime construction are provided later in this report.

6.1.1 Surface Preparation

Clear all trees, brush, roots, over-sized debris, and organic material from areas to be graded. Trees and large shrubs that will be removed (in structural areas) must also include removal of stumps, root balls and roots larger than two inches in diameter. Excavated areas (i.e., old fills and stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. Debris, rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas. Surface preparation should extend at least 5-feet beyond proposed structures and 3-feet beyond pavement areas.

6.1.2 Materials

Based on previous subsurface explorations, onsite granular soils that exhibit low to medium plasticity and may be suitable for use as fill, provided they meet the criteria for onsite and imported fill material. As previously discussed, highly plastic and expansive soils were also observed on the project site. These plastic and expansive soils are not suitable for fill in structural areas, unless they have been lime/cement treated.

Onsite and import soils shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 20, (3) have a maximum particle size of 6 inches, and (4) have more than 50% retained on the No. 200 sieve. Any imported fill material shall be tested and inspected by the project geotechnical engineer to determine its suitability for use as fill material.

6.1.3 Lime Treatment

As previously discussed, expansive soils were encountered on the project site. Lime treatment chemically alters the clay soils resulting in a reduction in their inherent plasticity, a significant reduction in their shrink/swell potential, an improvement to its workability (i.e., compaction), and an increase of its shear strength. If soil treatment is utilized during site grading, in structural areas we recommend at least 5% high calcium lime should be thoroughly mixed to the surficial soils (utilizing a 115 pcf soil density) resulting in a soil pH of at least 12.4 to promote the chemical reaction, to be confirmed with laboratory testing. The depth of treatment in building areas should extend at least 36-inches below the ground surface. The depth of treatment may be reduced to 18-inches in areas where flatwork is proposed. Soil treatment should extend at least 5-feet beyond the area of work where possible. Treated soils should then be compacted to at least 90% relative compaction in structural areas and 95% relative compaction in areas subject to vehicular loads.

6.1.4 Compacted Fill

On-site fill, backfill, and scarified subgrades (8-inches deep) should be conditioned to within 3% of the optimum moisture content. Properly moisture conditioned and cured on-site materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to a minimum of 90% R.C. Expansive soils should be further moisture conditioned to at least 3% over the optimum moisture content and compacted to between 88 and 92% R.C. To reduce the settlement potential, the compaction of fills taller than 5-feet should be increased to 95% R.C.

Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop". Relative compaction should increase to 95% in the upper foot where asphalt pavement is planned.

6.1.5 Slopes

Based on our slope stability analyses and the relatively short fill slopes proposed, roughly 5-feet, we judge a maximum cut and fill slope inclination of 2:1 is appropriate. Although not currently planned, intermediate terraces and surface drainage should be constructed on fill slopes greater than 20-feet in height.

6.1.6 Excavations

The subsurface conditions generally consist of medium stiff to stiff, highly plastic clay. As previously discussed, the site was previously used as an auto salvage yard for over 50-years. Test pits performed by others encountered significant automobile parts including tires, sheet metal and other debris. Although we anticipate the soil will be easily excavated with standard equipment (i.e., excavators, dozers, scrapers, etc.), the contractor should anticipate encountering some large debris that may impact the excavation conditions.

Soils in excavations appear to be Cal-OSHA “Type C” and excavations having a depth of five feet or more, and will be entered by workers must be sloped, braced, or shored in accordance with current Cal-OSHA regulations. All excavations can result in collapse of sidewalls, slopes and/or bottom that could result in injury or death of workers. Therefore, excavations should be evaluated by the Contractor’s safety officer and designated competent person prior to workers entering in accordance with current Cal-OSHA regulations.

6.2 Seismic Design

The project site is located in a seismically active area. Therefore, structures should be designed in conformance to the seismic provisions of the most recent (2019) California Building Code (CBC). However, since the goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking. Based on our review of the subsurface exploration performed by others it is our opinion the site may be classified as a “Stiff Soil Site Class D” site.

Per ASCE 7-16 Section. 11.4.8, a Site-Specific Ground Motion Hazard Analysis shall be performed in accordance with ASCE 7-16 Section 21.2 on sites classified as a "Site Class D" if the S_1 value is greater than or equal to 0.2 g. The S_1 value for the site conditions and location is 0.94 g; therefore, we performed a Site-Specific Ground Motion Hazard Analysis as presented in Appendix C and the results are presented below on Table D.

TABLE D
ASCE 7-16 SEISMIC PARAMETERS
Hillcrest Residential Subdivision
Watsonville, California

Factor Name	Coefficient	ASCE 7-16 Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	S_D
Spectral Response (short)	SM_S	1.98 g
Spectral Response (1-sec)	SM_1	1.89 g
Design Spectral Response (short)	SD_S	1.32 g
Design Spectral Response (1-sec)	SD_1	1.26 g
MCE_G[2] PGA adjusted for Site Class	PGA_M	1.24 g

Notes:
1. Site Class D Description: Stiff soil profile with shear wave velocities between 600 and 1,200 ft/sec, standard blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Maximum Considered Earthquake Geometric Mean.

6.3 Shallow Foundation Design

Based on the subsurface soil conditions, it is our opinion the planned residences may be supported on a shallow foundation system. However, due to the presence of surficial expansive soils, shallow foundations should be designed to withstand seasonal movement. For structures with raised floors, shallow foundation excavations should be designed to be at least 36-inches deep to extend at least 3-feet below the ground surface. The foundation system should be designed as a rigid system to span over 10 feet of non-uniform support within the structure and to cantilever 5 feet at the edges. The over-excavation may then be backfilled with non-expansive soil, as described above, or with cement slurry/control density fill (CDF).

Alternatively, shallow foundations may be constructed on at least 36 inches of select fill or lime treated soils as described above without the need for over-excavated or deepened foundations. Shallow foundations located adjacent to slopes should be deepened as necessary to allow at least 7-feet of horizontal confinement between the bottom of the footing and slope face.

Structures may also be supported on a rigid concrete slab-on-grade. The concrete slabs-on-grade should be designed to withstand seasonal movement due to expansive soils. Post tensioned slabs may be required to provide adequate strength. Shallow foundation and mat slab-on-grade design criteria are presented below on Table E.

TABLE E
FOUNDATION DESIGN CRITERIA
Hillcrest Residential Subdivision
Watsonville, California

Shallow Spread Footings

Minimum footing width[1]:		
One-story structure		12 inches
Two-story structure:		15 inches
Minimum footing depth:	Untreated	Select Fill
	36 inched	16 inches
Allowable weathered bedrock bearing pressure (dead plus live loads)[2]:		
Native soils:		1,500 psf
Lime treated soils:		3,500 psf
Base friction coefficient:		0.30
Lateral passive resistance[2, 3, 4]:		300 pcf

Mat Slab-on-Grade Criteria

Minimum thickness:	6-inches
Modulus of subgrade reaction:	100 pci
Edge moisture variation $e_{m,\ Edge}$	5.0 feet
Edge moisture variation $e_{m,\ Center}$	10.0 feet
Differential soil movement $y_{m,\ Edge}$	0.5 inches
Differential soil movement $y_{m,\ Center}$	1.0 inches

Notes:

1. Size footing widths to avoid significantly different foundation pressures.
2. May increase design values by 1/3 for total design loads including seismic.
3. Equivalent Fluid Pressure, not to exceed 3,000 psf.
4. Ignore uppermost 6-inches unless concrete or asphalt surfacing exists adjacent to foundation.

6.4 Deep Foundation Design

A drilled pier foundation system extending through the surficial expansive soils and embeds into the underlying stiff soils may also be utilized to support the proposed residential structures. Deep foundations should be spaced more than three pier/pile diameters apart from each other and interconnected with gradebeams. Gradebeams should be constructed on 4-inch-thick cardboard void boxes to prevent uplift pressure underlying expansive soils. Alternatively, gradebeams and drilled piers may be designed to withstand at least 3,500 psf of uplift pressure. Additionally, “mushrooming” of the top of the drilled piers should be prevented to reduce additional uplift pressure. Sonotubes should be utilized in the upper 3-feet if “mushrooming” of the pier tops occurs. Drilled piers may be designed utilizing the parameters outlined on Table F below.

TABLE F
DRILLED PIER FOUNDATION DESIGN CRITERIA
Hillcrest Residential Subdivision
Watsonville, California

Minimum diameter:	16-inches
Skin friction[1]:	
0 to 3-feet	Neglect
3 to 10-feet	300 psf
10 to 20-feet	750 psf
20 to 30-feet	1,000 psf
Lateral passive resistance[2,3,4]:	
0 to 3-feet	Neglect
3 to 10-feet	300 pcf
10 to 20-feet	400 psf
20 to 30-feet	500 psf

Notes:

1.) Uplift capacity is equal to 80% of the downward skin resistance.

2.) Ignore upper 6-inches unless concrete or asphalt surfacing exists adjacent to foundation.

3.) Apply passive resistance over two pier diameters.

4.) Lateral pile reduction factors, "P-multipliers" should be included in design when foundations are within groups. P-multipliers are dependent on pier/pile spacing (s) and diameter (d). The following equations should be utilized to calculate the p-multiplier:
   a. First (Lead) Row Piles: $P_m = 0.26*\ln(s/d) + 0.50 \leq 1.0$
   b. Second Row Piles: $P_m = 0.52*\ln(s/d) \leq 1.0$
   c. Third Row or Higher Piles: $P_m = 0.60*\ln(s/d) - 0.25 \leq 1.0$

Alternate deep foundation options are available including helical piles. Helical piles are slender (4-inches or less in diameter) steel pipes or shafts that have two or more steel circular plates welded near the tip. The piles are screwed into the ground and extend to a design depth and capacity that is determined in the field during installation. Based on the subsurface conditions we anticipate helical piles should be able to obtain 15 to 20-kips at depths around of 15-feet below the ground surface. Helical piles are typically interconnected with gradebeams and spaced 5 to 10-feet on center. Helical piles provide negligible lateral passive resistance due to the slender nature of the steel rods. Therefore, lateral passive resistance may be obtained from the grade beams. If helical piles are determined to be an economic option, we should be contacted to assist with the design of the foundation system.

6.5 Retaining Wall Design

We anticipate retaining walls up to 16-feet in height will be required to retain the cuts and fills needed to create level pedestrian paths, soil remediation areas, and rain gardens to the north and east of the proposed residences. The 16-foot-tall retaining wall will be located on the northern end of the property and will support the soil remediation soil, while 5-foot tall, tiered retaining walls will be constructed on the northern and eastern ends of the property. The tiered walls will be separated by a level pedestrian path with the upper wall supporting a roughly 5-foot tall 2:1 slope. We anticipate these walls will be free to rotate at the top, therefore may be designed with “unrestrained” soil lateral earth pressures.

The current plan is to construct the level pedestrian pathway by filling to raise grades. While typical reinforced concrete or concrete masonry unit (CMU) retaining walls may be utilized to retain the fill, it is our opinion mechanically stabilized earth (MSE) retaining walls with a stacked block face will be more cost effective. MSE walls are constructed by placing layers of compacted fill with interbedded geogrids every 18- to 24-inches. The geogrids are connected to concrete blocks located at the wall face. These walls do not require concrete or steel reinforcement and are built in conjunction with fill placement. Retaining wall design criteria is shown on Table G below.

TABLE G
RETAINING WALL DESIGN CRITERIA
Hillcrest Residential Subdivision
Watsonville, California

Foundations

See Table D or E

Unrestrained Earth Pressure[1,2]

Level Ground:	40 pcf
2:1 Slope:	60 pcf

MSE Wall Design

	Unit Weight, γ	Cohesion, c	Friction, ϕ
Reinforced Soil:	120 pcf	N/A	30º
Retained Soil:	110 pcf	N/A	30º
Foundation Soil:	110 pcf	500 psf	30º

Seismic Surcharge[3] 10 x H psf

Notes:

1. Interpolate earth pressures for intermediate slopes.
2. Equivalent fluid pressure.
3. Rectangular distribution. The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater. “H” = wall height.

Drainage shall be provided for all retaining walls taller than 3-feet consisting of either ¾-inch crushed rock, wrapped within filter fabric, or Caltrans Class 2 permeable material. The seepage should be collected in a 4-inch perforated PVC drain line at the base of the wall. The permeable material shall extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. Drainage panels, such as Mirifi 100N, may be utilized. If drainage panels are utilized, the perforated pipe locate at the base of the retaining wall should be surrounded in ¾-inch drain rock and wrapped in filter fabric. A schematic retaining wall drainage detail is presented on Figure 7.

Seepage collected in the drain line should be conveyed off-site by gravity in closed pipe to the storm drainage system. The pipe shall have a minimum slope of 1 percent to drain. To maintain the wall drainage system, clean outs shall be installed at the upstream end and at all major changes in direction. Water proofing of any below grade residential walls should be designed by the Architect to prevent moisture infiltration through the wall into living spaces.

6.6 Site Drainage Considerations

Careful consideration should be given to design of new finished grades at the site to ensure positive drainage. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at 5 percent for a distance of at least 5-feet from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10-feet in the first 5-feet (2 percent). Roof gutter downspouts may discharge onto the pavements but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a tight pipe collection system. The tight pipe system should discharge at an appropriate location unlikely to result in adverse erosion, preferably into an established municipal storm drain system. If it is not possible to discharge into the City’s storm drain system, collected water should be discharged near the base of the slope and spread laterally via dissipators.

6.7 Underground Utilities

Based on previous subsurface explorations performed by others, onsite soils are “Type C” per Cal-OSHA guidelines and will be prone to caving and raveling in open excavations. The Contractor is responsible for site safety and should provide adequate shoring as needed.

Bedding materials for utility pipes should be non-corrosive sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 15 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer’s recommendation, typically 3 to 6-inches. Utility excavations should be backfilled with select fill per criteria discussed previously and compacted to a minimum of 90 percent relative compaction. In pavement areas, relative compaction should be increased to a minimum of 95 percent in the upper 12-inches.

6.8 Asphalt Concrete Pavements

We have calculated preliminary pavement sections in accordance with Caltrans procedures for flexible pavement design using an assumed R-value of 5. The R-value of the subgrade soils may be increased to 40 provided they are lime treated. We have provided a range of Traffic Indices (TI) from 4 to 7 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles (such as fire lanes, loading dock access roads, trash enclosures, etc.) should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. Preliminary recommended pavement sections are shown in Table H; these should be verified on the basis of supplemental laboratory testing.

TABLE H
PAVEMENT DESIGN CRITERIA
Hillcrest Residential Subdivision
Watsonville, California

			Untreated Subgrade	**Lime Treated Subgrade**
	T.I.	Asphalt Concrete	Aggregate Baserock	Aggregate Baserock
Driveways & parking stalls	4.0	2.5-inches	8.0-inches	6.0-inches
Light truck traffic	5.0	3.0-inches	10.0-inches	6.0-inches
Moderate truck traffic	6.0	3.5-inches	13.0-inches	6.0-inches
Heavy truck traffic	7.0	4.0-inches	16.0-inches	8.0-inches

Subgrade preparation for asphalt-paved areas should be performed in accordance with the grading recommendations of this report. The base rock should consist of compacted Class 2 Aggregate Base (Caltrans, 2018), be conditioned to near optimum moisture content, placed in lifts no more than six inches thick, and compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment. The subgrade should also be maintained at near-optimum moisture content prior to placement of aggregate base rock. Areas of soft or saturated soils encountered during construction should be excavated and replaced with properly moisture conditioned fill or aggregate base.

7.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, we must observe and test site grading, foundation excavations for the structures and associated improvements to confirm that the soils encountered during construction are consistent with the design criteria.

8.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in the greater Santa Cruz area at the time the report was prepared. This report has been prepared for the exclusive use of California Sunshine Subdivision, LLC and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data performed by others and reviewed by us and our experience with soils in this geographic area.

Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless MPEG is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of MPEG.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

9.0 LIST OF REFERENCES

Abrahamson, Silva and Kamai, “Summary of the ASK14 Ground Motions Relation for Active Crustal Regions,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Aagard, B.T. et al (2016), “Earthquake Outlook for the San Francisco Bay Region 2014-2043”, United States Geological Survey Fact Sheet 2016-3020, Version 1.1, Revised August 2016.

American Society of Civil Engineers, “Minimum Design Loads for Buildings and Other Structures,” (ASCE 7), 2010.

American Society for Testing and Materials, “2016 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics,” ASTM, Philadelphia, 2019.

Association of Bay Area Governments (ABAG), Geographic Information System, http://quake.abag.ca.gov/mitigation/, 2017.

Boore, Stewart, Seyhan and Atkinson, “NGA-West2 Equations for Predicting PGA, PGV and 5% Damped PSA for Shallow Crustal Earthquakes,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Bray, J.D. & Travasarou, T., “Simplified Procedures for Estimating Earthquake Induced Deviatoric Slope Displacements,” Journal of Geotechnical Engineering and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007.

California Building Code, 2019 Edition, California Building Standards Commission, Sacramento, California.

California Geologic Survey, CGS Regulatory Maps Geo Application, “Earthquake Zones of Required Investigation” https://maps.conservation.ca.gov/cgs/EQZApp/app/, 2020.

California Department of Conservation, Division of Mines and Geology (2000), “Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region”, DMG CD 2000-004.

California Department of Transportation (Caltrans), “Acceleration Response Spectra (ARS) Online version 2.3.06” http://dap3.dot.ca.gov/ARS_Online/index.php, accessed August 16, 2017.

California Department of Transportation (Caltrans), “Standard Specifications,” 2015.

Campbell and Borzognia, “NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, GV and 5% Damped Linea Acceleration Response Spectra,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Chiou and Youngs, “Update of the Chiou and Youngs NGA Model for the Averafe Horizontal Component of Peak Ground Motion and Response Spectra,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

County of Los Angeles, “Review of Geotechnical Reports Addressing Liquefaction,” February 24, 2009.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3) – The time-independent model: U.S. Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

Herzog Geotechnical (2005), “Geotechnical Investigation, Trader Joe’s, 7514-7518 Redwood Boulevard, Novato, California”, Project No. 1158-01-04, dated July 19, 2005.

Idriss, I.M. & Boulanger, R.W. “Soil Liquefaction during Earthquakes”, Earthquake Engineering Research Institute Monograph 12, 2008

Idriss, I.M. & Boulanger, R.W. “SPT-Based Liquefaction Triggering Procedures” Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, UCD/GCM-10/02, December 2010.

Miller Pacific Engineering Group, “Geotechnical Peer Review Sunshine Vista Residential Development Watsonville, California,” June 10, 2020.

Ozocak, A & Sert, S., “Evaluation of Liquefaction Risk by a revised LPI approach,” 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA. Volume 2&3: Technical Papers, Session 3: Applications, Paper No. 3-30, 2010.

Occupational Safety and Health Administration (OSHA)(2005), Title 29 Code of Federal Regulations, Part 1926, 2005.

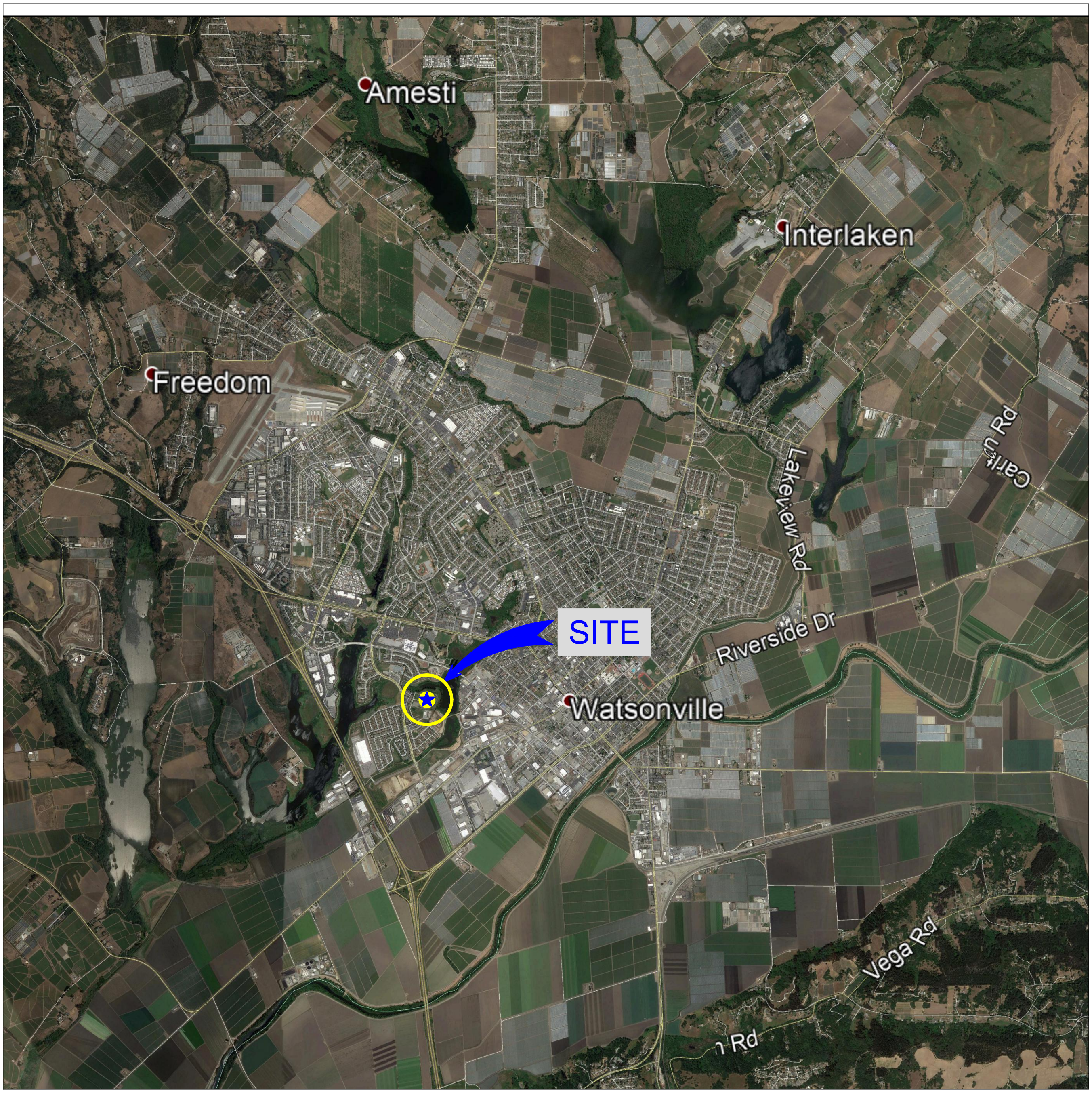
Structural Engineers Association of California (SEAOC), “Seismic Design Map Tool,” https://seismicmaps.org/, accessed November 2020.

United States Environmental Protection Agency (USEPA)(2017), EPA Map of Radon Zones in California, http://www.epa.gov/radon/states/california.html, accessed August 15, 2017.

United States Geologic Survey, “Geologic Map of Santa Cruz County,” Brabb, E., Graham, S., Wentworth, C., Knifong, D., Graymer, R. & Bussenbach, J., Open File Report 97-489, 1997.

United States Geological Survey (2003), “Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032,” The 2003 Working Group on California Earthquake Probabilities, 2003.

United States Geological Survey (2008), “The Uniform California Earthquake Rupture Forecast, Version 2,” The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.



SITE: LATITUDE, 36.9105°
LONGITUDE, -121.7719°

SITE LOCATION

N.T.S.

REFERENCE: Google Earth, 2020

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FILENAME: 1680.023 Figures_recover.dwg

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SITE LOCATION MAP

Hillcrest Residential Subdivision
Watsonville, California

Project No. 1680.023

Date: 1/09/2021

Drawn BSP

Checked

1

FIGURE

REFERENCE: Ramsey Civil Engineering Inc. "Hillcrest Subdivision, Rough Grading and Site Drainage Plan," December 14, 2020

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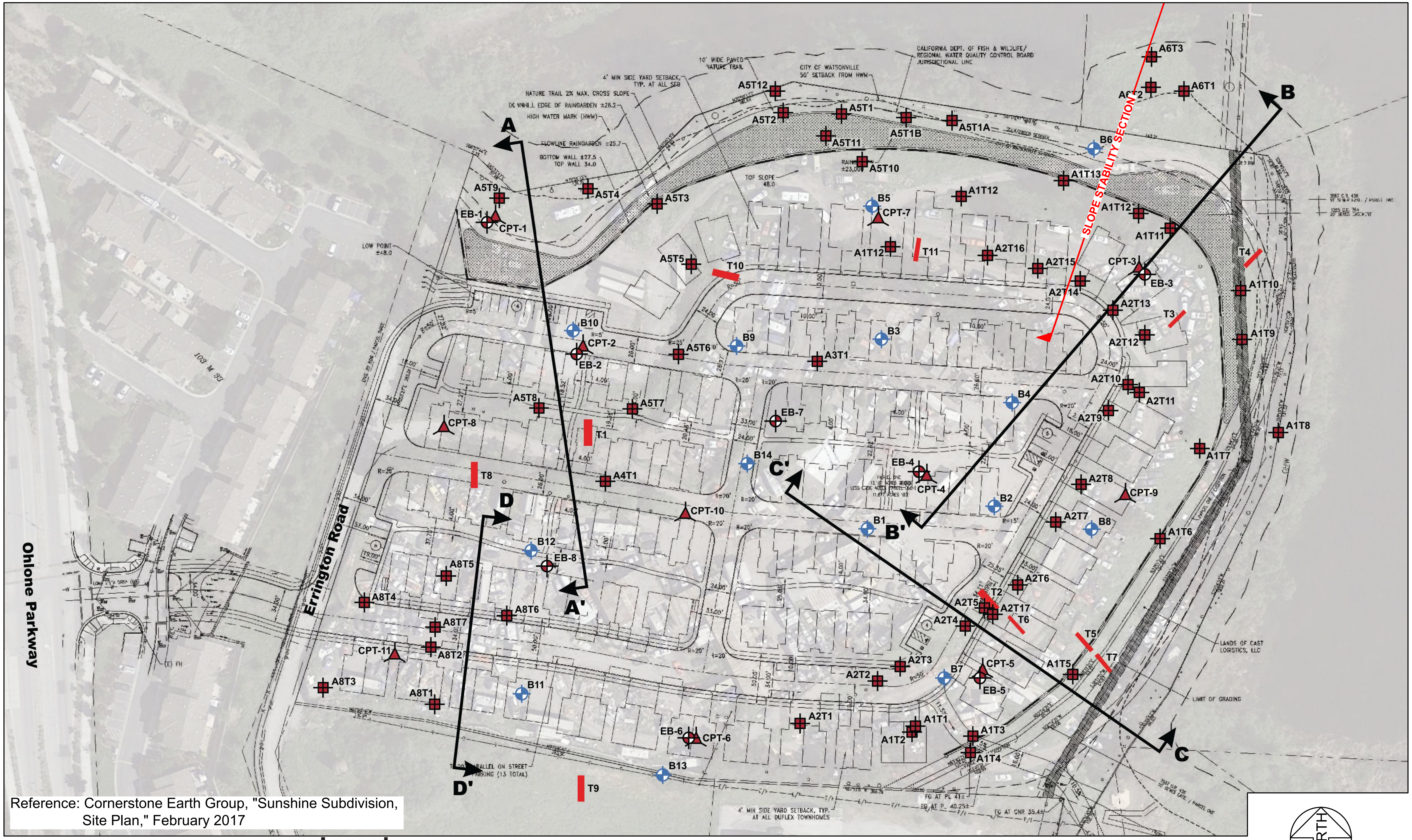
SITE PLAN

Hillcrest Residential Subdivsion
Watsonville, California

Project No. 1680.023 Date: 6/04/2021

Drawn BSP
Checked

2
FIGURE



Reference: Cornerstone Earth Group, "Sunshine Subdivision, Site Plan," February 2017

Legend

⊕ Approximate location of exploratory boring (B) (Butano, 2016)

▮ Approximate location of exploratory trench (T) (Butano, 2016)

⊕ Approximate location of exploratory boring (EB)

▲ Approximate location of cone penetration test (CPT)

⊞ Approximate location of test pit (Cornerstone, October 2016)

SCALE

0 50 100 200 FEET

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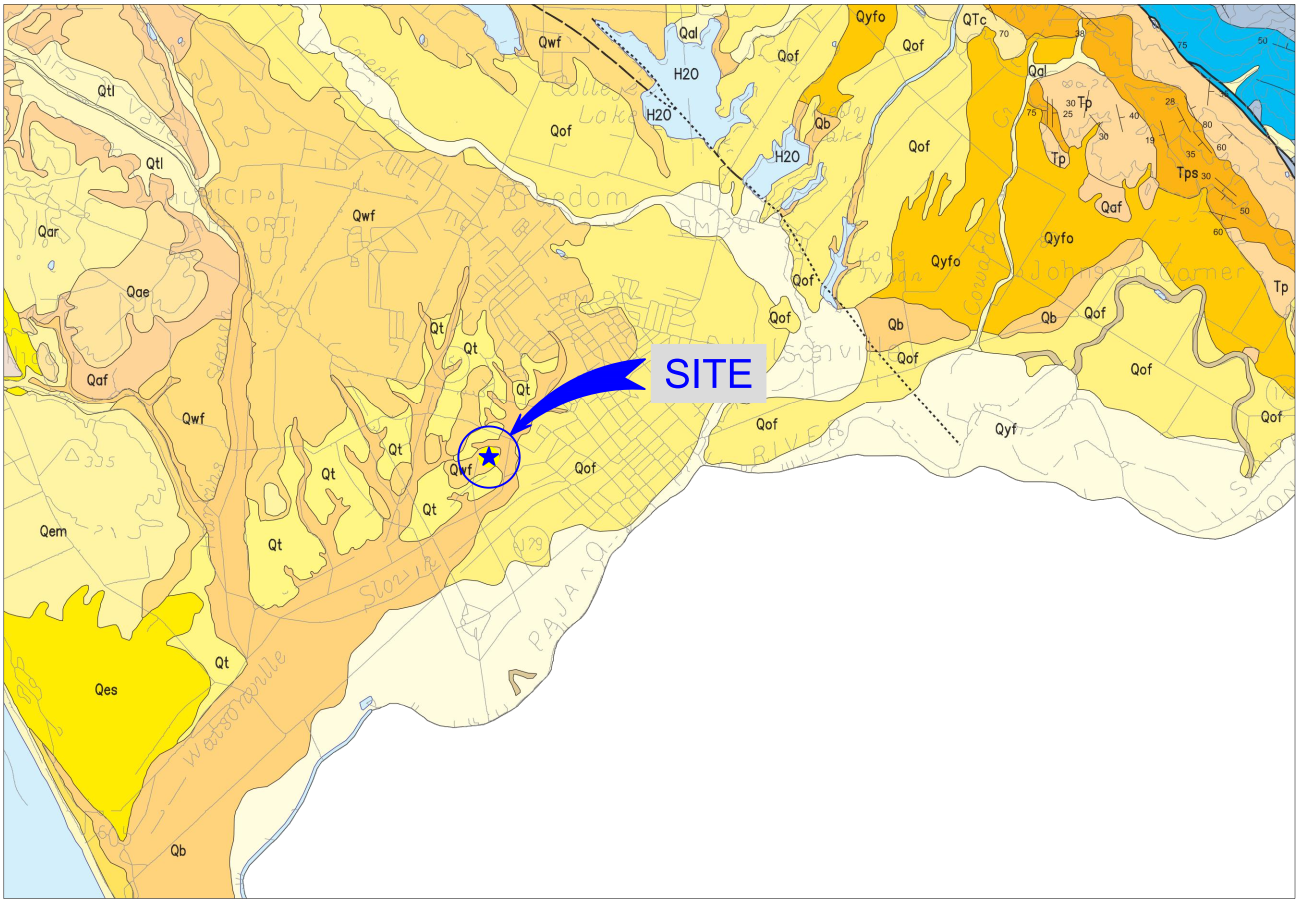
SITE PLAN

Hillcrest Residential Subdivision
Watsonville, California

Project No. 1680.023 Date: 1/09/21

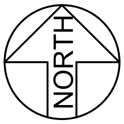
Designed
Drawn BSP
Checked

2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)



LEGEND

Qb	**Basin Deposits (Holocene)**
Qof	**Alluvial Fan (Pleistocene)**
Qt	**Terrace Deposits (Pleistocene)**
Qwf	**Fluvial Deposits (Pleistocene)**

Reference: United States Geologic Survey, "Geologic Map of Santa Cruz County," Brabb, E., Graham, S., Wentworth, C., Knifong, D., Graymer, R. & Bussenbach, J., Open File Report 97-489, 1997

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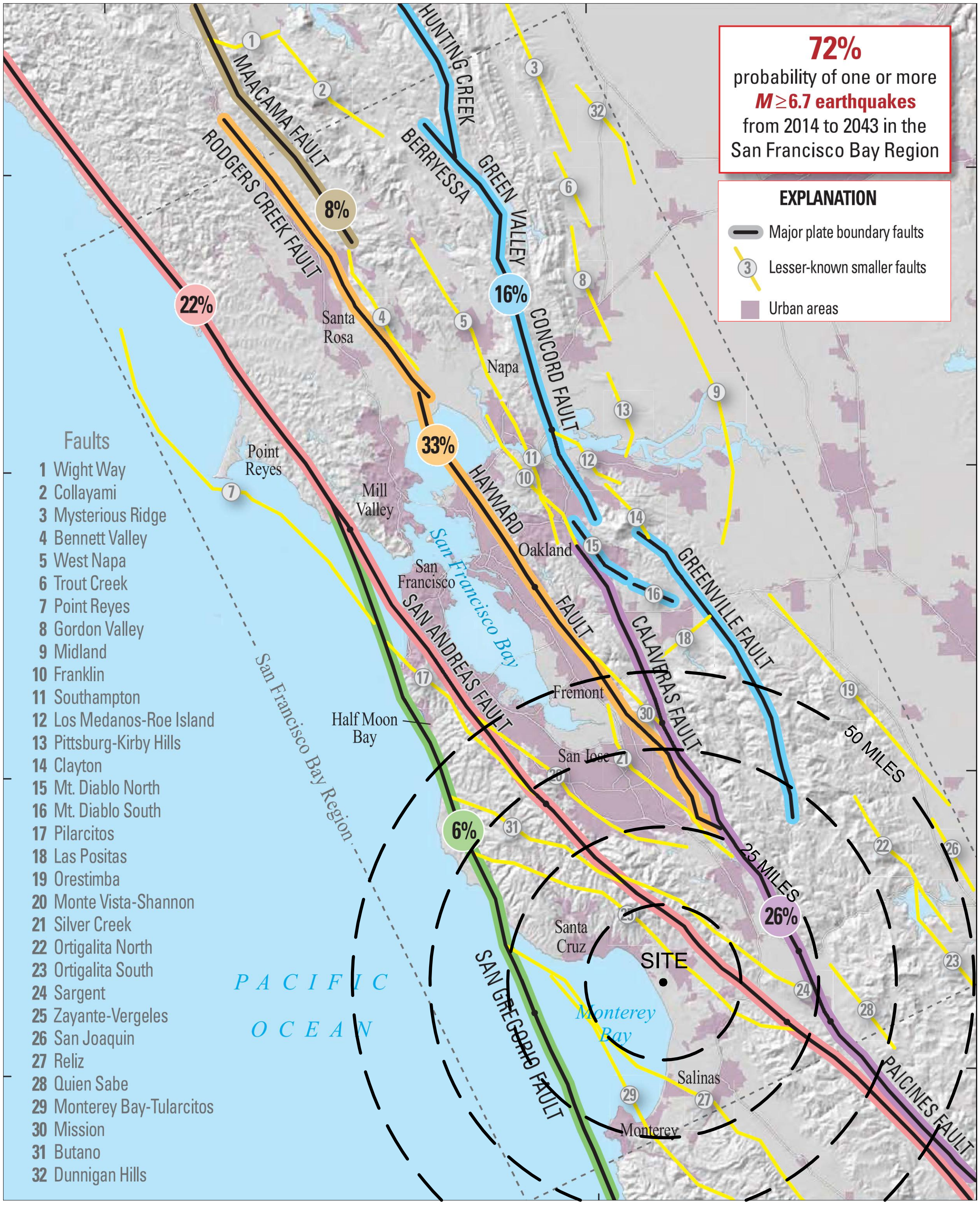
REGIONAL GEOLOGIC MAP

Hillcrest Residential Development
Watsonville, California

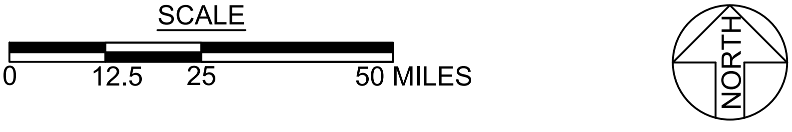
Project No. 1680.023
Date: 1/09/2021

Drawn BSP
Checked

3
FIGURE



SITE COORDINATES
LAT. 36.9105°
LON. -121.7719°



DATA SOURCE:

1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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FILENAME: 1680.023 Figures_recover.dwg

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ACTIVE FAULT MAP

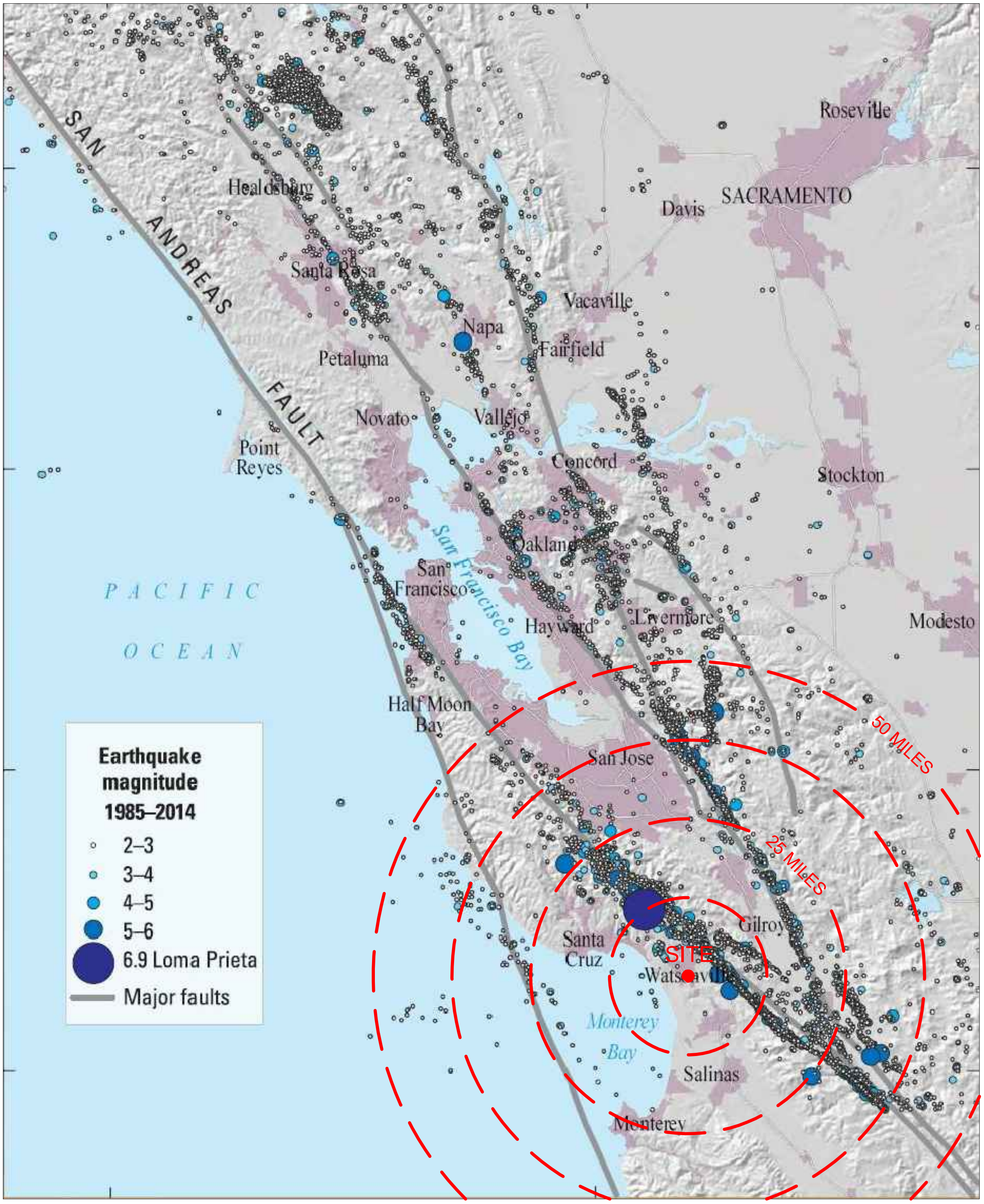
Hillcrest Residential Subdivision
Watsonville, California

Drawn BSP
Checked

Project No. 1680.023

Date: 1/09/2021

4
FIGURE



SITE COORDINATES
LAT. 38.1514°
LON. -122.2424°

SCALE

0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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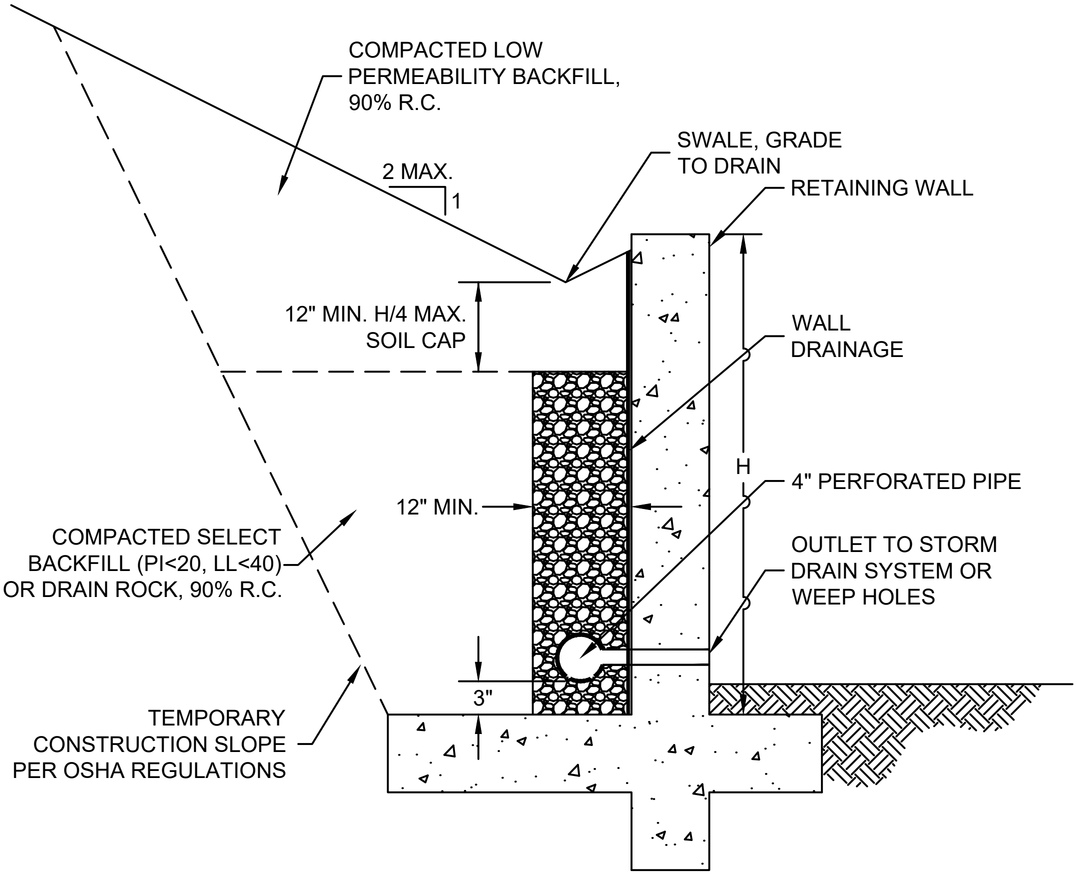
HISTORIC EARTHQUAKE ACTIVITY

Hillcrest Residential Subdivision
Watsonville, California

Project No. 1680.023 Date: 1/09/2021

Drawn BSP
Checked

5
FIGURE



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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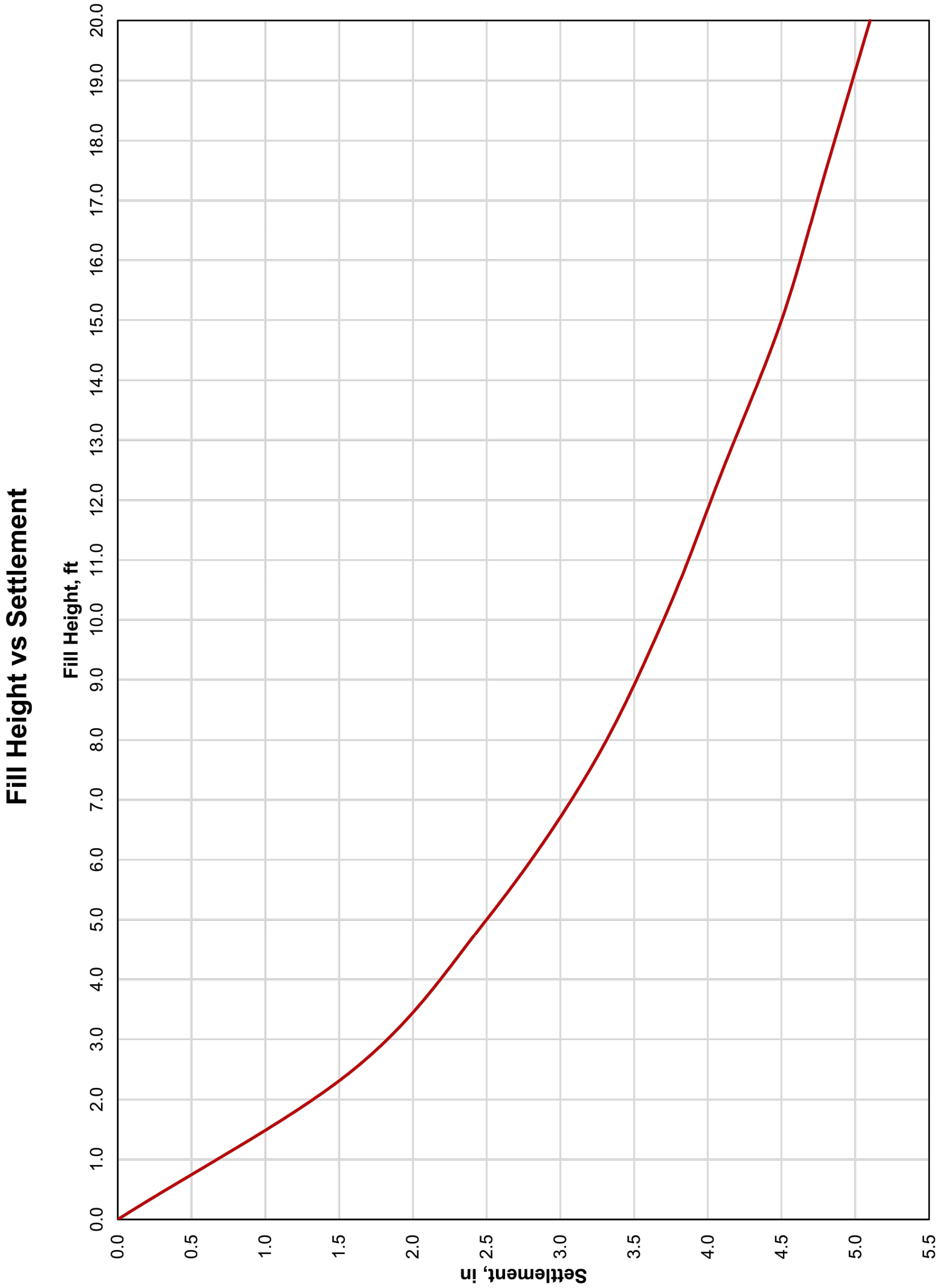
RETAINING WALL BACKDRAIN

Hillcrest Residential Subdivision
Watsonville, California

Project No. 1680.023
Date: 1/09/2021

Drawn BSP
Checked

7
FIGURE



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FILE: 1680.023 Figures(rev).dwg

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FILL HEIGHT vs SETTLEMENT

Hillcrest Residential Subdivision
Watsonville, California

Project No. 1680.023 Date: 6/04/2021

Drawn BSP
Checked

6
FIGURE

APPENDIX A:
PREVIOUS SUBSURFACE EXPLORATION

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Nine 8-inch-diameter exploratory borings were drilled on November 22, 2016, December 21, 2016, and December 28, 2016, to depths of 20½ to 54½ feet. Eleven CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on November 12 and 13, 2016, to depths ranging from approximately 21 to 52 feet. In addition to the borings and CPT explorations, we observed 58 test pit excavations performed by Trinity Source Group, Inc. on October 11 to 13, 2016. The depth of test pits ranged up to about 13 feet deep. The approximate locations of exploratory borings, CPTs, and test pits are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

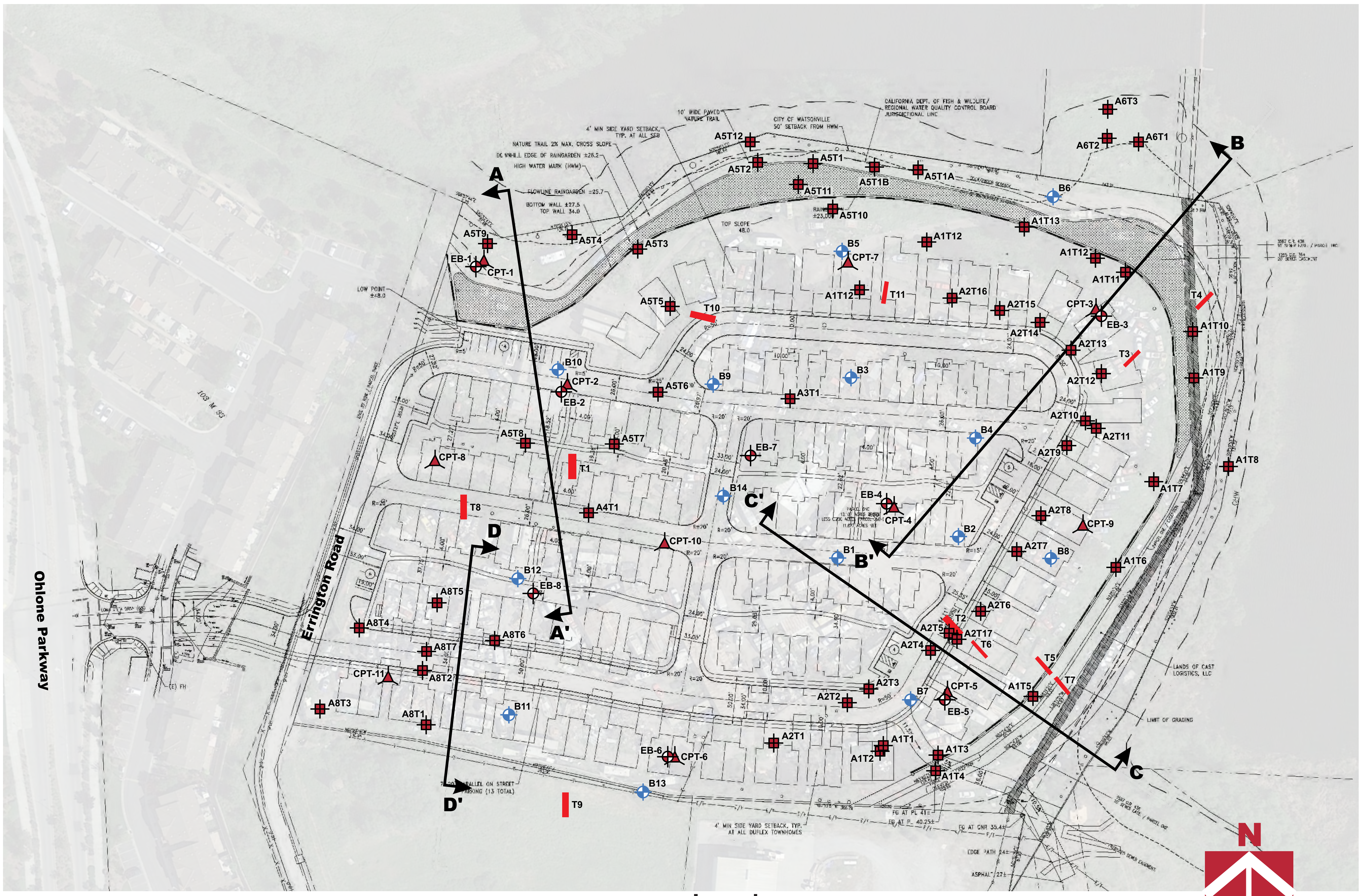
Boring, CPT, and test pit locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and CPT elevations were based on interpolation of the topographic map provided from Ifland engineers. The locations and elevations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.



Legend

- ⊕ Approximate location of exploratory boring (B) (Butano, 2016)
- ▮ Approximate location of exploratory trench (T) (Butano, 2016)
- ⊕ Approximate location of exploratory boring (EB)
- ▲ Approximate location of cone penetration test (CPT)
- ⊞ Approximate location of test pit (Cornerstone, October 2016)

N

0 100 200
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2016
Overlay: SECTIONS FOR EXPORT (1)-Site Plan (No Topo) (Mono).PDF

CORNERSTONE
EARTH GROUP

Site Plan

Sunshine Vista
Residential Development
Watsonville, CA

Project Number
928-1-2

Figure Number
Figure 2

Date
February 2017

Drawn By
RRN

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Graded Gravelly Sand
- Gravelly Silt
- Asphalt
- Boulders and Cobble

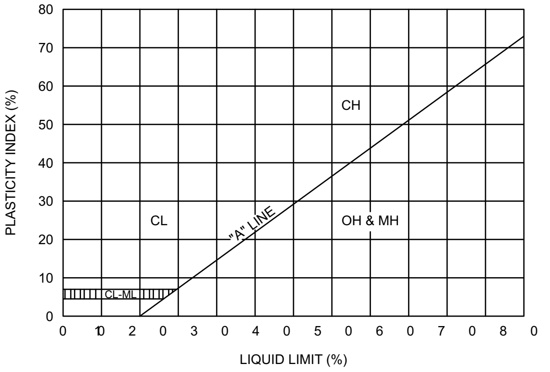
SAMPLER TYPES

☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core	✋	Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW		SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)		-	
RV	-	R-VALUE	UU		UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE		-	
▼	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 2

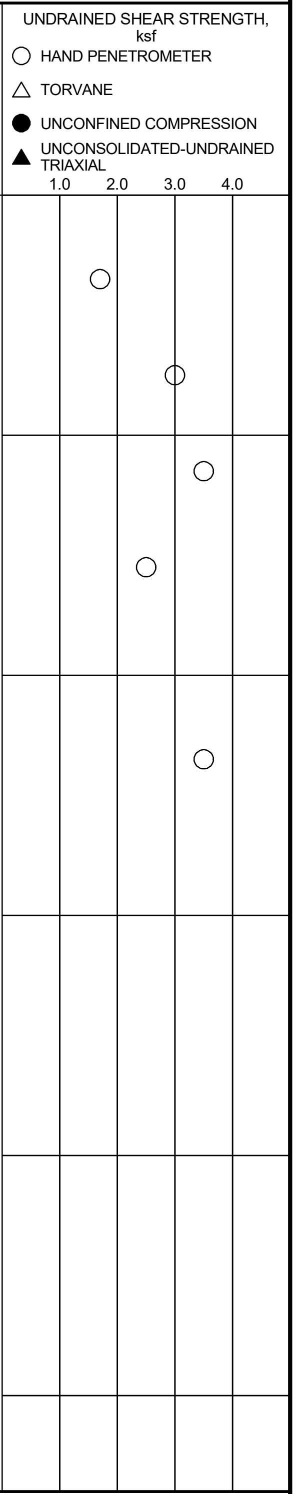
CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 12/21/16 **DATE COMPLETED** 12/21/16
GROUND ELEVATION 30 FT +/- **BORING DEPTH** 44.8 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.910875° **LONGITUDE** -121.773461°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** 30 ft.
NOTES
▼ **AT END OF DRILLING** 30 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
30.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel
29.0		**Fat Clay (CH) [Fill]** stiff to very stiff, moist, dark brown with brown mottles, some fine sand, high plasticity
25.0	5	**Fat Clay (CH)** very stiff, moist, dark brown, some fine sand, high plasticity
20.5	10	**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, some silt, low plasticity
17.5		**Silty Sand (SM)** medium dense, moist, brown, fine to medium sand
12.5	15	**Poorly Graded Sand with Silt (SP-SM)** medium dense to dense, moist, brown, fine to medium sand
	20	
	25	
4.0		

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
14	MC-1		25		
21	MC-2B	97	21		
21	MC-3	94	28		
26	MC-4	95	21		
25	MC-5B	103	23		
27	MC-6B	98	9		
	ST				
28	SPT-8		8		
33	SPT				
62	MC-10B	90	4		



Continued Next Page

BORING NUMBER EB-1

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
4.0			**Poorly Graded Sand with Silt (SP-SM)** medium dense to dense, moist, brown, fine to medium sand							
				34	SPT-11		13			
	▼ 30									
-1.5			**Poorly Graded Sand with Silt (SP-SM)** very dense, wet, gray and brown, fine to coarse sand							
				50 5.5"	SPT-12		16			
	35									
				50 5"	SPT-13		22			
	40									
			decreasing coarse sand	50 4.5"	SPT-14		21			
-14.8	45		Bottom of Boring at 44.8 feet.							
	50									
	55									

BORING NUMBER EB-2

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 12/21/16 **DATE COMPLETED** 12/21/16
GROUND ELEVATION 31 FT +/- **BORING DEPTH** 36.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.910540° **LONGITUDE** -121.773219°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** 32 ft.
NOTES
▼ **AT END OF DRILLING** 31 ft.

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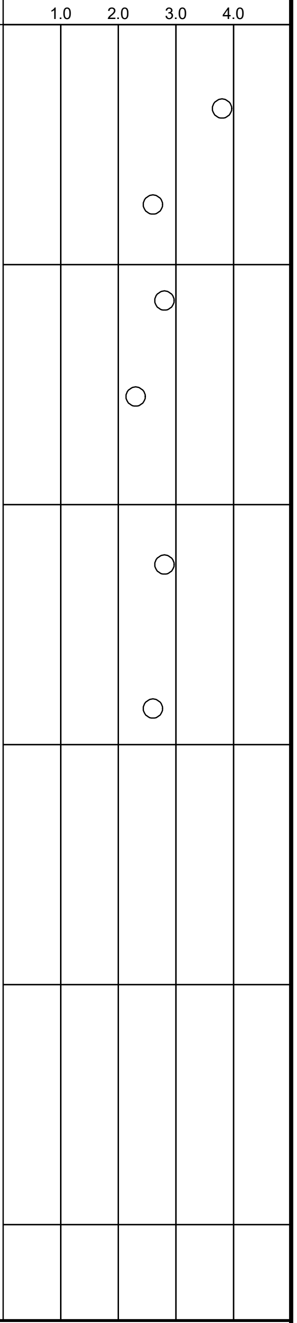
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
31.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel						
30.0		**Fat Clay (CH)** very stiff, moist, dark brown, some fine sand, high plasticity						
			25	MC-1	99	21		
			18	MC-2	96	22		
	5							
			22	MC-3B	98	22		
24.5		**Lean Clay with Sand (CL)** very stiff, moist, gray and brown mottled, fine sand, moderate plasticity						
			21	MC-4B	104	21		
	10							
				ST-5	94	27		
18.5		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity						
			22	MC-6B	93	29		
	15							
15.5		**Clayey Sand (SC)** medium dense, moist, brown, fine to medium sand	47	MC-7C	93	11		
13.0		**Poorly Graded Sand with Silt (SP-SM)** medium dense to dense, moist, brown, fine to medium sand						
			19	MC-8B	94	9		7
	20							
			29	SPT				
		becomes dense	35	SPT-10		10		
	25							
5.5								
5.0								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



Continued Next Page

BORING NUMBER EB-2

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
5.0		**Lean Clay with Sand (CL)** very stiff, moist, gray and brown mottled, fine sand, moderate plasticity							
3.5		**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown, fine to medium sand	39	MC-11C	96	6			○
	30								
		becomes wet, some fine gravel	32	MC-12B	101	19			
-2.0		**Silty Sand (SM)** medium dense to dense, moist, brown, fine to medium sand							
			20	SPT					
	35								
			46	SPT-14		25			
-5.5		Bottom of Boring at 36.5 feet.							
	40								
	45								
	50								
	55								

BORING NUMBER EB-3
PAGE 1 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 12/21/16 **DATE COMPLETED** 12/21/16
GROUND ELEVATION 51 FT +/- **BORING DEPTH** 50 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.91081° **LONGITUDE** -121.77136°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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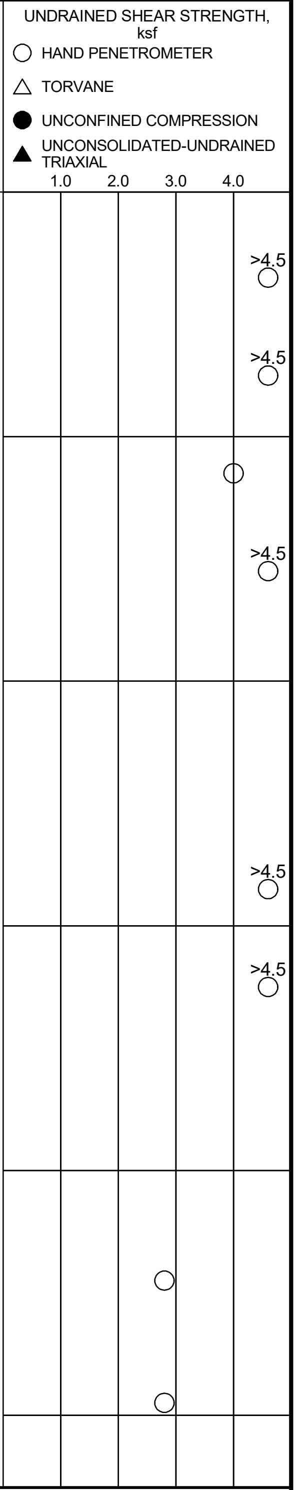
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
51.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel							
50.0		**Fat Clay (CH)** hard, moist, dark brown, some fine sand, high plasticity	28	MC-1B	98	20			>4.5
48.0		**Lean Clay with Sand (CL)** hard, moist, brown with light brown mottles, fine sand, moderate plasticity	23	MC-2B	92	21			>4.5
46.0	5	**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	26	MC-3B	82	37			
44.0		**Elastic Silt (MH)** hard, moist, brown with gray mottles, some fine sand, high plasticity	29	MC-4B	90	32			>4.5
	10								
40.0		**Silt with Sand (ML)** hard, moist, brown, fine sand, low plasticity							
			69	MC-5B	98	16			>4.5
	15								
		some thin interbedded clay layers	59	MC-6B	98	22			>4.5
31.5	20	**Silty Sand (SM)** medium dense, moist, brown, fine sand							
29.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	40	MC-7C	86	35			
	25		32	MC					
25.5									



Continued Next Page

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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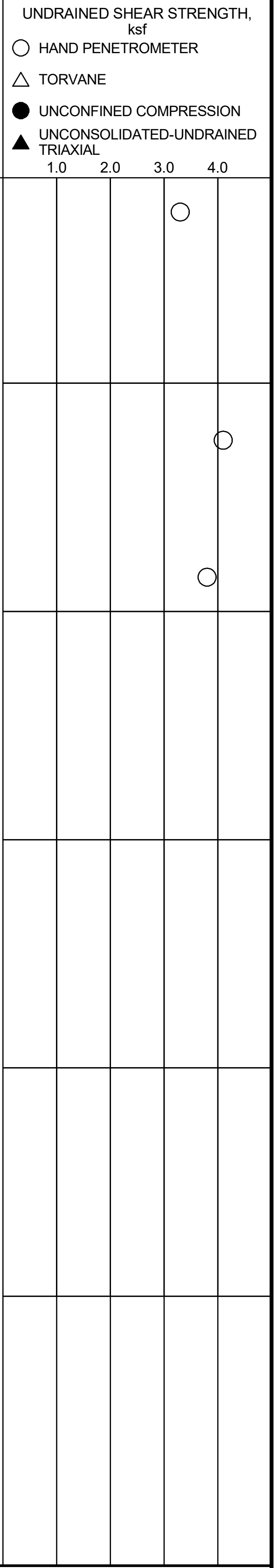
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
25.5			**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	37	MC-9B	87	33		
	30		hard	45	MC-10B	89	31		
	35		some thin interbedded silt layers	39	MC-11B	94	31		
13.5	40		**Silty Sand (SM)** medium dense, moist, brown, fine to medium sand	39	MC-12B	103	9		40
	45			57	MC-13B	101	9		
4.5			**Poorly Graded Sand with Silt (SP-SM)** dense, moist, brown, fine to medium sand	68	MC-14C	99	4		
1.0	50		Bottom of Boring at 50.0 feet.						



BORING NUMBER EB-4

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 11/22/16 **DATE COMPLETED** 11/22/16
GROUND ELEVATION 69 FT +/- **BORING DEPTH** 41.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.91022° **LONGITUDE** -121.77205°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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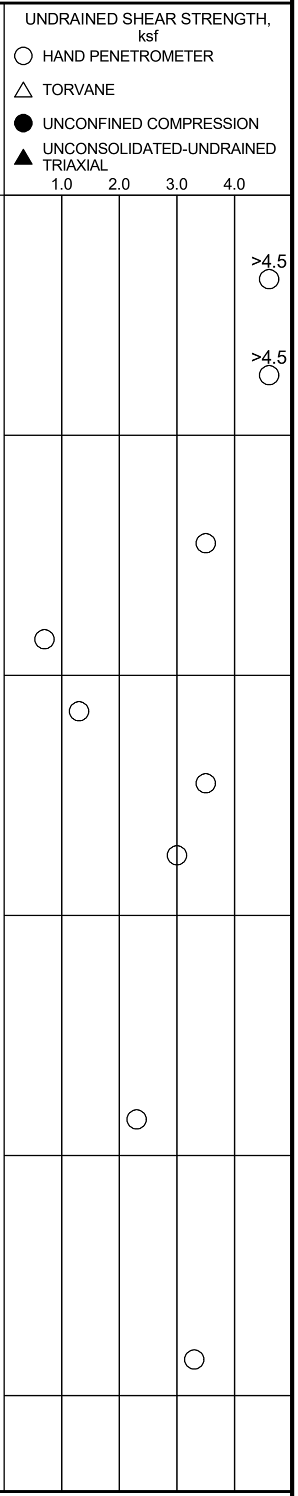
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
69.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel
68.0		**Fat Clay (CH)** hard, moist, gray with brown mottles, some fine sand, high plasticity Liquid Limit = 80, Plastic Limit = 25
64.5		**Silty Sand (SM)** medium dense, moist, brown, fine sand
62.5		**Silt with Sand (ML)** very stiff, moist, brown, fine sand, low plasticity
60.8		**Lean Clay with Sand (CL)** medium stiff to stiff, moist, brown, fine sand, some silt, moderate plasticity
57.5		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity
54.0	15	**Silty Sand (SM)** medium dense, moist, brown, fine sand
52.3		**Lean Clay (CL)** very stiff, moist, brown, some fine sand, some silt, low plasticity
48.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity
43.0		

Continued Next Page

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
34	MC-1B	91	30	55		>4.5
41	MC					>4.5
25	MC-3B	87	12			
22	SPT-4		20		79	
8	SPT-5		34			
13	SPT					
30	MC-7B	83	37			
35	MC-8B	85	37			
24	MC-9B	87	34			
32	MC-10B	83	41			



CORNERSTONE EARTH GROUP

BORING NUMBER EB-4

PAGE 2 OF 2

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
43.0									
42.0		**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, some silt, low plasticity							
			45	MC-11B	94	30			
	30								
		becomes hard	33	MC-12B	94	30			>4.5
	35								
			38	MC-13B	106	13			
29.5		**Silty Sand (SM)** dense, moist, brown, fine sand							
	40		34	SPT					
27.5		Bottom of Boring at 41.5 feet.							
	45								
	50								
	55								

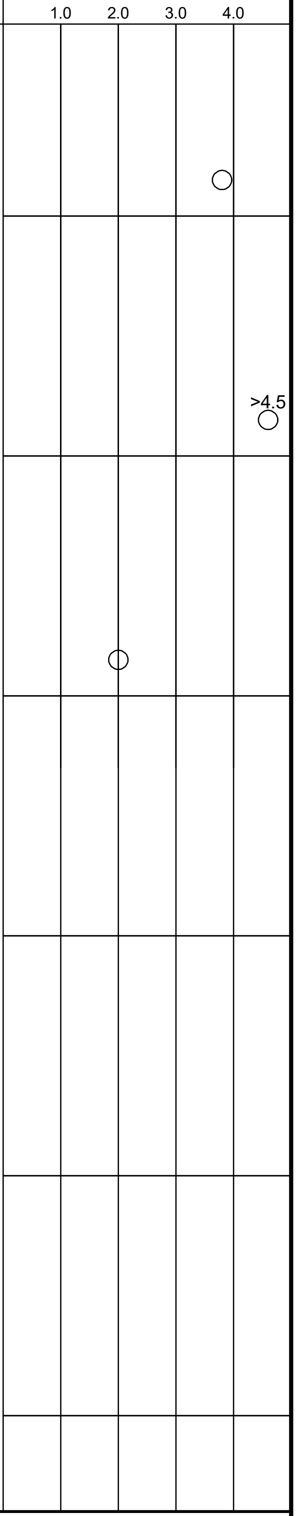
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

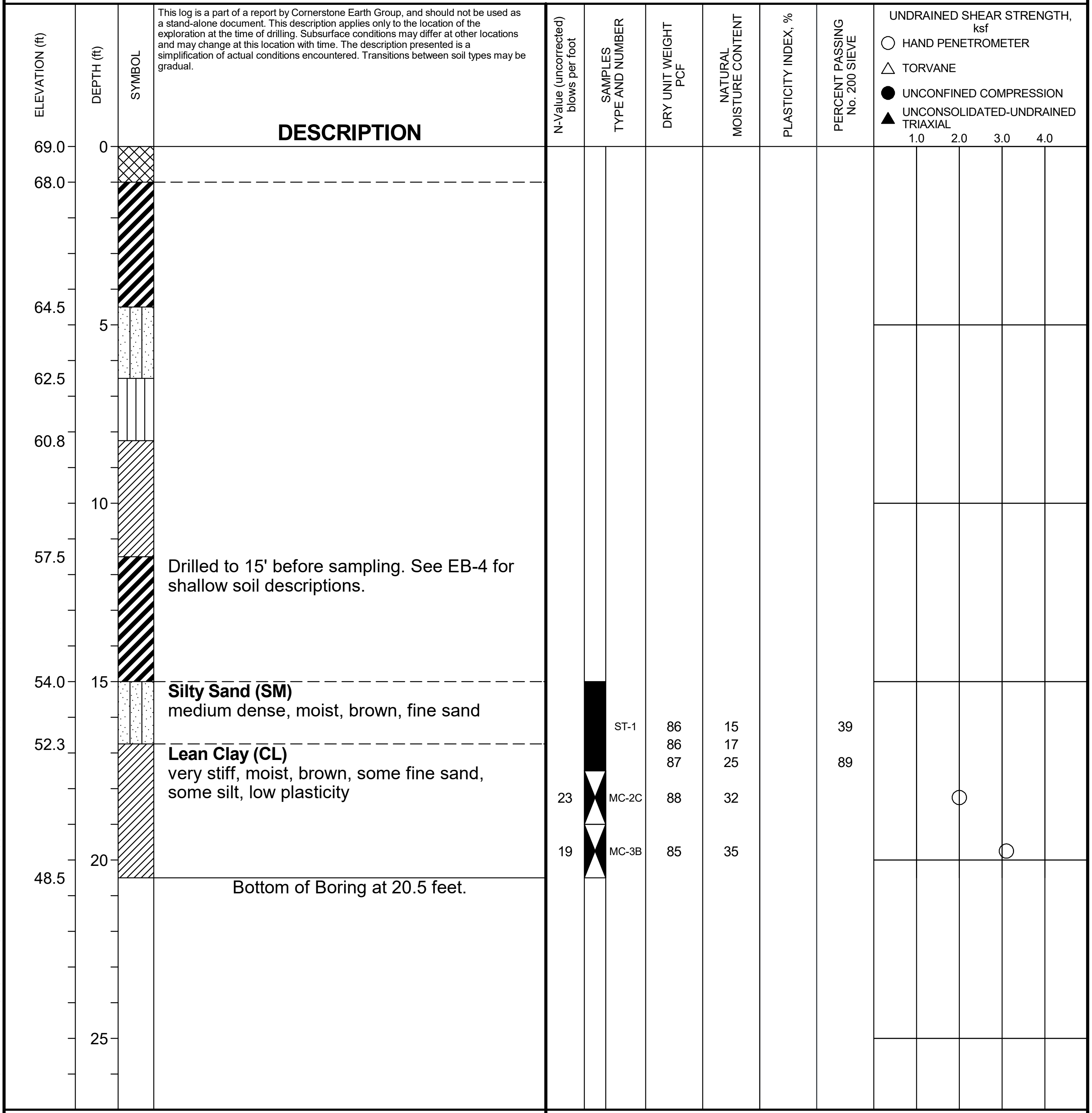


BORING NUMBER EB-4A

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 12/21/16 **DATE COMPLETED** 12/21/16
GROUND ELEVATION 69 FT +/- **BORING DEPTH** 20.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.91022° **LONGITUDE** -121.77205°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered



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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
69.0	0								
68.0									
64.5									
62.5									
60.8									
57.5		Drilled to 15' before sampling. See EB-4 for shallow soil descriptions.							
54.0	15	**Silty Sand (SM)** medium dense, moist, brown, fine sand		ST-1	86	15		39	
52.3		**Lean Clay (CL)** very stiff, moist, brown, some fine sand, some silt, low plasticity			86 87	17 25		89	
			23	MC-2C	88	32			○
			19	MC-3B	85	35			○
48.5		Bottom of Boring at 20.5 feet.							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-5

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 12/28/16 **DATE COMPLETED** 12/28/16
GROUND ELEVATION 48 FT +/- **BORING DEPTH** 54.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.90965° **LONGITUDE** -121.77188°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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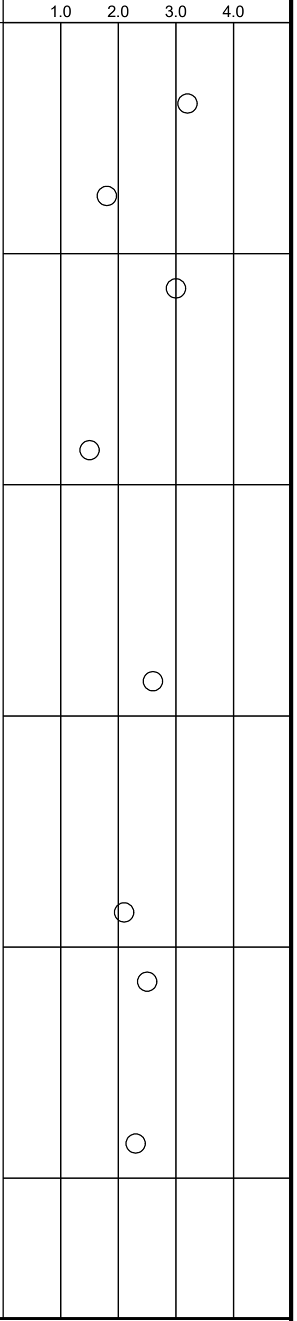
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
48.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel						
47.0		**Fat Clay (CH)** very stiff, moist, dark brown, some fine sand, high plasticity						
			39	MC-1B	97	25		
45.0		**Fat Clay (CH)** stiff to very stiff, moist, gray with brown mottles, some fine sand, high plasticity						
			34	MC-2B	79	39		
	5		26	MC-3B	82	39		
40.5		**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, some silt, low plasticity						
			26	MC-4B	89	29		
	10							
36.0		**Lean Clay (CL)** very stiff, moist, gray with brown mottles, some fine sand, low to moderate plasticity						
			27	MC-5B	89	33		
	15							
30.5		**Silt (ML)** very stiff, moist, brown, fine sand, some thin interbedded clay layers, low plasticity						
			29	MC-6B	92	29		96
28.0	20	**Lean Clay with Sand (CL)** very stiff, moist, gray and brown mottled, fine sand, some silt, low plasticity						
			20	SPT				
25.5		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity						
			30	MC-8B	87	32		
	25							
21.0								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
21.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity							
		Some thin interbedded silt layers	51	MC-9B	92	29			○
	30								
16.5		**Silt with Sand (ML)** very stiff, moist, gray and brown mottled, fine sand, low plasticity							
			59	MC-10B	95	26			○
	35								
11.0		**Silty Sand (SM)** dense, moist, brown, fine sand							
			59	MC-11B	98	20		47	
	40								
5.0		**Fat Clay (CH)** hard, moist, gray with brown mottles, trace fine sand, high plasticity							
			67	MC-12C	89	32			>4.5 ○
	45								
1.0		**Silt with Sand (ML)** hard, moist, gray and brown mottled, fine sand, low plasticity							
			54	MC-13B	94	29			>4.5 ○
-2.0	50	**Poorly Graded Sand with Silt (SP-SM)** dense, moist, brown, fine to medium sand, some coarse sand							
			74	MC-14B	103	16			
-4.5		**Lean Clay with Sand (CL)** very stiff, moist, gray brown, fine sand, low plasticity	41	SPT					○
-5.8			50 6"	MC					
-6.5		**Silty Sand (SM)** very dense, moist, brown, fine sand							
	55								

Bottom of Boring at 54.5 feet.

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-6

PAGE 1 OF 2

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA
DATE STARTED 11/22/16 **DATE COMPLETED** 11/22/16
GROUND ELEVATION 67 FT +/- **BORING DEPTH** 35 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.90952° **LONGITUDE** -121.77284°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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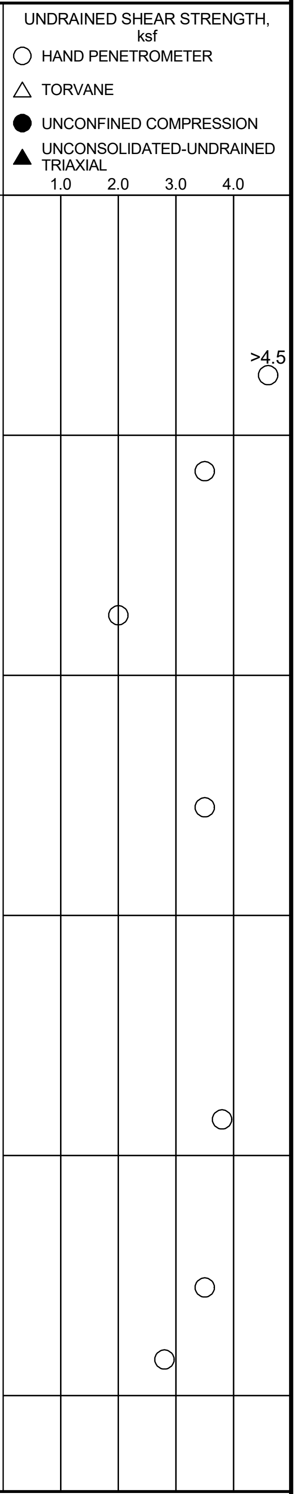
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
67.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to medium sand, fine to coarse gravel
64.0		**Fat Clay (CH)** hard, moist, dark brown, some fine sand, high plasticity Liquid Limit = 61, Plastic Limit = 20
61.8	5	**Silt (ML)** very stiff, moist, brown, fine sand, low plasticity
60.5		**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, some silt, low plasticity
59.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity
	10	
53.5		**Silt (ML)** very stiff, moist, brown, fine sand, low plasticity
52.0	15	**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity
	20	
	25	
41.0		

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
31	MC-1B	119	8		
29	MC-2B	115	20	41	
14	MC-3B	97	21		92
12	SPT				
15	MC-5B	81	36		
33	MC-6B	93	31		
24	SPT-7		27		90
26	MC-8B	87	33		
27	MC-9B	82	39		
32	MC				



Continued Next Page

BORING NUMBER EB-6

PAGE 2 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (1.0 2.0 3.0 4.0)
41.0		▨	**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity							
		▨		22	MC-11B	93	30			○ 2.5
	30	▨								
35.0		⁘	**Silty Sand (SM)** dense, moist, brown, fine sand, trace clay							
		⁘		58	MC-12B	113	10			
32.0	35		Bottom of Boring at 35.0 feet.							
	40									
	45									
	50									
	55									

BORING NUMBER EB-7

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

DATE STARTED 11/22/16 **DATE COMPLETED** 11/22/16

GROUND ELEVATION 56 FT +/- **BORING DEPTH** 25 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 36.91035° **LONGITUDE** -121.77263°

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
56.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, dark gray, fine to coarse sand, fine to coarse subangular gravel	49	MC-1B	142	8		
53.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	23	MC-2		32		
51.0	5	**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, some silt, low plasticity	13	MC-3B	97	24		
48.5		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	22	MC-4B	90	31		
	10							
			31	MC-5B	96	28		
	15							
			32	MC-6B	82	40		
	20							
34.0		**Silty Sand (SM)** dense, moist, brown, fine sand						
31.8		**Lean Clay with Sand (CL)** hard, moist, brown, fine sand, moderate plasticity	52	MC-7B	94	29		
31.0	25	Bottom of Boring at 25.0 feet.						

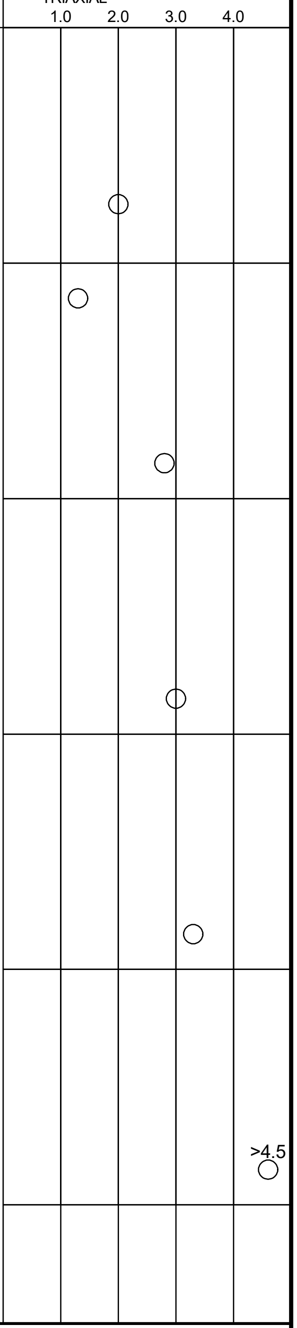
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-8

CORNERSTONE EARTH GROUP

PROJECT NAME Sunshine Vista Residential Development
PROJECT NUMBER 928-1-2
PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

DATE STARTED 11/22/16 **DATE COMPLETED** 11/22/16
GROUND ELEVATION 69 FT +/- **BORING DEPTH** 35 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.90998° **LONGITUDE** -121.77329°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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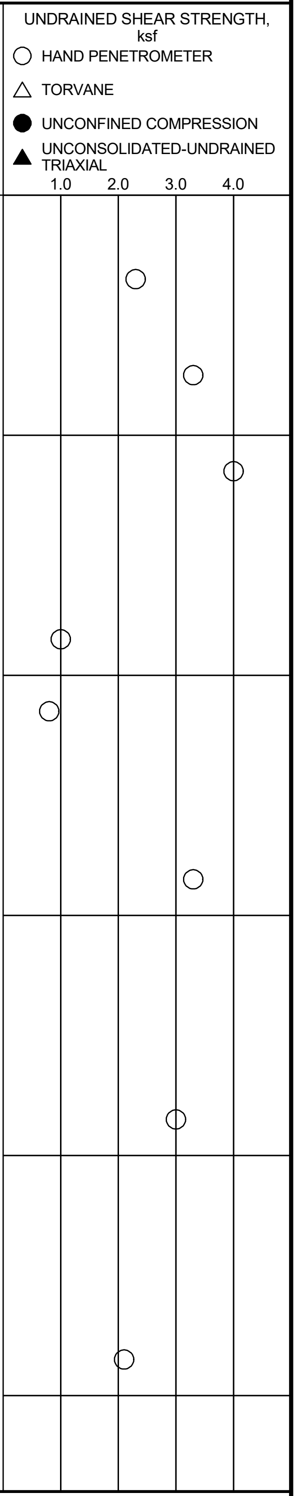
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
69.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, gray brown, fine to coarse sand, fine to coarse gravel						
68.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	19	MC-1B	85	34		
			26	MC				
	5		51	MC-3B	101	16		
63.0		**Silty Sand (SM)** medium dense, moist, brown, fine sand						
60.5		**Silt (ML)** medium stiff, moist, brown, some fine sand, some clay, low plasticity	15	SPT-4		34		90
	10		14	SPT-5		34		
56.5		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	53	MC-6B	84	37		
	15							
51.5		**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, some silt, low plasticity	41	MC-7B	92	32		
	20							
47.0		**Fat Clay (CH)** very stiff, moist, gray with brown mottles, some fine sand, high plasticity	28	MC-8B	73	43		
	25							
43.0								



Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-8

PAGE 2 OF 2

PROJECT NAME Sunshine Vista Residential Development

PROJECT NUMBER 928-1-2

PROJECT LOCATION 511 Ohlone Parkway, Watsonville, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (1.0 2.0 3.0 4.0)
43.0		▨								
42.0			**Silt (ML)** very stiff, moist, brown, some fine sand, some clay, low plasticity	35	MC-9B	93	28			○
	30									
38.0			**Silty Sand (SM)** dense, moist, brown, fine sand	64	MC-10B	104	12			
34.0	35		Bottom of Boring at 35.0 feet.							
	40									
	45									
	50									
	55									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 2/7/17 09:56 - P:\DRAFTING\GINT FILES\928-1-2 SUNSHINE VISTA RESIDENTIAL.GPJ

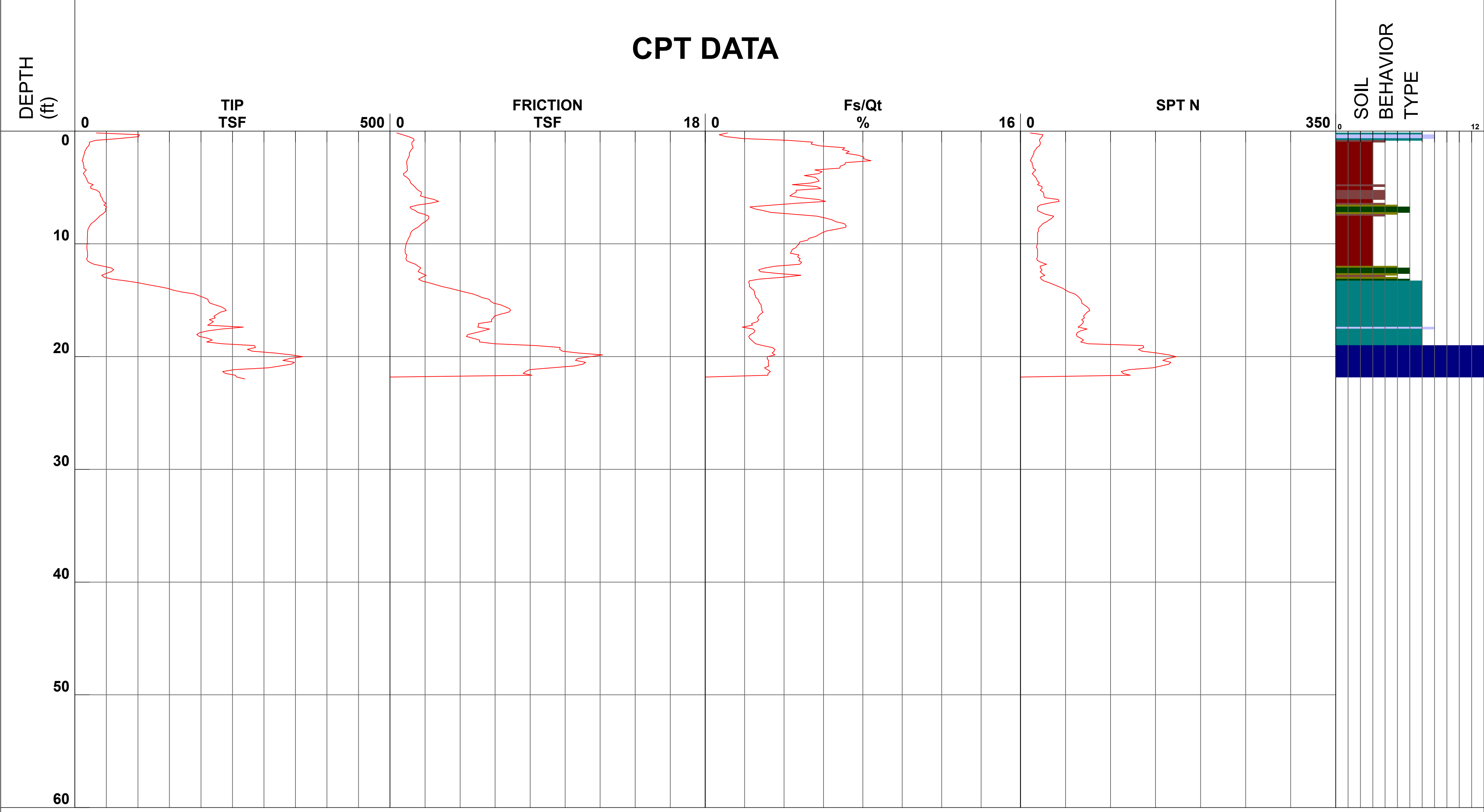
Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project Sunshine Vista Residential Development
Job Number 928-1-2
Hole Number CPT-01
EST GW Depth During Test 15.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/12/2016 10:29:29 AM
Filename SDF(362).cpt
GPS
Maximum Depth 21.98 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained	■ 4 - silty clay to clay	■ 7 - silty sand to sandy silt	■ 10 - gravelly sand to sand
■ 2 - organic material	■ 5 - clayey silt to silty clay	■ 8 - sand to silty sand	■ 11 - very stiff fine grained (*)
■ 3 - clay	■ 6 - sandy silt to clayey silt	■ 9 - sand	■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

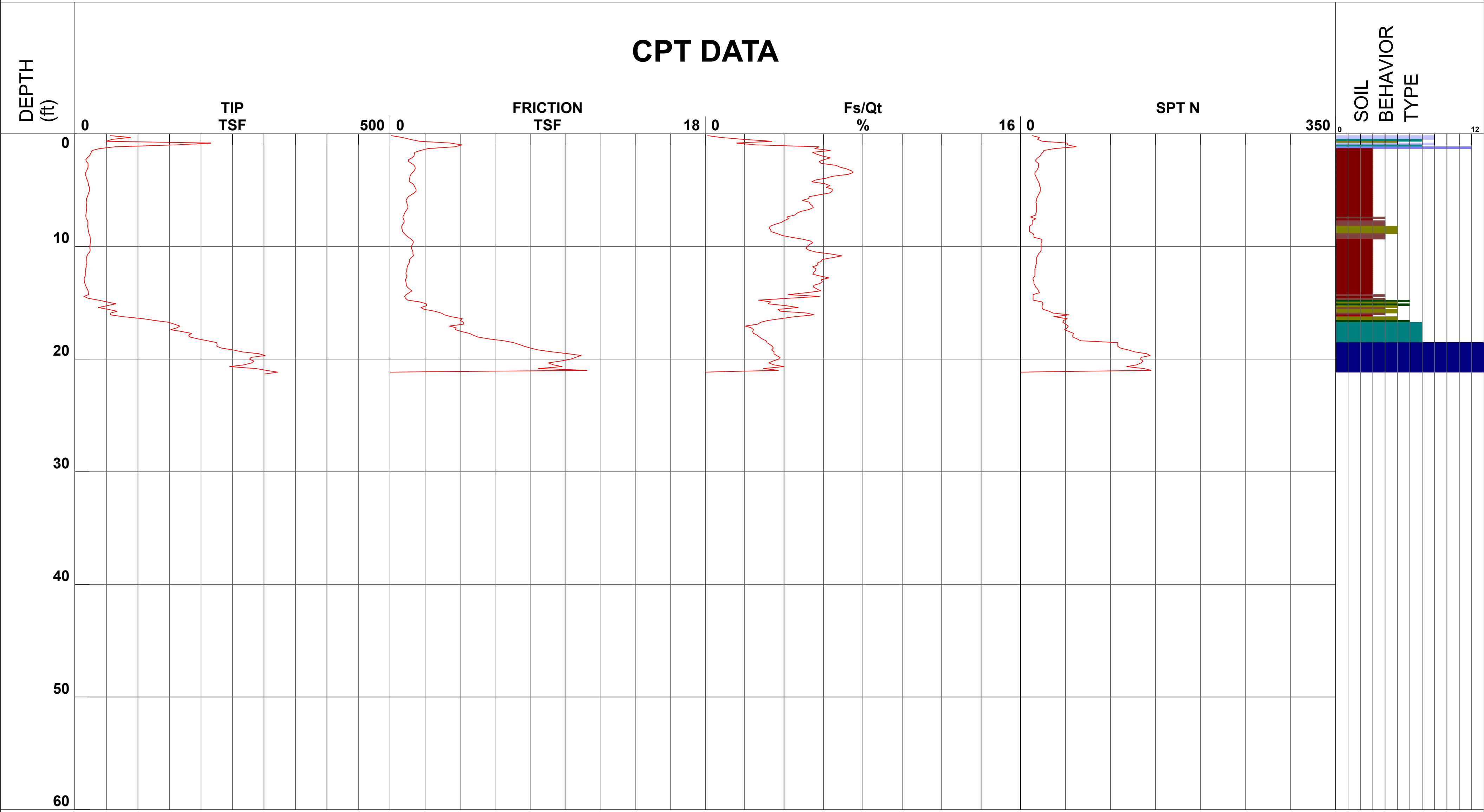
Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project Sunshine Vista Residential Developmen **Operator** KK-RB **Filename** SDF(364).cpt
Job Number 928-1-2 **Cone Number** DDG1333 **GPS**
Hole Number CPT-02 **Date and Time** 11/12/2016 12:13:17 PM **Maximum Depth** 21.33 ft
EST GW Depth During Test 15.20 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

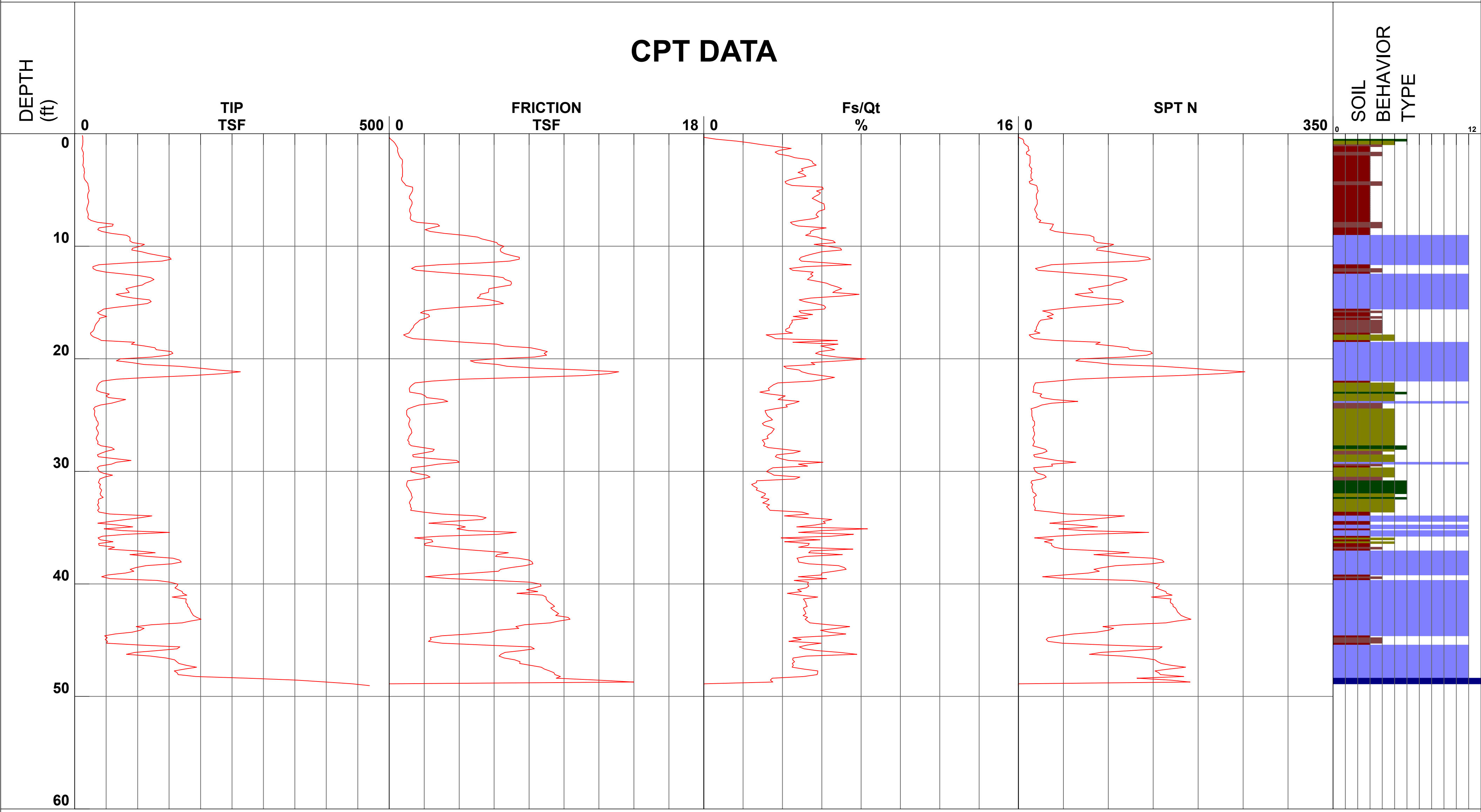
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-03
EST GW Depth During Test 27.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/12/2016 2:51:19 PM
Filename SDF(367).cpt
GPS
Maximum Depth 49.05 ft

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

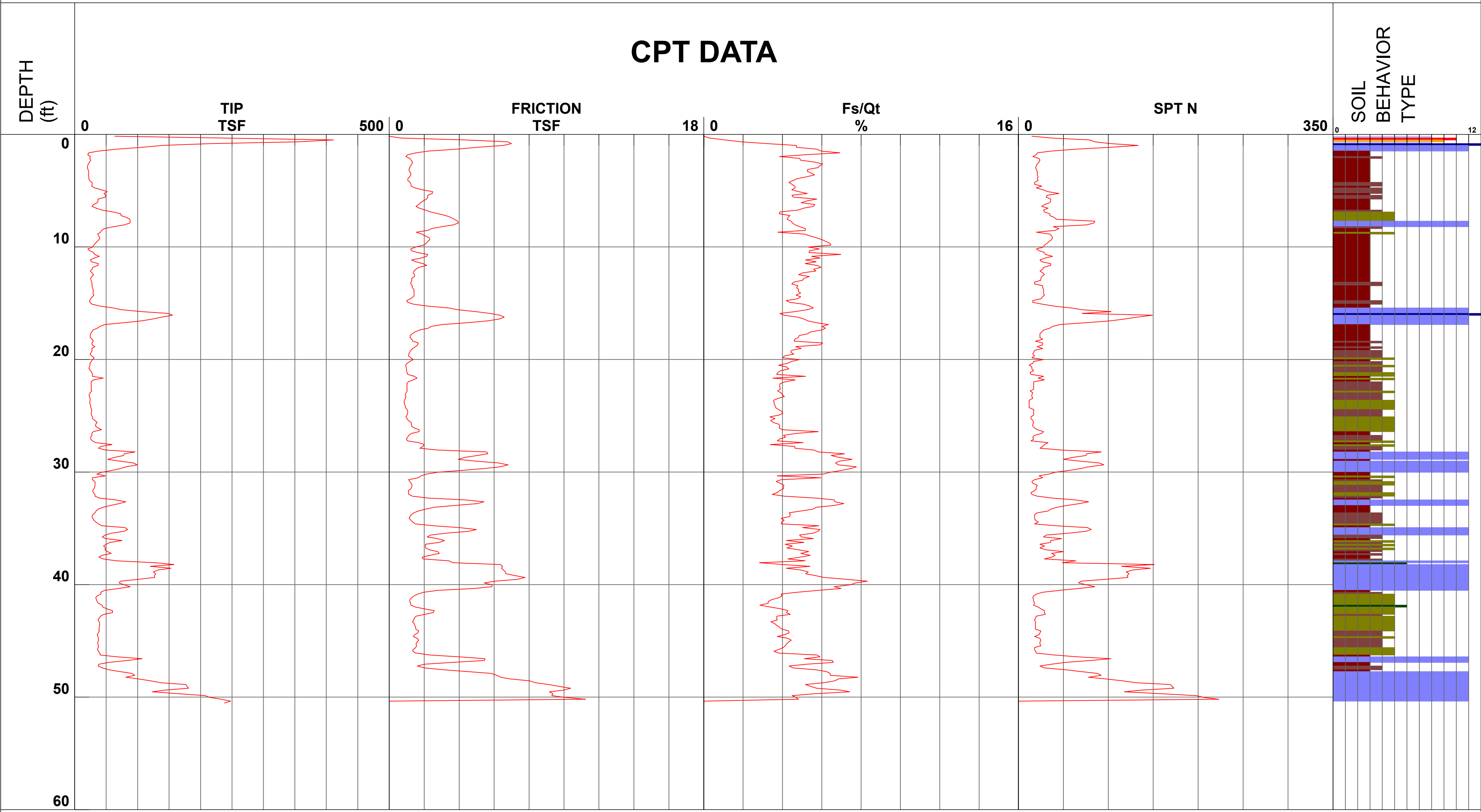
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	**Sunshine Vista Residential Developmen**	**Operator**	**KK-RB**	**Filename**	**SDF(370).cpt**
Job Number	**928-1-2**	**Cone Number**	**DDG1333**	**GPS**	
Hole Number	**CPT-04**	**Date and Time**	**11/13/2016 8:08:10 AM**	**Maximum Depth**	**50.52 ft**
EST GW Depth During Test	**4.60 ft**				

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

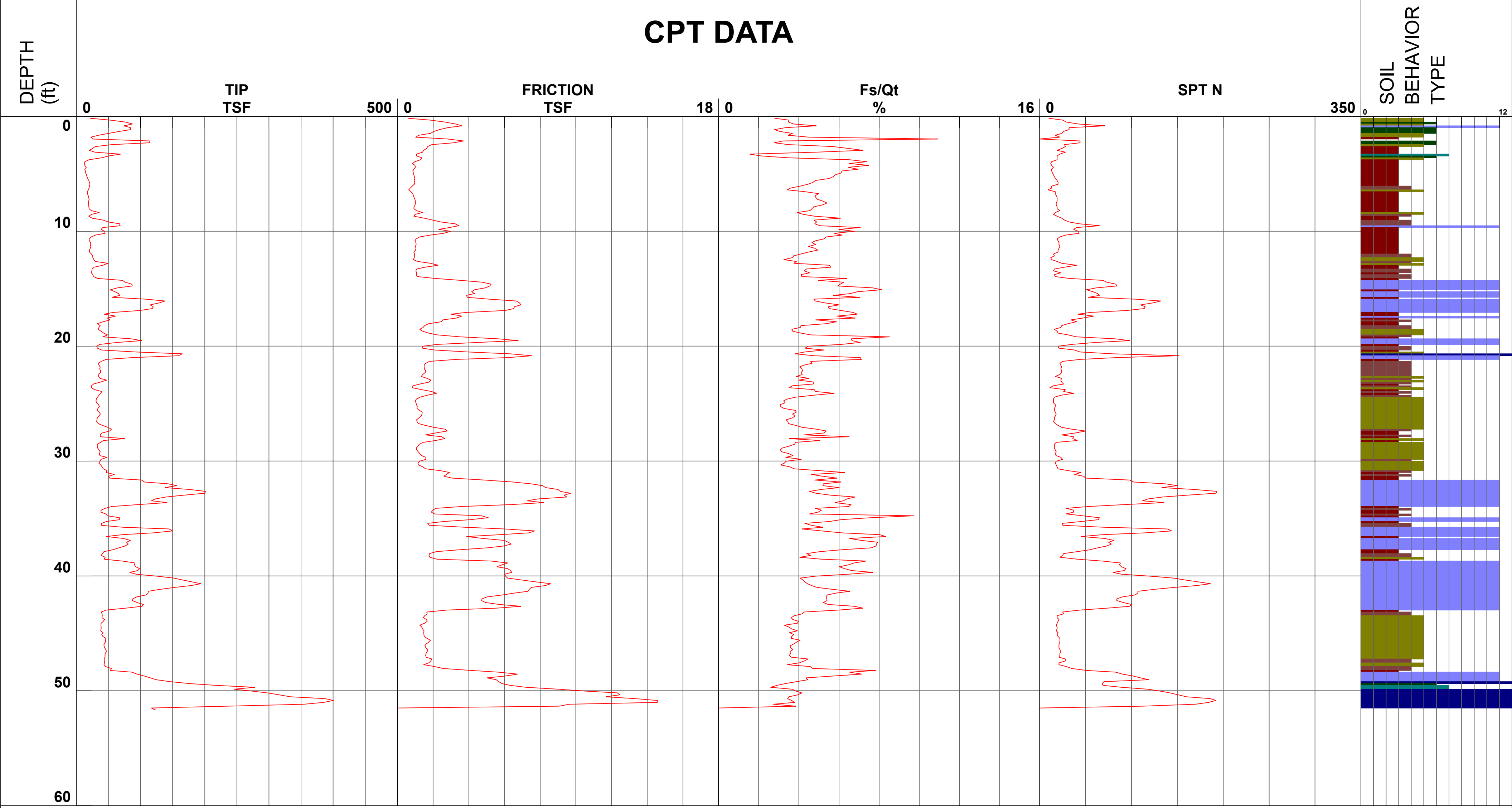
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project	Sunshine Vista Residential Developmen	Operator	KK-RB	Filename	SDF(365).cpt
Job Number	928-1-2	Cone Number	DDG1333	GPS	
Hole Number	CPT-05	Date and Time	11/12/2016 1:15:20 PM	Maximum Depth	51.67 ft
EST GW Depth During Test		27.70 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained	■ 4 - silty clay to clay	■ 7 - silty sand to sandy silt	🟥 10 - gravelly sand to sand
■ 2 - organic material	■ 5 - clayey silt to silty clay	■ 8 - sand to silty sand	■ 11 - very stiff fine grained (*)
■ 3 - clay	■ 6 - sandy silt to clayey silt	■ 9 - sand	■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

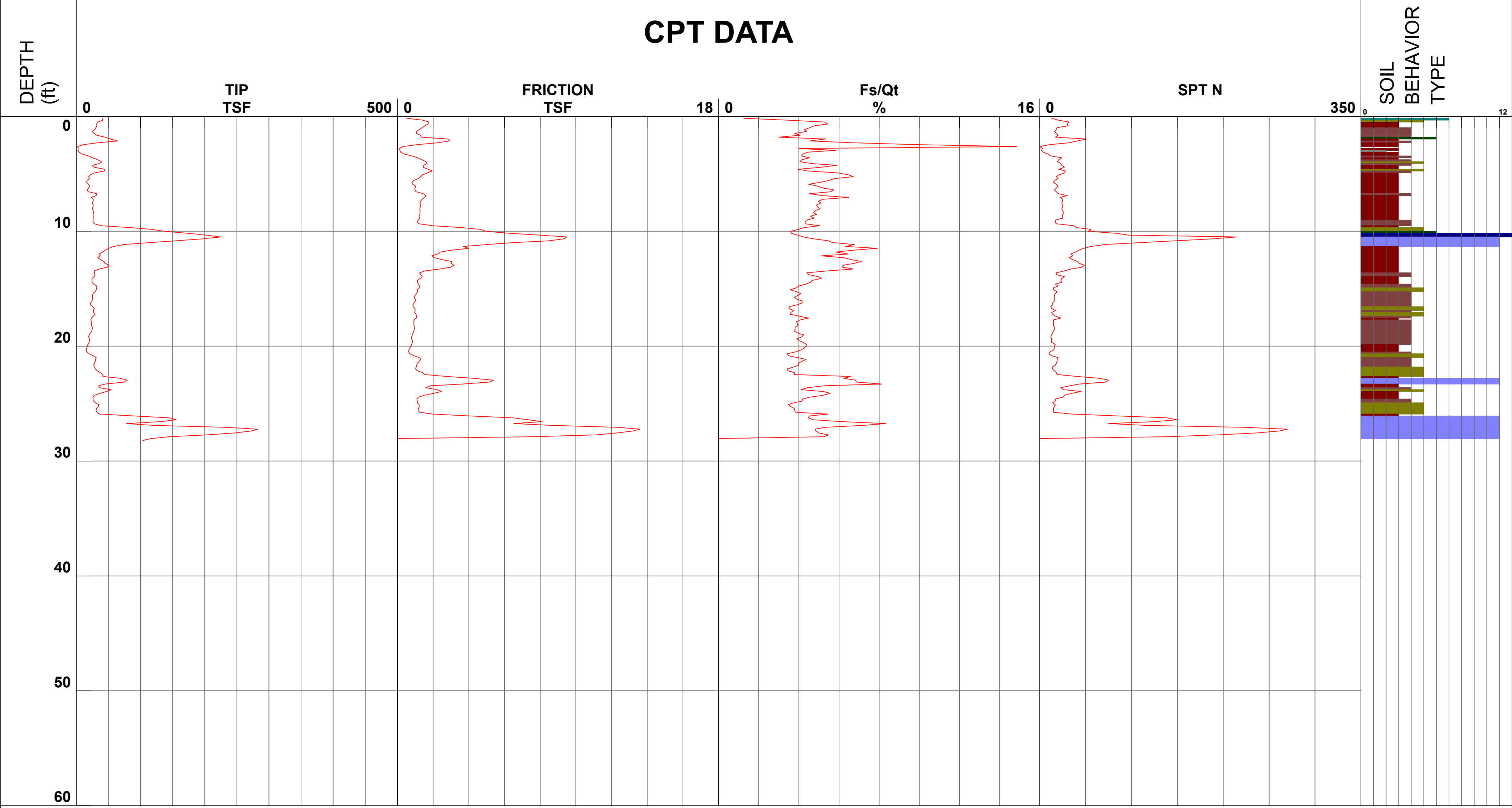
Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-06
EST GW Depth During Test 4.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/12/2016 3:40:00 PM
Filename SDF(368).cpt
GPS
Maximum Depth 28.21 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

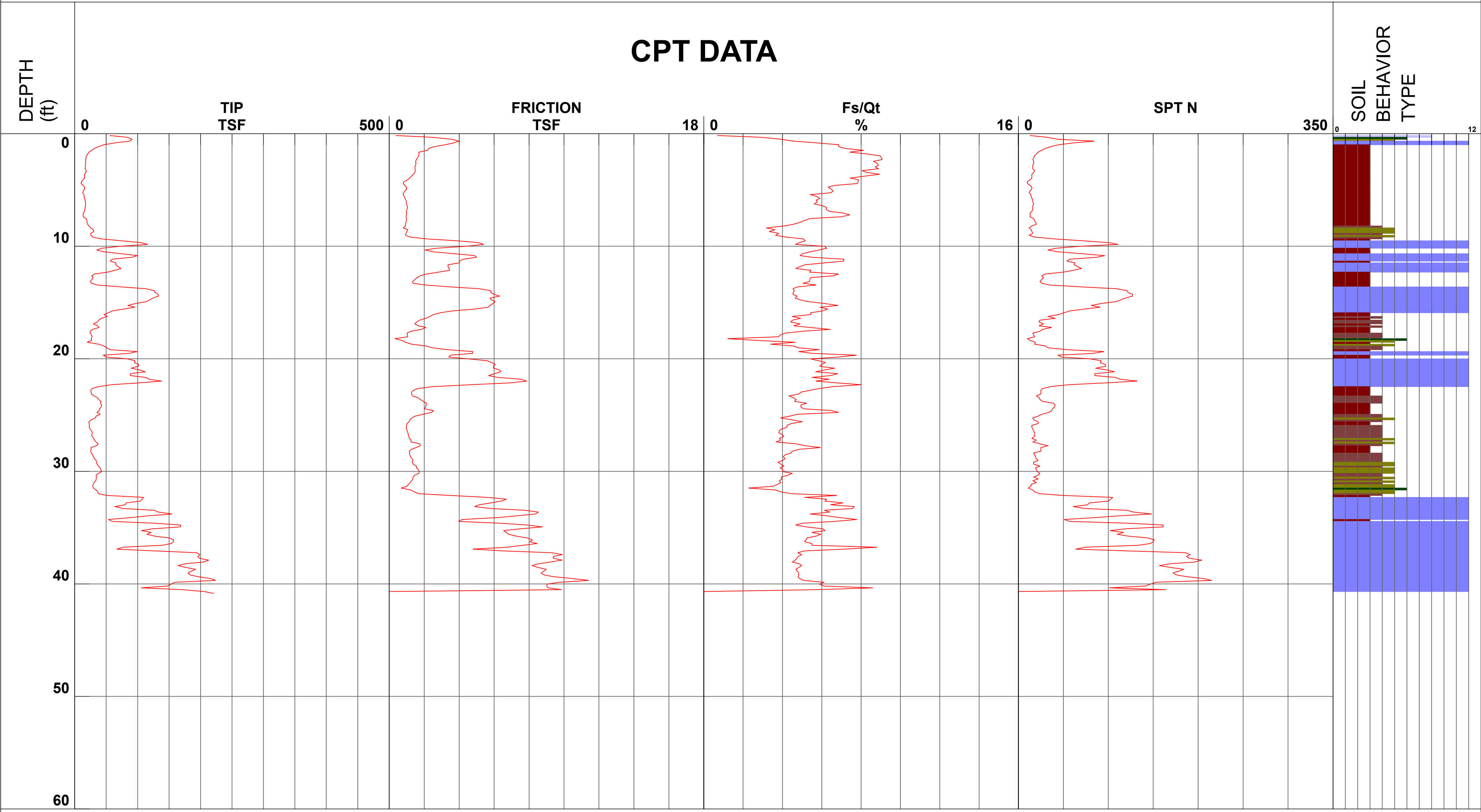
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-07
EST GW Depth During Test 4.20 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/13/2016 9:13:42 AM
Filename SDF(371).cpt
GPS
Maximum Depth 40.85 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

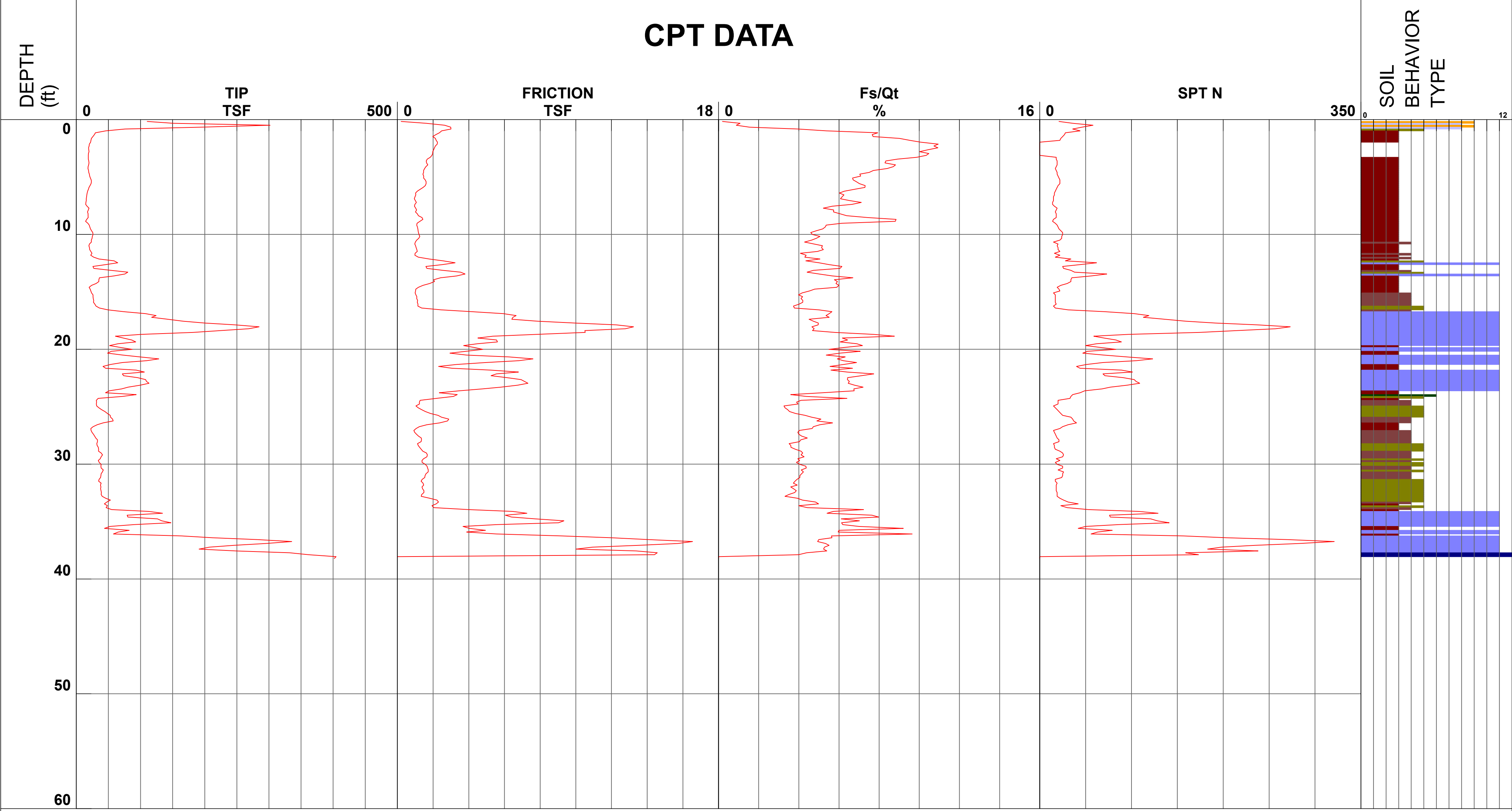
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-08
EST GW Depth During Test 4.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/13/2016 11:50:25 AM
Filename SDF(373).cpt
GPS
Maximum Depth 38.22 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

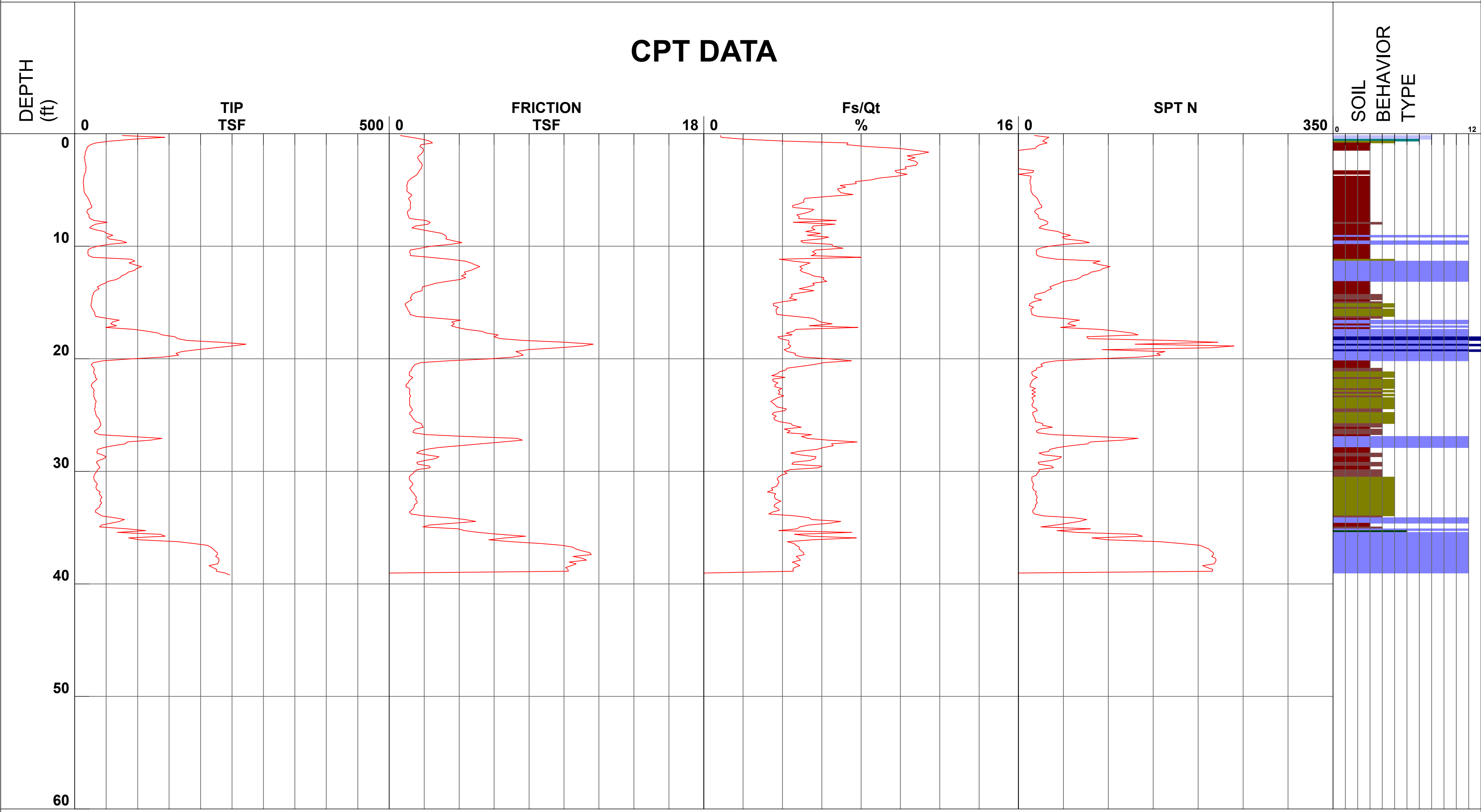
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-09
EST GW Depth During Test 4.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/12/2016 2:14:56 PM
Filename SDF(366).cpt
GPS
Maximum Depth 39.21 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

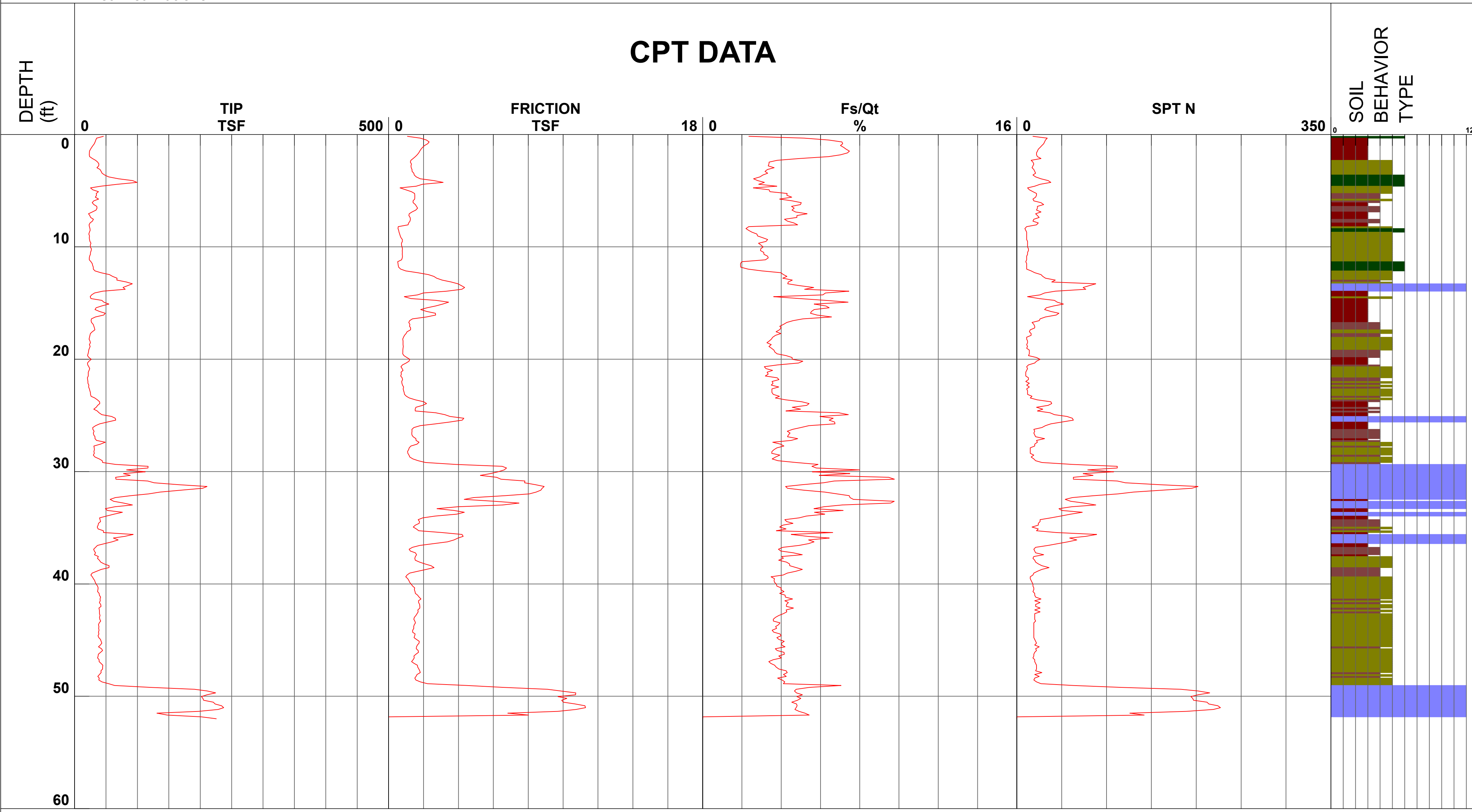
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-10
EST GW Depth During Test 4.00 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/13/2016 10:23:50 AM
Filename SDF(372).cpt
GPS
Maximum Depth 52.00 ft

Net Area Ratio .8



- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

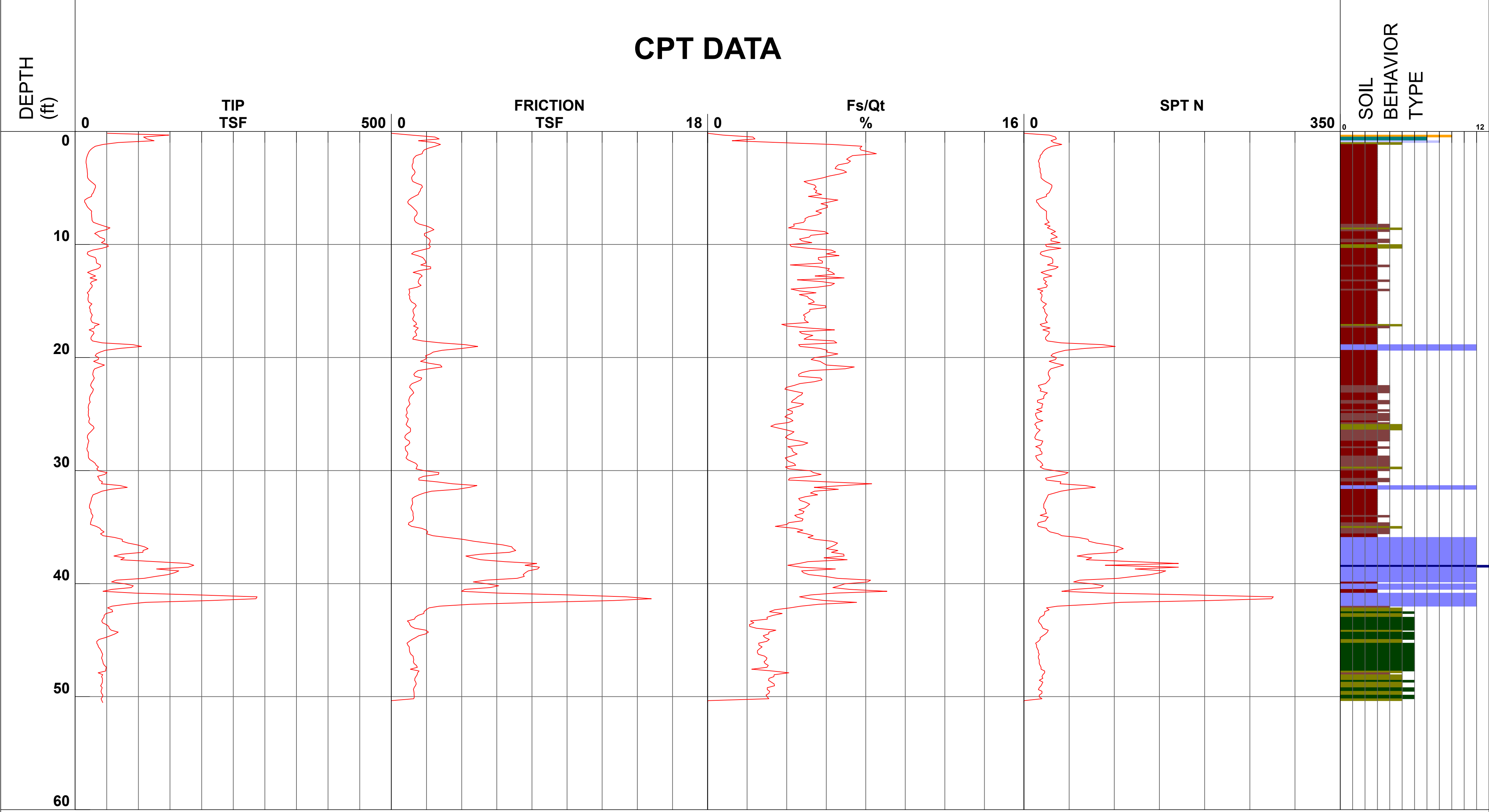
Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

Project Sunshine Vista Residential Developmen
Job Number 928-1-2
Hole Number CPT-11
EST GW Depth During Test 8.70 ft
Operator KK-RB
Cone Number DDG1333
Date and Time 11/12/2016 4:10:46 PM
Filename SDF(369).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8

CPT DATA



- ⬜ 1 - sensitive fine grained
- ⬛ 2 - organic material
- 🟥 3 - clay
- 🟫 4 - silty clay to clay
- 🟨 5 - clayey silt to silty clay
- 🟩 6 - sandy silt to clayey silt
- 🟦 7 - silty sand to sandy silt
- ⬜ 8 - sand to silty sand
- 🟧 9 - sand
- 🟥 10 - gravelly sand to sand
- 🟪 11 - very stiff fine grained (*)
- 🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 92 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 78 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 8 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

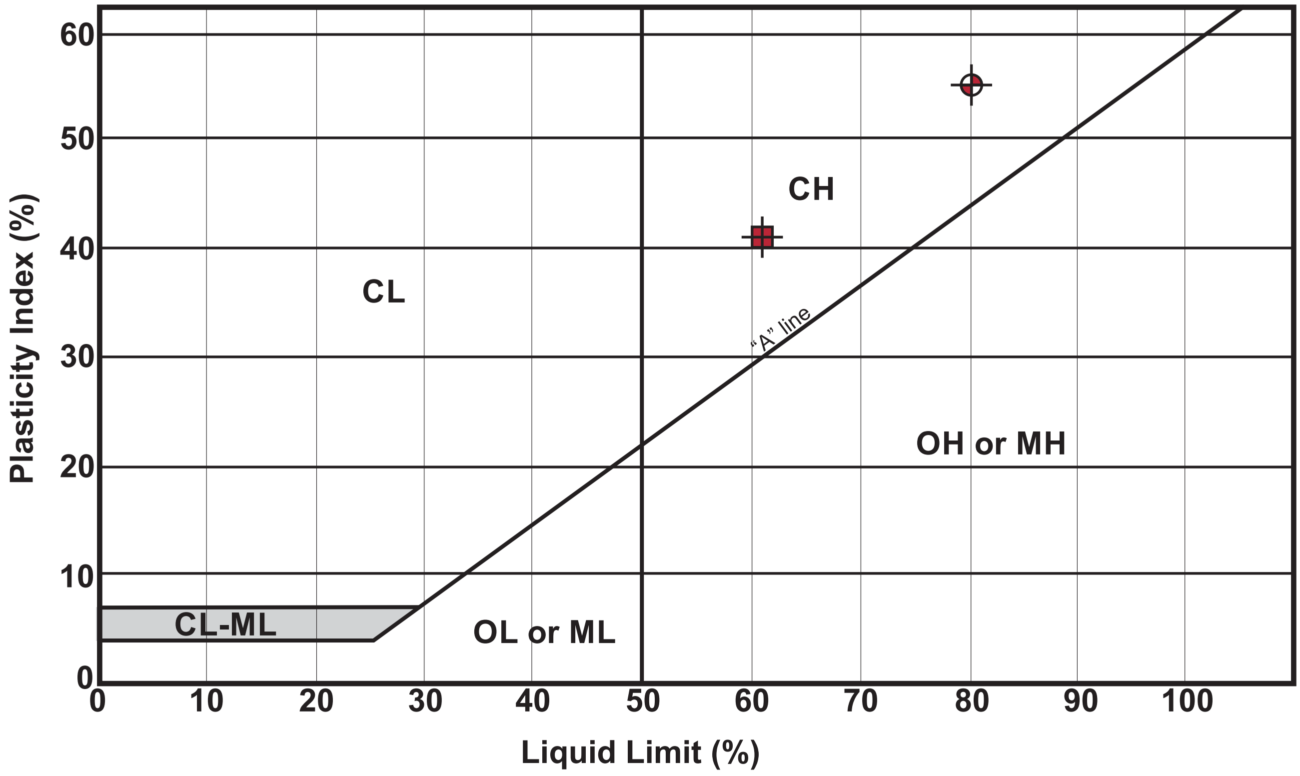
Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Consolidated-Undrained Triaxial Compression with Pore Pressure Measurements: The undrained shear strength was determined on three remolded and three relatively undisturbed sample of soil material by undrained triaxial shear strength testing with pore pressure measurements (ASTM D4767). The results of these tests are included as part of this appendix.

Consolidation: One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically in this appendix.

Compaction: One compaction test (ASTM D 1557-00 Method B) was performed on a composite sample of the subsurface soils to measure the relative maximum dry density and optimum moisture content. Results of the compaction test is included as part of this appendix.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-4	2.0	30	80	25	55	—	Fat Clay (CH)
■	EB-6	4.0	20	61	20	41	—	Fat Clay (CH)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Sunshine Vista
Residential Development
Watsonville, CA

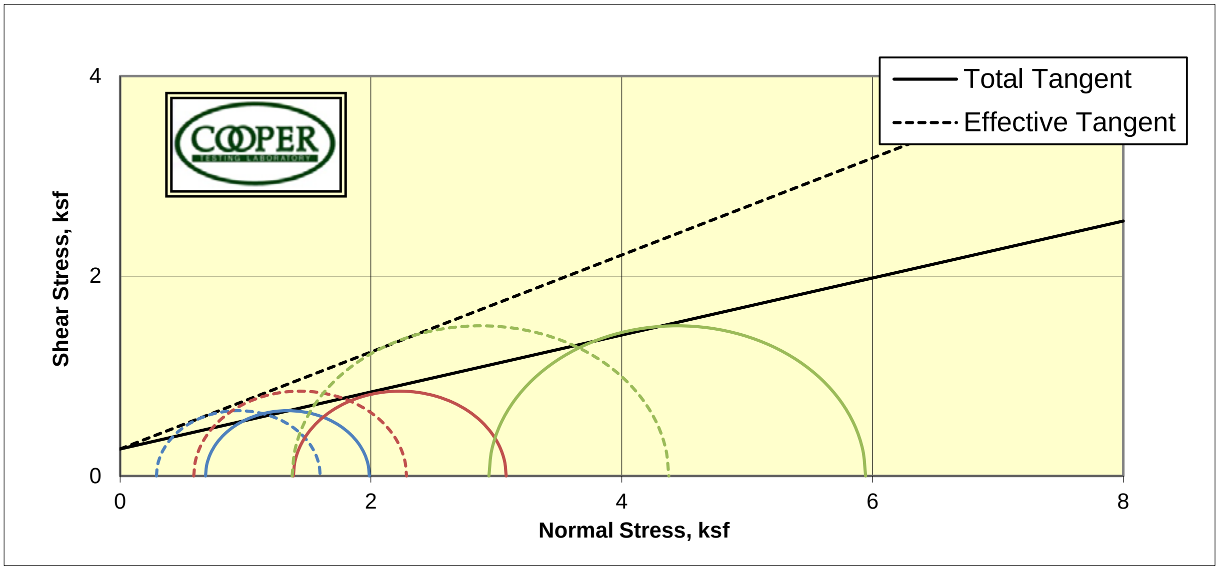
Project Number 928-1-2

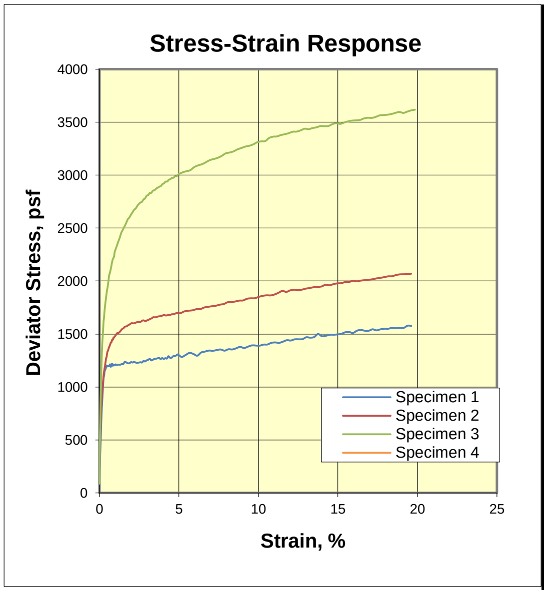
Figure Number Figure B1

Date February 2017

Drawn By FLL

Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767





Specimen	1	2	3	4
Boring	EB-4A	EB-4A	EB-4A	
Sample	Composite	Composite	Composite	
Depth	10-20	10-20	10-20	
Visual Description	Yellowish Brown CLAY	Yellowish Brown CLAY	Yellowish Brown CLAY	
MC (%)	15.1	15.1	15.1	
Dry Density (pcf)	102.9	102.9	102.7	
Saturation (%)	60.5	60.4	60.3	
Void Ratio	0.699	0.699	0.702	
Diameter (in)	2.38	2.38	2.38	
Height (in)	5.00	5.00	5.00	
	Final			
MC (%)	30.3	29.4	27.3	
Dry Density (pcf)	94.6	95.9	99.1	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.848	0.823	0.765	
Diameter (in)	2.45	2.44	2.41	
Height (in)	5.10	5.09	5.05	
Cell Pressure (psi)	115.0	119.8	130.2	
Back Pressure (psi)	110.2	110.2	109.8	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	1.306	1.697	3.004	
Excess PP (psi)	2.7	5.5	10.9	
Sigma 1 (ksf)	1.596	2.284	4.376	
Sigma 3 (ksf)	0.289	0.587	1.371	
P (ksf)	0.943	1.436	2.874	
Q (ksf)	0.653	0.849	1.502	
Stress Ratio	5.514	3.891	3.191	
Rate (in/min)	0.0004	0.0004	0.0004	

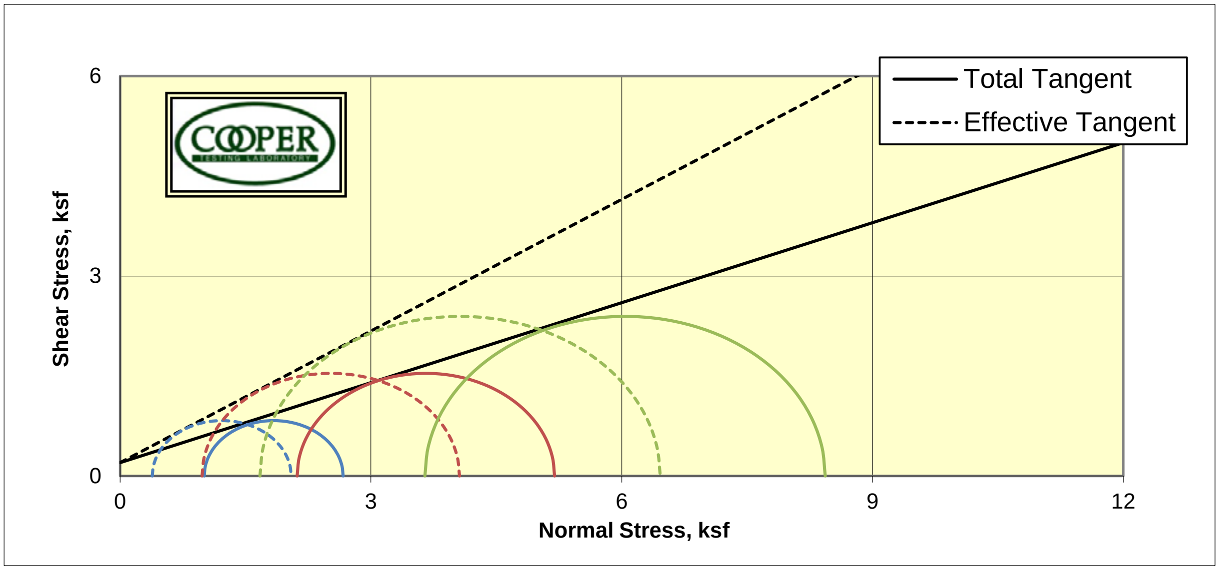
CTL Number: 640-1069
Client Name: Cornerstone Earth Group
Project Name: Sunshine Vista Residential
Project Number: 928-1-2
Date: 1/13/2017 By: MD/DC

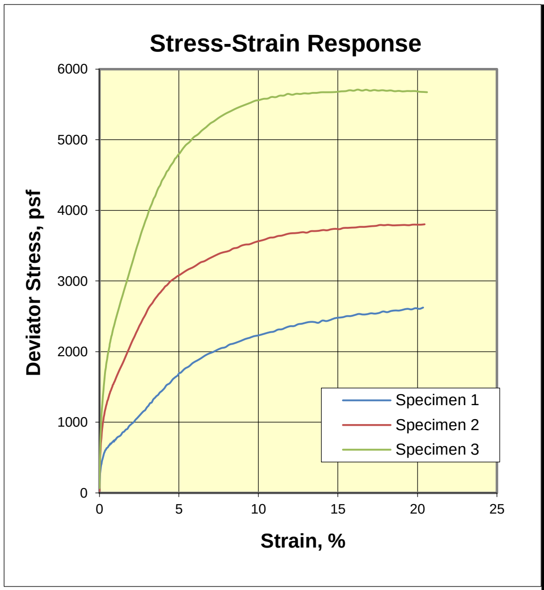
Total C	**0.270**	**ksf**
Total phi	**15.9**	**degrees**
Eff. C	**0.270**	**ksf**
Eff. Phi	**25.9**	**degrees**

©

Remarks: Remolded to 90% of 114.8 pcf @ 15.0%(OPT).

Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767





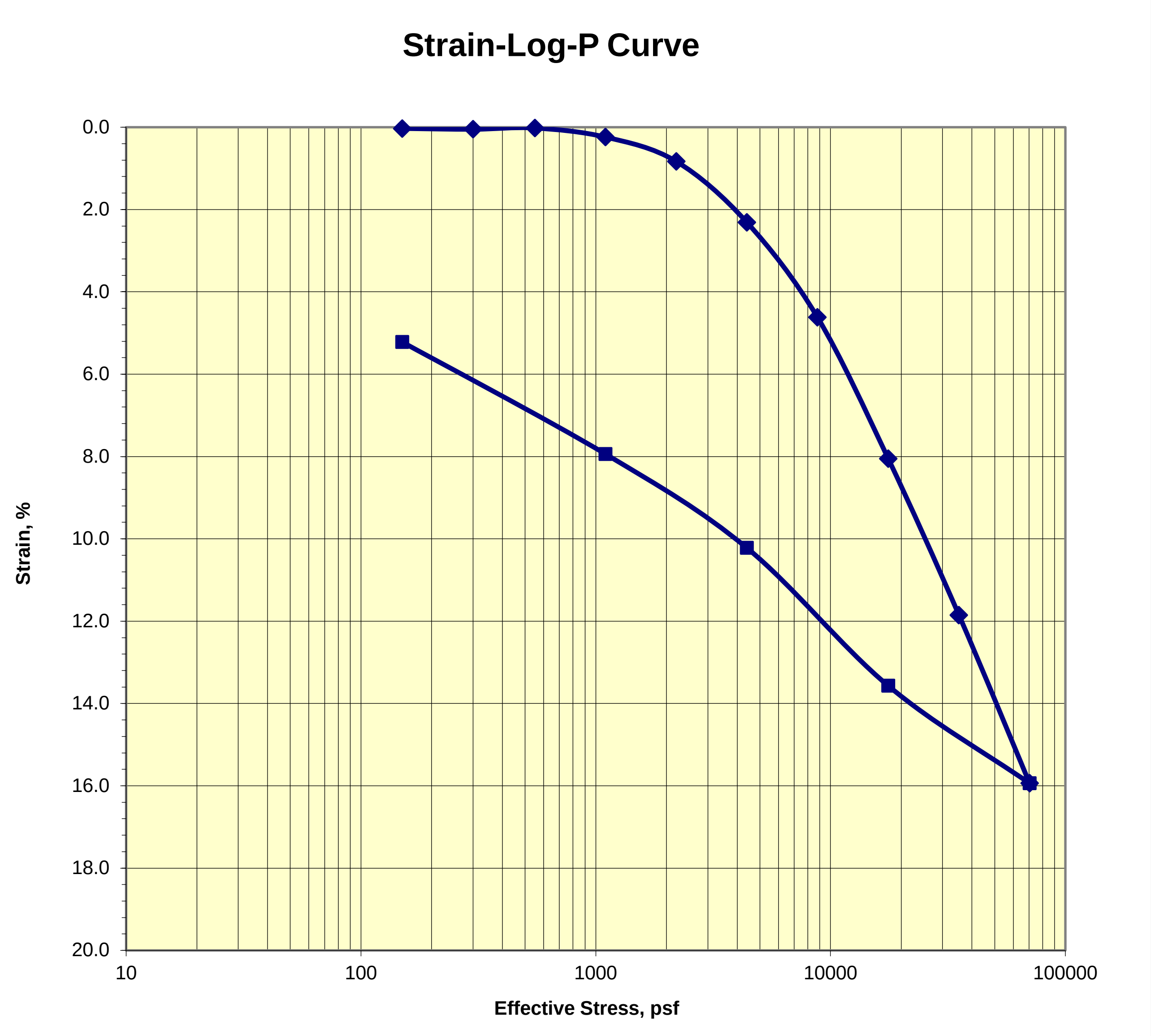
Specimen	1	2	3	4
Boring	EB-4A	EB-4	EB-4A	
Sample	1	1	1	
Depth	15(Tip-13.5")	15(Tip-6.5")	15(Tip-1/2")	
Visual Description	Light Olive Brown Silty SAND	Light Olive Brown Silty SAND	Light Olive Brown Silty SAND near Sandy SILT (more Silty than other 2 pts.)	
MC (%)	14.7	17.4	25.2	
Dry Density (pcf)	86.4	86.1	86.9	
Saturation (%)	42.6	50.0	74.0	
Void Ratio	0.916	0.921	0.904	
Diameter (in)	2.87	2.87	2.87	
Height (in)	6.07	6.07	6.08	
	Final			
MC (%)	31.1	31.4	31.1	
Dry Density (pcf)	90.7	90.3	90.7	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.824	0.832	0.824	
Diameter (in)	2.83	2.84	2.85	
Height (in)	5.97	5.92	5.91	
Cell Pressure (psi)	127.0	135.0	145.0	
Back Pressure (psi)	120.0	120.3	119.6	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	1.661	3.080	4.789	
Excess PP (psi)	4.3	7.9	13.7	
Sigma 1 (ksf)	2.046	4.060	6.461	
Sigma 3 (ksf)	0.385	0.980	1.672	
P (ksf)	1.215	2.520	4.067	
Q (ksf)	0.831	1.540	2.394	
Stress Ratio	5.321	4.142	3.864	
Rate (in/min)	0.0005	0.0005	0.0005	

CTL Number:	640-1069		
Client Name:	Cornerstone Earth Group		
Project Name:	Sunshine Vista Residentail		
Project Number:	928-1-2		
Date:	1/6/2017	By:	MD/DC
Total C	**0.200**	**ksf**	
Total phi	**21.8**	**degrees**	
Eff. C	**0.200**	**ksf**	
Eff. Phi	**33.4**	**degrees**	©

Consolidation Test

ASTM D2435

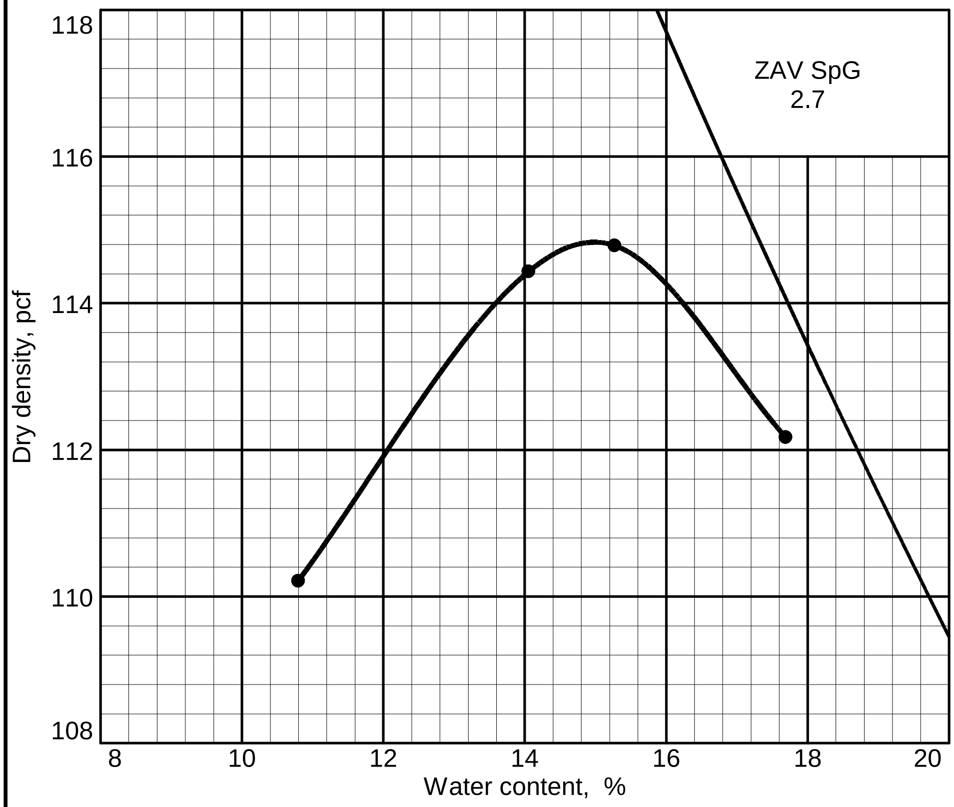
Job No.:	640-1069	**Boring:**	EB-2	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	5	**Reduced:**	PJ
Project:	928-1-2	**Depth, ft.:**	10(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Olive Brown CLAY			**Date:**	1/11/2017



Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	27.3	26.0
Dry Density, pcf:	93.6	99.0
Void Ratio:	0.801	0.702
% Saturation:	91.9	100.0

Remarks:

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-00 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: .03333 cu.ft.
Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM ______ **Sp.G.** 2.7
LL ______ **PI** ______
%>3/8 in. ______ **%<#200** ______
USCS ______ **AASHTO** ______

TESTING DATA

	1	2	3	4	5	6
WM + WS	8.87	8.93	8.92	8.59		
WM	4.52	4.52	4.52	4.52		
WW + T #1	907.60	1083.20	880.80	845.50		
WD + T #1	835.90	983.30	795.90	794.20		
TARE #1	325.70	329.00	315.90	319.00		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	14.1	15.3	17.7	10.8		
DRY DENSITY	114.4	114.8	112.2	110.2		

TEST RESULTS

Maximum dry density = 114.8 pcf

Optimum moisture = 15.0 %

Material Description

Yellowish Brown CLAY

Project No. 640-1069 **Client:** Cornerstone Earth Group
Project: Sunshine Vista Residential - 928-1-2

● **Source:** EB-4A **Sample No.:** Composite **Elev./Depth:** 10-20'

Remarks:

COMPACTION TEST REPORT

COOPER TESTING LABORATORY

Figure

APPENDIX D: PREVIOUS GEOTECHNICAL INVESTIGATION DATA

⌖ B-X Exploratory Boring ▨ T-X Exploratory Trench

Scale: 1" = 80'

Note: Aerial photograph provided by Weber, Hayes & Associates, June 2016.

BUTANO	BORING SITE PLAN	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	B-2a

KEY TO LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of the material is larger than the No. 200 sieve	GRAVELS More than half of the coarse fraction is larger than the No. 4 sieve	CLEAN GRAVELS (Less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS More than half of the coarse fraction is smaller than the No. 4 sieve	CLEAN SANDS (Less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS More than half of the material is smaller than the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomacaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GRAIN SIZE LIMITS

SILT AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. 12 in.

US STANDARD SIEVE SIZE

RELATIVE DENSITY	
SAND AND GRAVEL	BLOWS/FT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY	
SILT AND CLAY	BLOWS/FT*
VERY SOFT	0 - 2
SOFT	2 - 4
FIRM	4 - 8
STIFF	8 - 16
VERY STIFF	16 - 32
HARD	OVER 32

MOISTURE CONDITION	
CLAY	DRY
	MOIST
	SATURATED
SAND	DRY
	DAMP
	WET
	SATURATED

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 inch I.D.) split spoon (ASTM D-1586).

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE B-3

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: June 20, 2016
Logged By: AP
Boring: B1
Location:
Elevation:
Method of Drilling: 6 inch solid stem truck mounted auger

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Swell (psf)	Particle Size	Atterberg Limits	
												L.L.	P.I.
	SM	⧅		Grey silty SAND with gravel (FILL).									
	CH			Dark brown FAT CLAY, stiff, slightly moist (FILL).	29	11	93.8	27.8		2,300			
		◫			19	15		24.7					
5		⧅											
	CL			Brown-tan mottled LEAN CLAY with sand , stiff, moist.	38	14	93.1	26.7					
		◫			12	9		34.1					
			⊠						35		✓	42	20
10		⧅											
					17	9		38.8					
		◫		Very stiff.	22	18		39.6					
15		⧅											
					44	21	86.9	34.3					
		◫			18	14		34.2					
20		⧅											
					40	20		40.0					
				Drilling terminated at a depth of 21 1/2 feet. No groundwater was encountered during drilling.									
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Boring: B2
Project: 511 Ohlone Parkway
Location:
Elevation:
Date: June 20, 2016
Method of Drilling: 6 inch solid stem truck mounted auger
Logged By: AP

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
	SM (FILL)	⧅		Gray silty SAND with gravel (FILL), grading into a 6" black FAT CLAY layer.	24	9		26.1					
	CL	◫		Brown-orange mottled LEAN CLAY, very stiff, slightly moist.	22	18		28.9					
5		⧅			39	14	97.3	20.3					
	SM	◫		Brown-orange silty SAND, medium dense, damp.	19	15		25.9					
10	CL	⧅		Brown-orange LEAN CLAY with sand, very stiff, moist.	36	18	84.5	36.9					
15		⧅			64	30		21.8					
				Drilling terminated at a depth of 16 1/2 feet. No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B3
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 20, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												Swell (psf)	
				6" Gray silty SAND over:									
	CH	⧅		Black FAT CLAY with sand, stiff, slightly moist (FILL).	38	19	102.5	20.9					
	(FILL	◫		Increasing sand.	21	17		17.4					
5													
	CH	⧅		Tan FAT CLAY, very stiff, moist.	40	15							
		◫			11	8		34.4			✓		
10													
	CH	⧅		Increase in fine-grained sand.	34	17	84.4	36.8				1,800	
15													
	SM	⧅		Tan silty SAND, medium dense, damp.	51	19		24.4					
				Boring terminated at a depth of 16 1/2 feet.									
				No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B4
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 20, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ⊠ Bulk Sample

◫ Terzaghi Split Spoon Sample ∇ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
	SM (FILL)	⧅		Gray silty SAND with gravel, medium dense, slightly damp (FILL).	28	10	93.3	27.8					
	CH (FILL	◫		Black FAT CLAY with sand, gravel (FILL).	17	13		23.2					
5													
	CH	⧅		Brown-orange mottled FAT CLAY with sand,	53	25	91.2	12.0					
		◫		very stiff, moist.	25	21		15.0					
10													
		⧅			35	17	85.1	36.0		4,700			
15		⧅											
				Hard.	79	36	94.2	21.0					
				Drilling terminated at a depth of 16 1/2 feet. No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: June 20, 2016
Logged By: AP

Boring: B5
Location:
Elevation:
Method of Drilling: 6 inch solid stem truck mounted auger

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												Swell (psf)	
	CH			Brown and black sandy FAT CLAY with trace gravel,									
	(FILL)	⧅		stiff, moist (FILL).	41	15	98.2	19.6					
	CH	◫		Orange-brown mottled FAT CLAY with sand,	16	13		35.0					
5				stiff, moist.									
		⧅			41	20	78.2	44.3				3,500	
		◫			21	17		37.6					
10													
		⧅			50/6"		97.3	25.2					
15													
20													
	CH	◫		Hard.	46	42					✓		
25													
30													
	SM	◫		Tan silty SAND, very dense, damp.	72	65		15.5					
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-8a

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B5 continued.
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 20, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
‖ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
40	SP-SM	‖		Tan poorly-graded SAND, very dense, damp.	80	73		8.6			✓		
				Drilling terminated at a depth of 41 1/2 feet. No groundwater encountered during drilling.									

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-8b

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: June 20, 2016
Logged By: AP

Boring: B6
Location:
Elevation:
Method of Drilling: 6 inch solid stem truck mounted auger

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
	CH	⧅		Dark brown-black FAT CLAY, stiff, moist.	44	21	102.7	22.0					
		◫			19	15		23.1					
			☒						103				
5													
	CL	⧅		Tan-olive brown-orange mottled LEAN CLAY, stiff, moist.	31	16	92.6	28.8					
		◫			23	19		29.0					
10													
		⧅		Increase in sand.	50/6"			15.8					
15													
	SP	◫		Poorly-graded SAND, medium dense, damp.	42	38		7.3					
20													
		◫		Coarser sand.	38	34		8.1					
25				▽									
30													
				Hole collapse, unable to obtain sample.									
				Drilling terminated at a depth of 31 1/2 feet. Groundwater encountered at 25 1/2 feet.									
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-9

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B7
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 20, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Atterberg Limits	
												L.L.	P.I.
	CH	⧅		1 foot of gray silty SAND (FILL).									
	(FILL)			Black FAT CLAY with sand, gravel, stiff (FILL).	31	16	105.9	19.3					
	CH	◫		Olive brown-tan FAT CLAY, very stiff, moist.	30	26		23.6					
5													
		⧅			49	23	82.7	37.2				84	57
		◫			27	23		33.4					
10													
		⧅			42	20		29.5					
15		⧅											
				Increase in sand.	60	28		19.6			✓		
				Drilling terminated at a depth of 16 1/2 feet.									
				No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: June 20, 2016
Logged By: AP

Boring: B8
Location:
Elevation:
Method of Drilling: 6 inch solid stem truck mounted auger

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
	SC (FILL)	⧅		Gray-black clayey SAND with gravel (FILL).	25	13		23.5					
	CH	◫		Olive-brown FAT CLAY with trace gravel, stiff, moist.	16	13		36.8					
5													
		⧅		Very stiff.	40	20	92.7	27.2		4,000			
		◫			26	22		27.6					
10		⧅											
	SC			Brown-orange mottled clayey SAND, very dense, damp.	70	32		14.7					
15		◫			30	26		19.6					
				Drilling terminated at a depth of 16 1/2 feet. No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B9
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 27, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

2" Ring Sample

2.5" Ring Sample

Bulk Sample

Terzaghi Split Spoon Sample

Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests Swell (psf)	
	SM (FILL)			Gray silty SAND with gravel (FILL).									
5	CH (FILL)			Sandy black FAT CLAY with gravel and debris (rubber inner tubes, plastic, tires).	19 18	10 14		20.2 26.8					
	CH		☒	Black FAT CLAY, stiff, moist.	22 17	12 14	100.5	23.3 24.1	41			1,200	
10	CH			Olive-brown/tan FAT CLAY with sand.	24	14	93.8	28.5		8,600			
15	CL			Gray-orange mottled sandy LEAN CLAY, stiff, moist.	30 24	18 25	94.5	24.6 29.5		3,300			
20 25 30 35				Drilling terminated at a depth of 18 feet. No groundwater encountered during drilling.									

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B10
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 27, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	
	SM (FILL)			Gray silty SAND with gravel (FILL).									
	CH (FILL)	⧅		Black FAT CLAY with gravel, glass fragments (FILL).	23	11		21.6					
		◫			16	12		21.5					
5													
	CH	⧅		Black sandy FAT CLAY with trace gravel.	20	11	95.3	20.3					
		◫			21	18		20.6					
			⊠										
10													
		⧅			31	17	111.7	17.7					
			⊠										
15													
		⧅		Brown FAT CLAY, stiff, moist.	27	16	104.7	22.6					
20													
		⧅		Olive-brown FAT CLAY, very stiff, moist.	30	25		27.4					
25													
	SM	◫		Brown-orange mottled silty SAND, dense, damp.	34			9.9					
				Drilling terminated at a depth of 26 1/2 feet. No groundwater encountered during drilling.									
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Boring: B11
Project: 511 Ohlone Parkway
Location:
Elevation:
Date: June 27, 2016
Method of Drilling: 6 inch solid stem truck mounted auger
Logged By: AP

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	CH	⧅		6 inches of gray silty SAND (FILL) over:									
	(FILL)			Black FAT CLAY, stiff, moist.	20	11	95.7	24.2		6,600			
		◫	☒		11	8		26.5					
5	CH	⧅		Olive brown-orange mottled FAT CLAY with sand,									
				trace gravel, stiff, moist.	24	13	95.2	25.2					
		◫			12	10		22.6			✓		
10		⧅											
					15	9		35.1					
15		⧅		2 inch gravel layer at 15 1/2 feet.									
				Very stiff.	30	18		37.8					
				Drilling terminated at a depth of 16 1/2 feet.									
				No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B12
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 27, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample

⧅ 2.5" Ring Sample

☒ Bulk Sample

‖ Terzaghi Split Spoon Sample

▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	
	CH	⧅		6 inches gray silty SAND with gravel (FILL) over:									
	(FILL)			Black FAT CLAY, stiff, slightly moist.	21	15	95.0	25.4					
	CH	‖		Olive-brown FAT CLAY with trace sand, gravel,	13	9		31.9					
	(FILL)			stiff, moist (FILL).									
5													
	CL	⧅		Orange-brown mottled LEAN CLAY with sand,	34	17	101.1	16.7			✓		
		‖		stiff, moist.	12	10		27.3					
			☒						0				
10		‖											
				Trace organics, firm, moist.	10	8		35.3					
15		‖											
				Very stiff.	27	29		18.1					
20		‖											
	CH			Olive-green-brown FAT CLAY, stiff, moist.	12	11		31.2					
25				Drilling terminated at a depth of 21 1/2 feet. No groundwater encountered during drilling.									
30													
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE B-15

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Boring: B13
Project: 511 Ohlone Parkway
Location:
Elevation:
Date: June 27, 2016
Method of Drilling: 6 inch solid stem truck mounted auger
Logged By: AP

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	SM (FILL)	⧅		Black-brown silty SAND with gravel, loose, slightly damp (FILL).	23	8		9.7					
	CH (FILL)	◫		Black FAT CLAY with trace gravel, sand, stiff (FILL).	18	14		23.1					
5													
	SC	⧅		Brown clayey SAND, loose, damp.	25	9		15.2					
		◫		Very loose.	4	3		22.8					
10													
	CH	⧅		Olive-brown FAT CLAY, stiff, moist.	23	13	82.2	40.6					
		◫			14	13		36.4					
15		◫											
				Very stiff.	18	18		33.7					
				Drilling terminated at a depth of 16 1/2 feet. No groundwater encountered during drilling.									
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	B14
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	June 27, 2016	Method of Drilling:	6 inch solid stem truck mounted auger
Logged By:	AP		

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ⊠ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	
	SC (FILL)	⧅		Brown-black clayey SAND with gravel, loose, slightly damp (FILL).	14	5		15.2					
	CH (FILL)	◫	⊠	Black FAT CLAY with coarse sand, stiff, moist (FILL).	14	10		26.5					
5	CL	⧅		Tan sandy LEAN CLAY, firm, moist.	12	7		25.2					
		◫			8	6		32.4					
10	CH	⧅		Olive-brown FAT CLAY, stiff, moist.	21	12		32.8					
15		⧅		Very stiff.	33	20	96.9	27.1					
20				Drilling terminated at a depth of 16 1/2 feet. No groundwater encountered during drilling.									
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway

Date: June 29, 2016
Logged By: AP

Boring: T1
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample　⧅ 2.5" Ring Sample　☒ Bulk Sample
◫ Terzaghi Split Spoon Sample　▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Very loose rubble: large concrete blocks, asphalt blocks, gravel, soil (FILL).									
5													
	FILL			Tires, soil (FILL).									
10													
	CH			Black-tan FAT CLAY (native).									
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: June 29, 2016
Logged By: AP

Boring: T2
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Very loose rubble: concrete, wood, metal rims, plastic, soil (FILL).									
5													
10	CH			Black-tan FAT CLAY (native).									
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP

Boring: T3
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	Other Tests
	FILL			Soil, vegetation, rubber hoses, metal fenders, concrete rubble (debris).									
	CH			Black-tan FAT CLAY (native).									
5													
10													
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP

Boring: T4
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Soil, vegetation, metal debris, wood planks, gravel (debris).									
	FILL			Brown clayey SAND, loose, some gravel.									
5	CH			Black-tan FAT CLAY (native).									
10													
15													
20													
25													
30													
35													

FIGURE
B-21

LOG OF EXPLORATORY BORING

Project No.:	16-135-SC	Boring:	T5
Project:	511 Ohlone Parkway	Location:	
		Elevation:	
Date:	August 10, 2016	Method of Drilling:	12 inch backhoe bucket
Logged By:	AP		

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Loose soil, vegetation, gravel, tires, metal debris, concrete rubble, fabric, rope, rubber inner tubes (debris).									
5													
10													
	CH			Olive-brown FAT CLAY (native).									
15													
20													
25													
30													
35													

Debris from trench.

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Boring: T6
Project: 511 Ohlone Parkway
Location: Below T2, observation of contact
Elevation:
Date: August 10, 2016
Method of Drilling: 12 inch backhoe bucket
Logged By: AP

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Loose soil, vegetation, gravel, tires, metal debris, concrete rubble, fabric, rope, rubber inner tubes (debris).									
5													
	CH			Black-brown FAT CLAY (native).									
10													
15													
20													
25													
30													
35													

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE
B-23

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP
Boring: T7
Location: Below T5, observation of contact
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
▯ Terzaghi Split Spoon Sample
∇ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	
	FILL			Loose soil, vegetation, gravel, tires, metal debris, concrete rubble, fabric, rope, rubber inner tubes (debris).									
5													
10													
15													
	CH			Black-brown FAT CLAY (native).									
20													
25													
30													
35													

Debris from trench.

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP

Boring: T8
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	FILL			Loose soil, vegetation, gravel, concrete rubble (FILL).									
	CH			Black-brown FAT CLAY (native).									
5													
10													
15													
20													
25													
30													
35													

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP
Boring: T9
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	CH			≤ 6" loose sand, gravel, debris (FILL) over stiff black FAT CLAY (native).									
5													
10													
15													
20													
25													
30													
35													

FIGURE B-26

LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Project: 511 Ohlone Parkway
Date: August 10, 2016
Logged By: AP

Boring: T10
Location:
Elevation:
Method of Drilling: 12 inch backhoe bucket

◩ 2" Ring Sample
◪ 2.5" Ring Sample
☒ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests R-Value	Other Tests
	FILL			Loose soil, vegetation, gravel, concrete rubble, asphalt rubble, rubber innertubes, corrugated pipe (debris).									
5	FILL			Loose silty SAND with gravel.									
10				Trenching terminated at a depth of approximately 8 feet.									
15 20 25 30 35				 Debris from trench.									

BUTANO GEOTECHNICAL ENGINEERING, INC.

FIGURE B-27

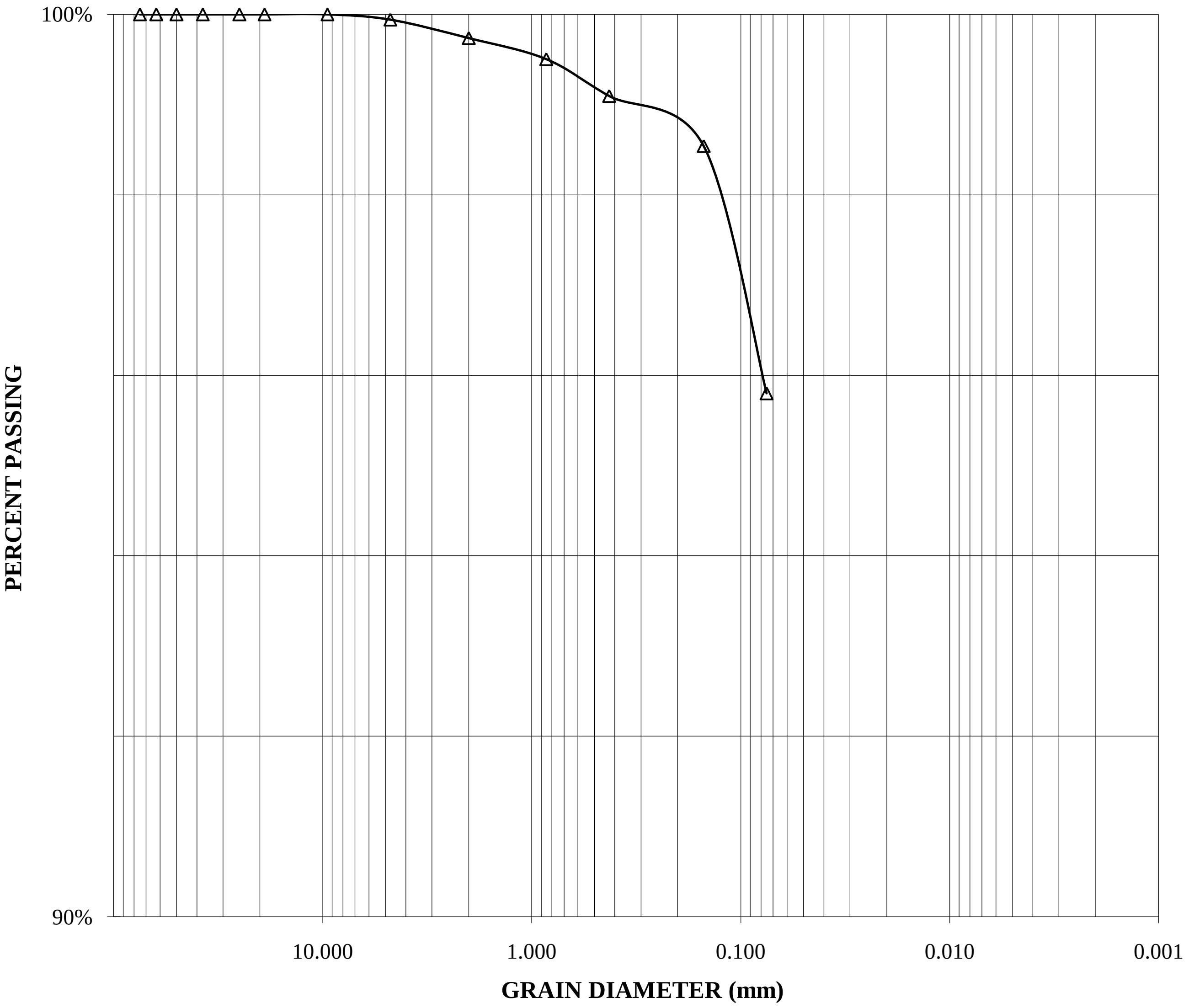
LOG OF EXPLORATORY BORING

Project No.: 16-135-SC
Boring: T11
Project: 511 Ohlone Parkway
Location:
Elevation:
Date: August 10, 2016
Method of Drilling: 12 inch backhoe bucket
Logged By: AP

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

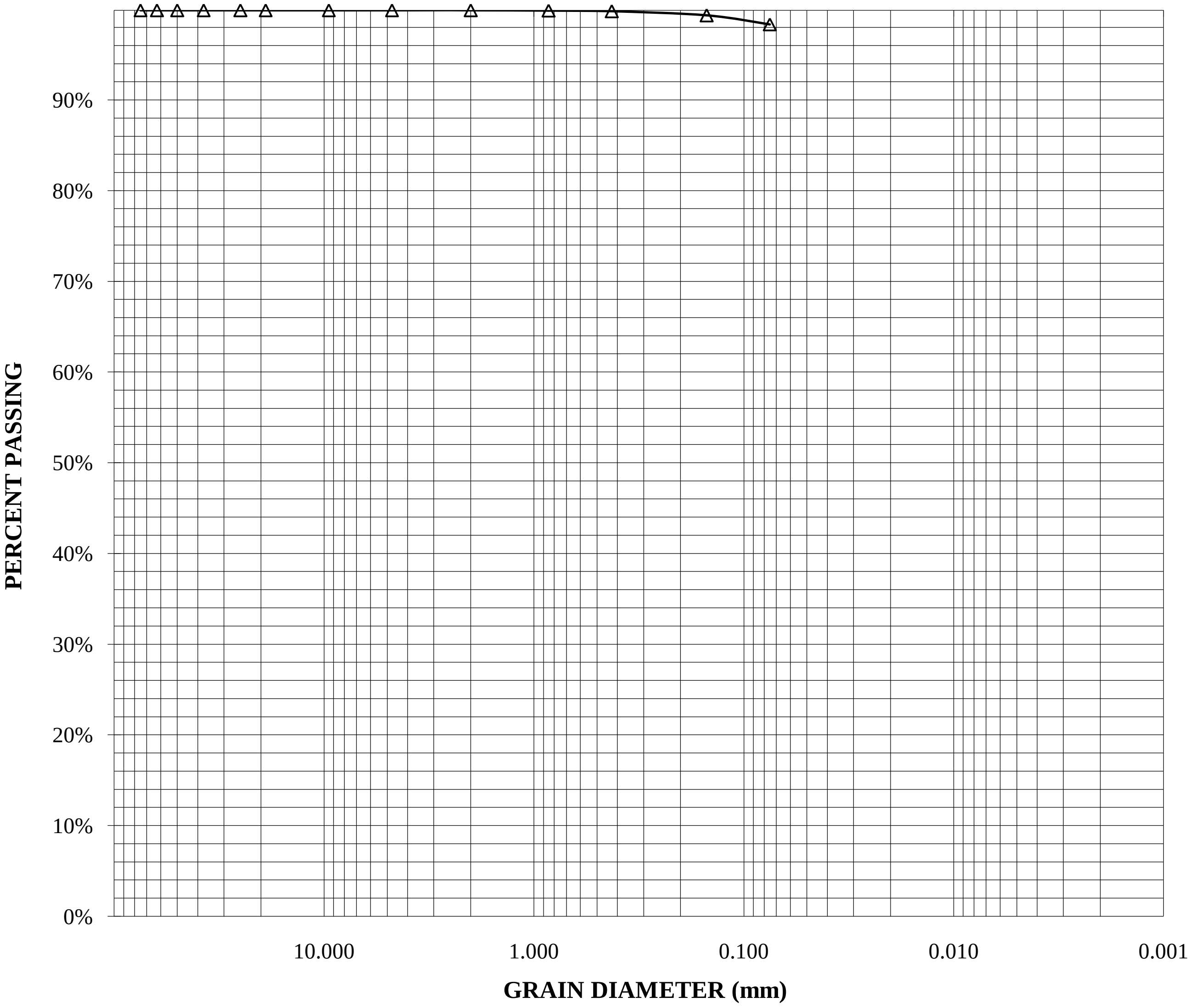
Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Expansion Index	Unconfined Comp. (psf)	Particle Size	Other Tests	
												R-Value	
	CL/ CH			≤ 6" loose sand, gravel, debris (FILL) over brown sandy LEAN CLAY with organics (native) over black FAT CLAY (native).									
5													
10													
15													
20													
25													
30													
35													

BORING:	B1 Bulk	PERCENT PASSING No. 4	PERCENT PASSING No. 200
DEPTH (ft):	6.0		
SOIL TYPE (USCS):	CL	99.9%	95.8%



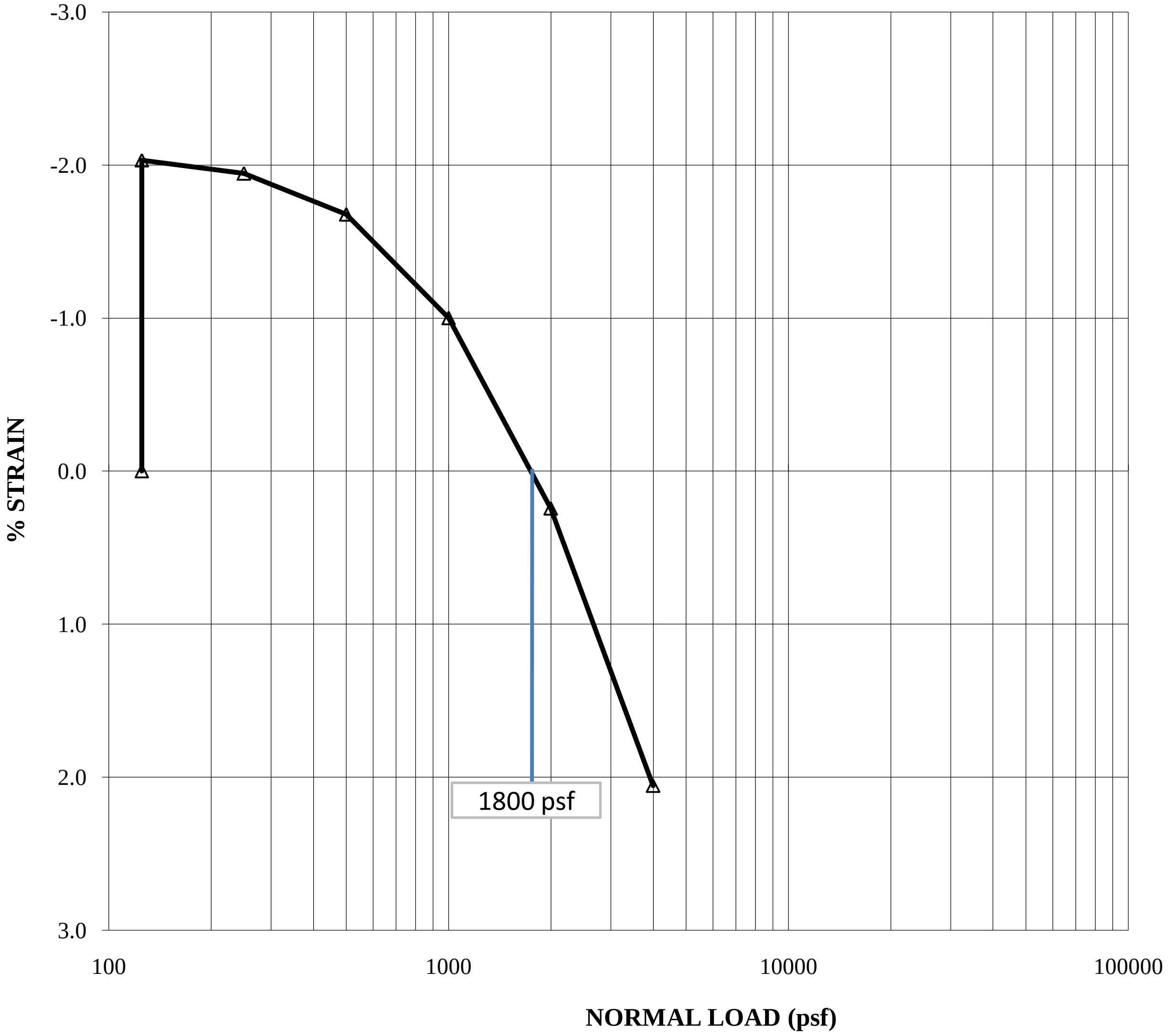
BUTANO GEOTECHNICAL ENGINEERING, INC.	**GRAIN SIZE DISTRIBUTION** 511 Ohlone Parkway	FIGURE C-1

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B3-4		
DEPTH (ft):	6.5		
SOIL TYPE (USCS):	CL	100.0%	98.4%



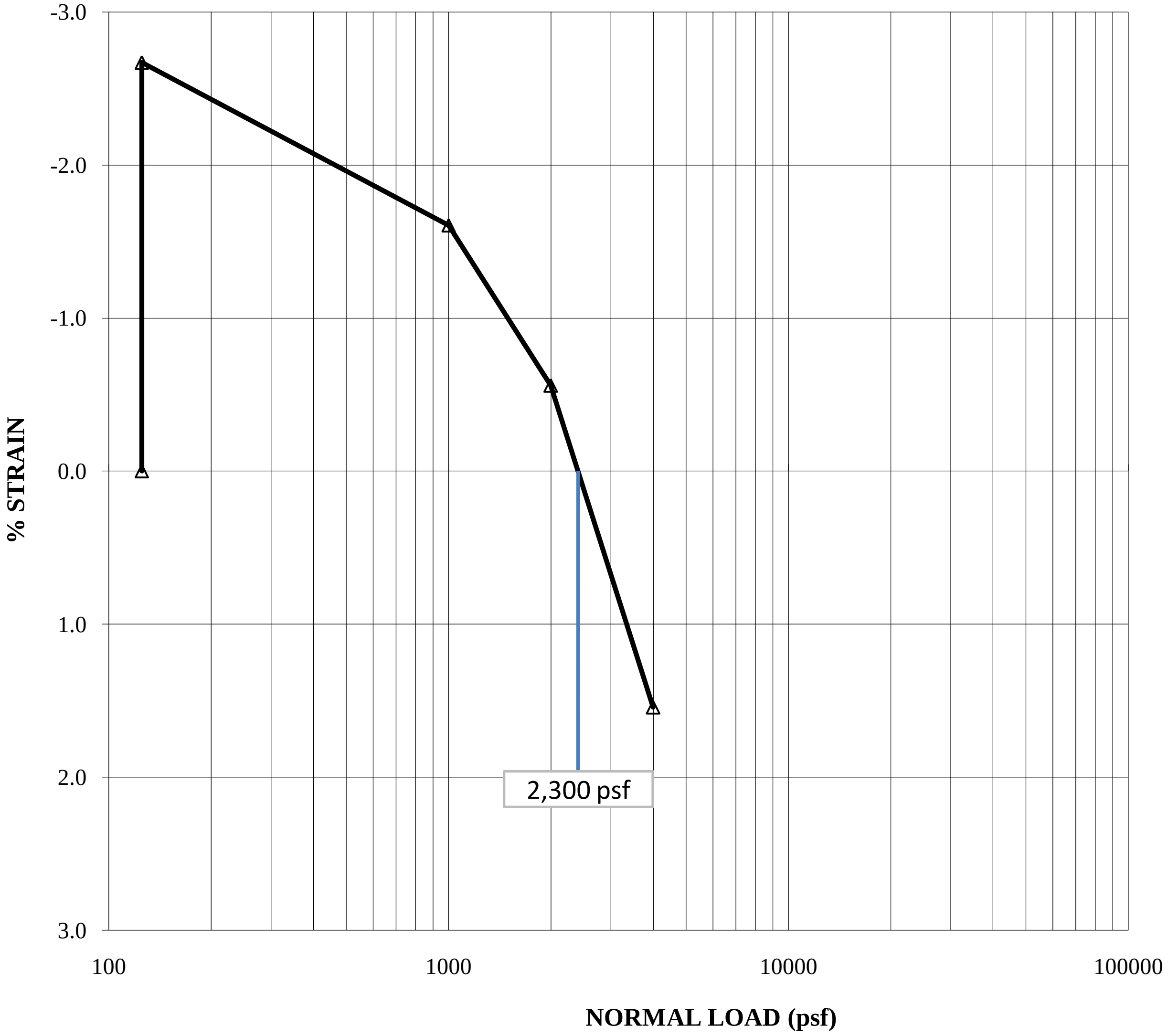
BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-2

BORING:	B3-5		
DEPTH (ft):	10.0		
SOIL TYPE (USCS):	CH	FIELD MOISTURE:	36.9%
		FINAL MOISTURE:	39.0%



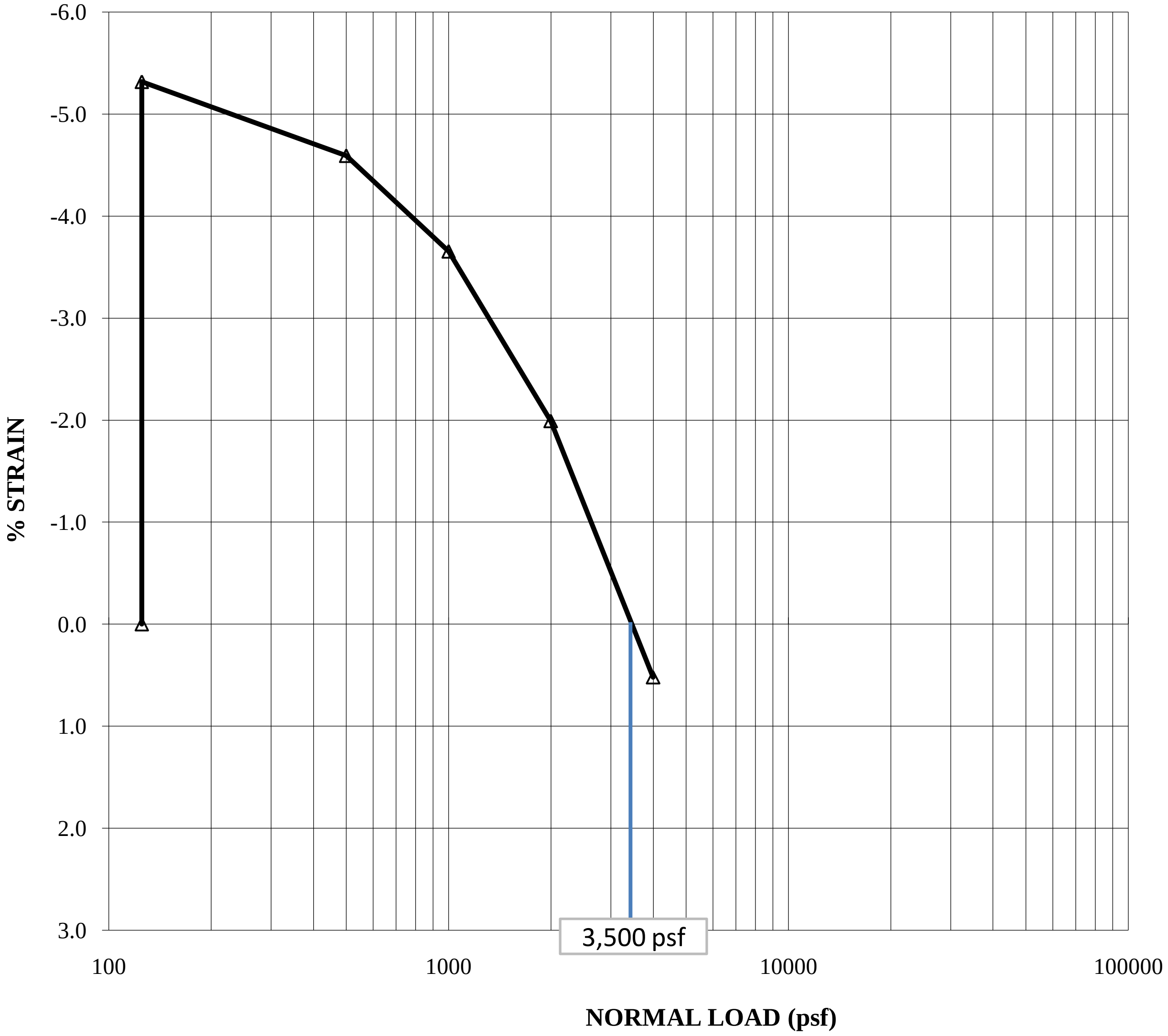
BUTANO GEOTECHNICAL ENGINEERING, INC.	**SWELL TEST RESULTS** 511 Ohlone Parkway	FIGURE C-10

BORING:	B1-1		
DEPTH (ft):	2.5		
SOIL TYPE (USCS):	CH	FIELD MOISTURE:	26.3%
		FINAL MOISTURE:	28.3%



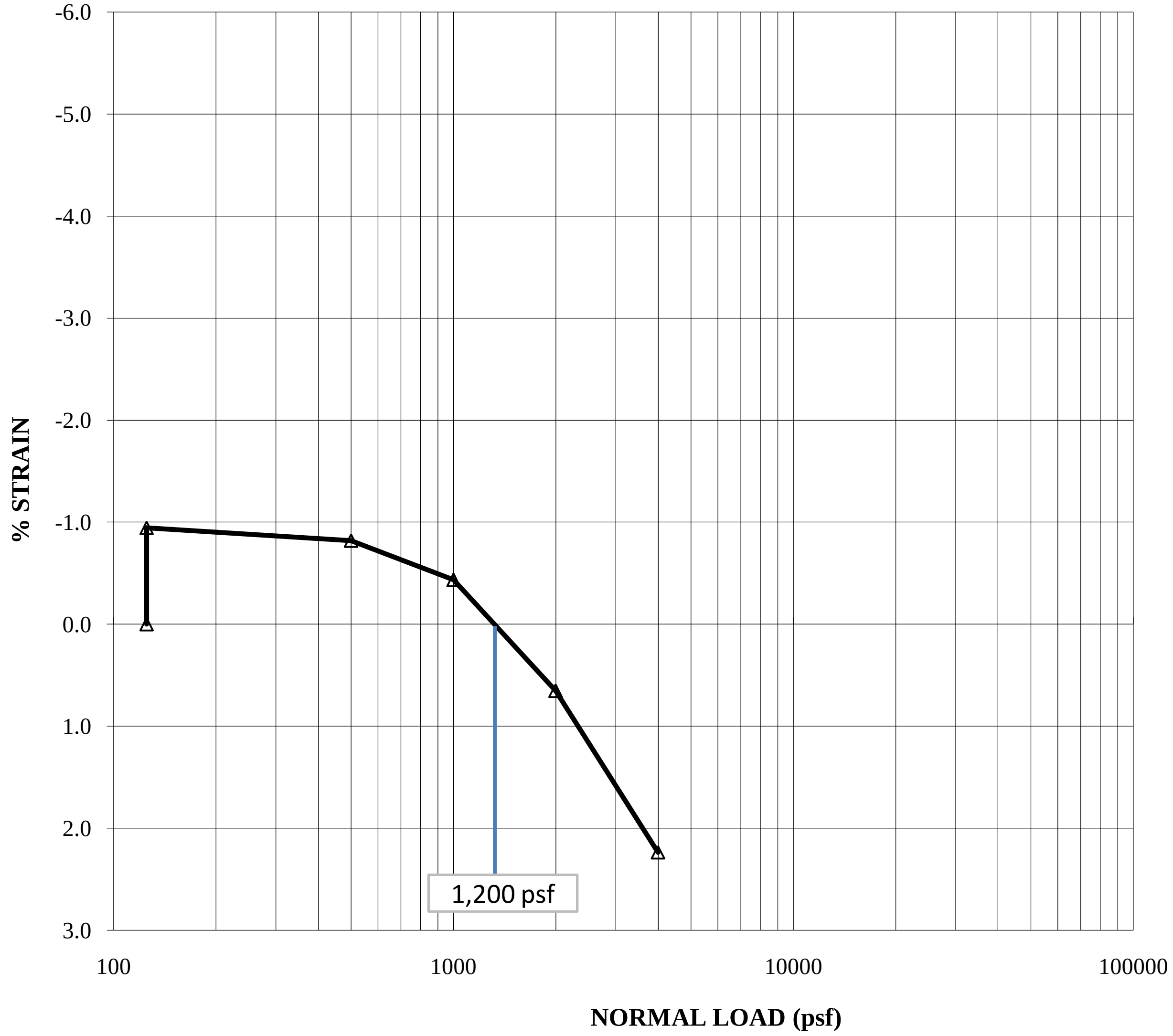
BUTANO GEOTECHNICAL ENGINEERING, INC.	**SWELL TEST RESULTS** 511 Ohlone Parkway	FIGURE C-11

BORING:	B5-3		
DEPTH (ft):	5.0		
SOIL TYPE (USCS):	CH	FIELD MOISTURE:	36.5%
		FINAL MOISTURE:	69.3%



BUTANO	**SWELL TEST RESULTS**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-12

BORING:	B9-3		
DEPTH (ft):	5.0		
SOIL TYPE (USCS):	CH	FIELD MOISTURE:	21.2%
		FINAL MOISTURE:	23.9%



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GEOTECHNICAL ENGINEERING, INC.

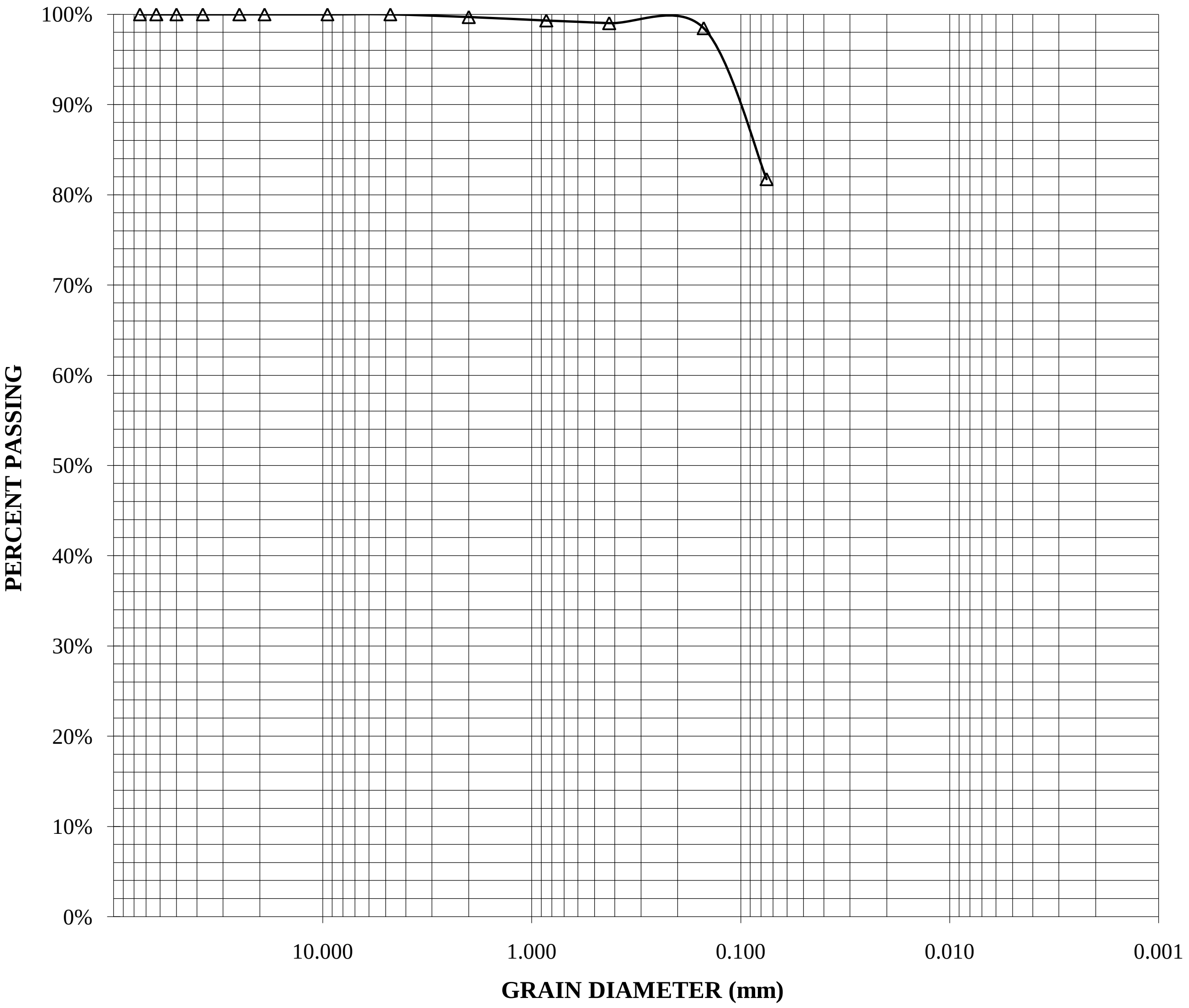
SWELL TEST RESULTS

511 Ohlone Parkway

FIGURE

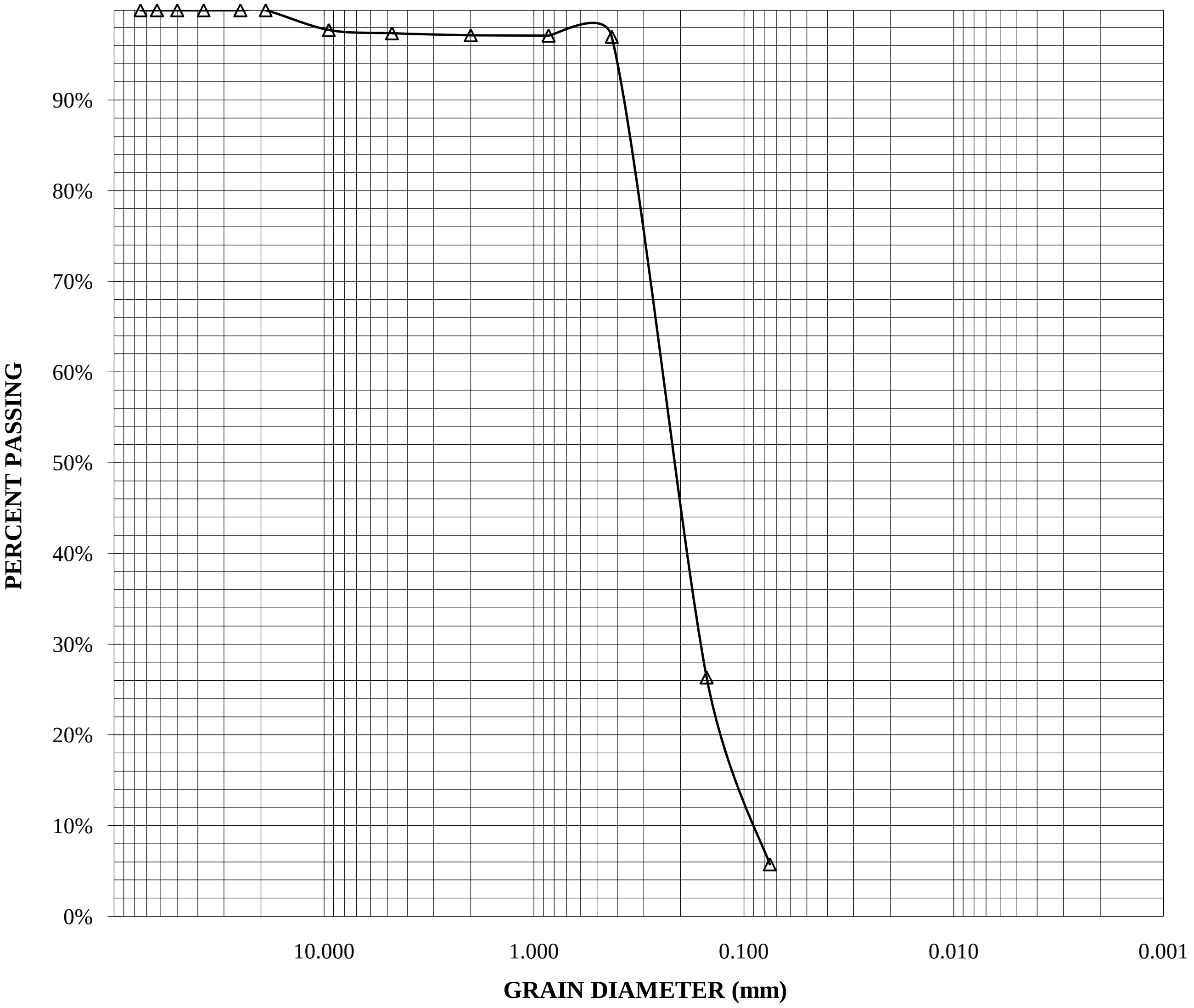
C-13

BORING:	B5-6	PERCENT PASSING No. 4	PERCENT PASSING No. 200
DEPTH (ft):	20.0		
SOIL TYPE (USCS):	CL	100.0%	81.7%



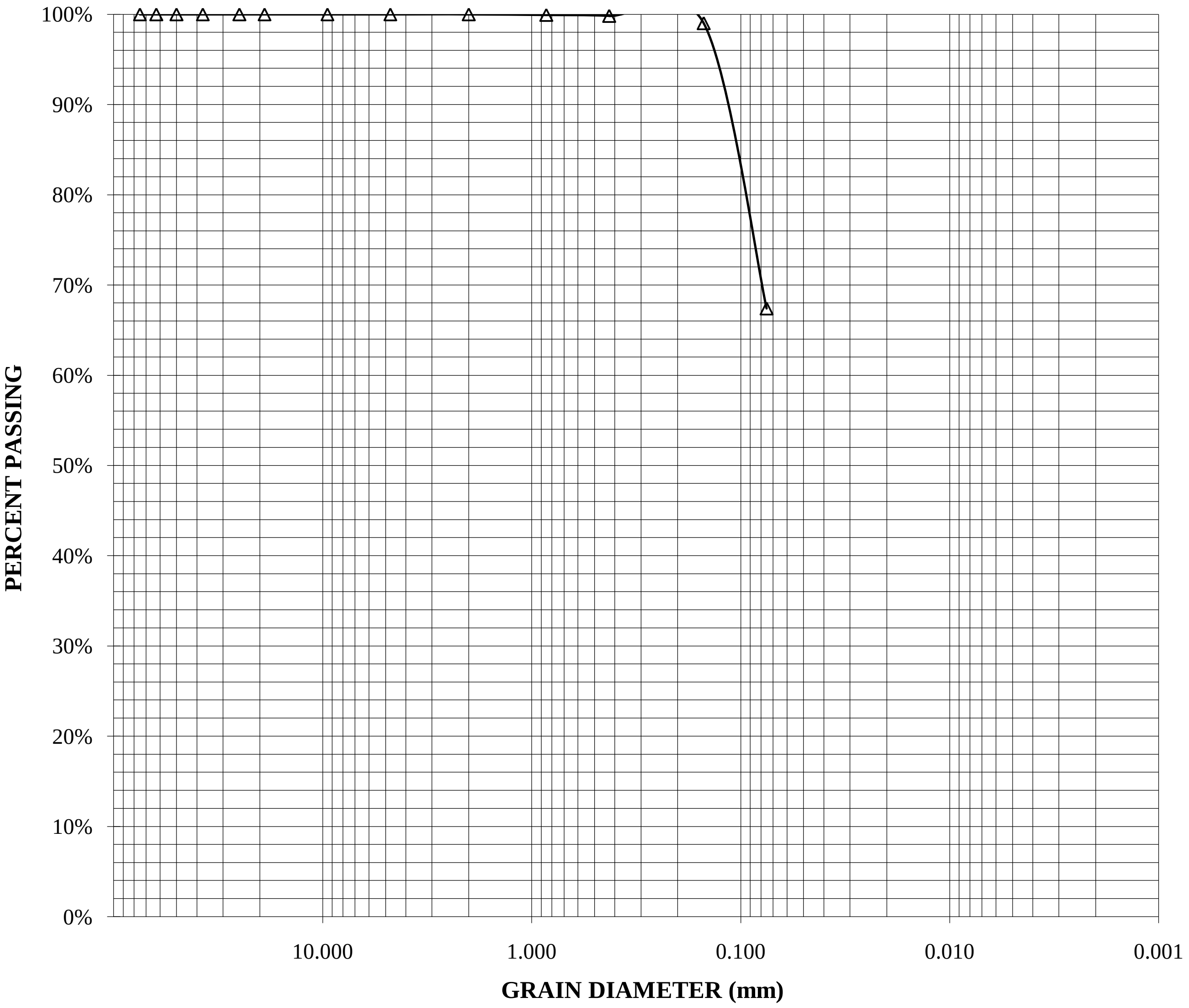
BUTANO GEOTECHNICAL ENGINEERING, INC.	**GRAIN SIZE DISTRIBUTION** 511 Ohlone Parkway	FIGURE C-3

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B5-8		
DEPTH (ft):	40		
SOIL TYPE (USCS):	SP-SM	97.5%	5.8%



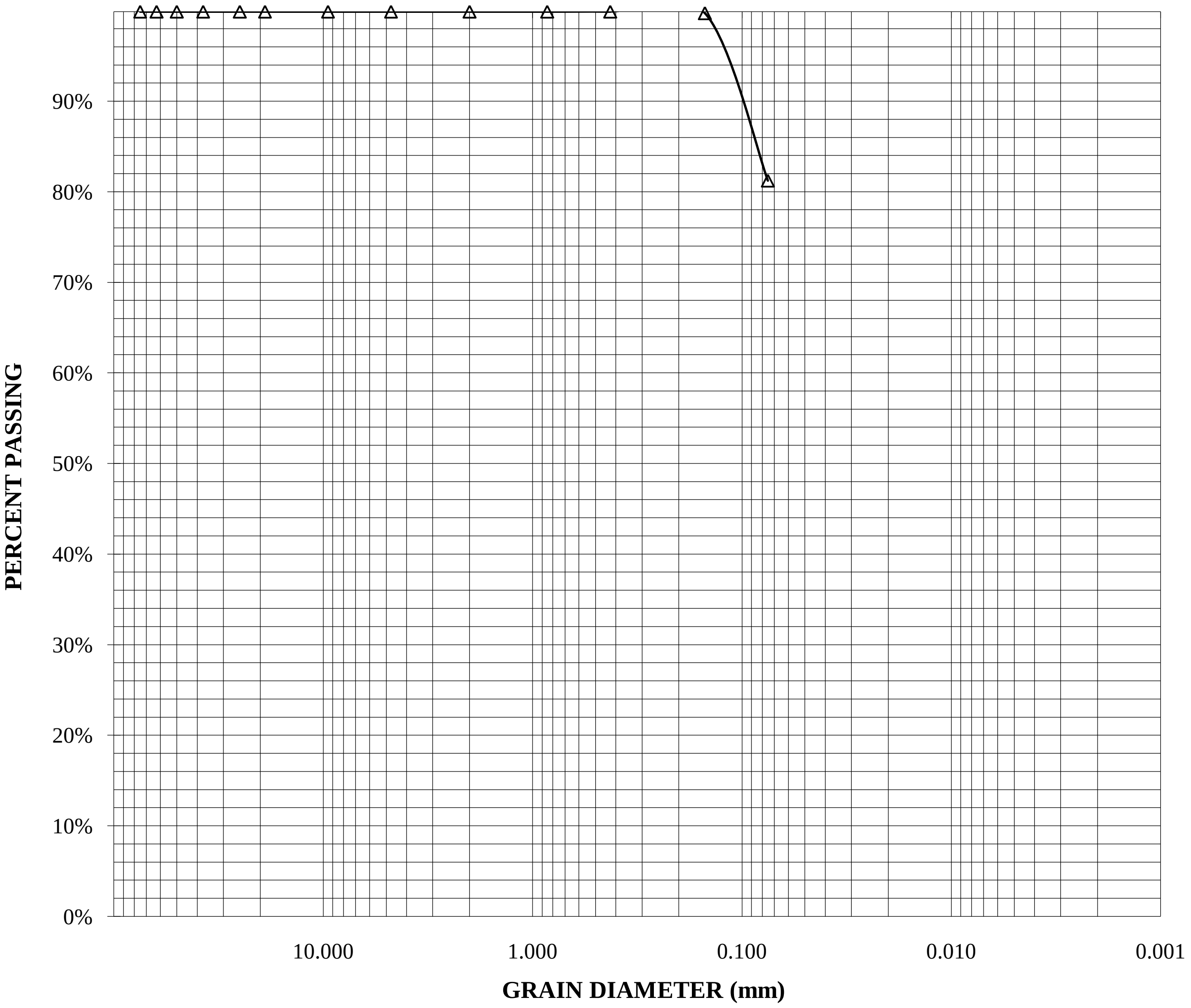
BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-4

BORING:	B7-8	PERCENT	PERCENT
DEPTH (ft):	15.0	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	CL	100.0%	67.4%



BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-5

BORING:	B12-3	PERCENT	PERCENT
DEPTH (ft):	5	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	CH	100.0%	81.3%



BUTANO

GEOTECHNICAL ENGINEERING, INC.

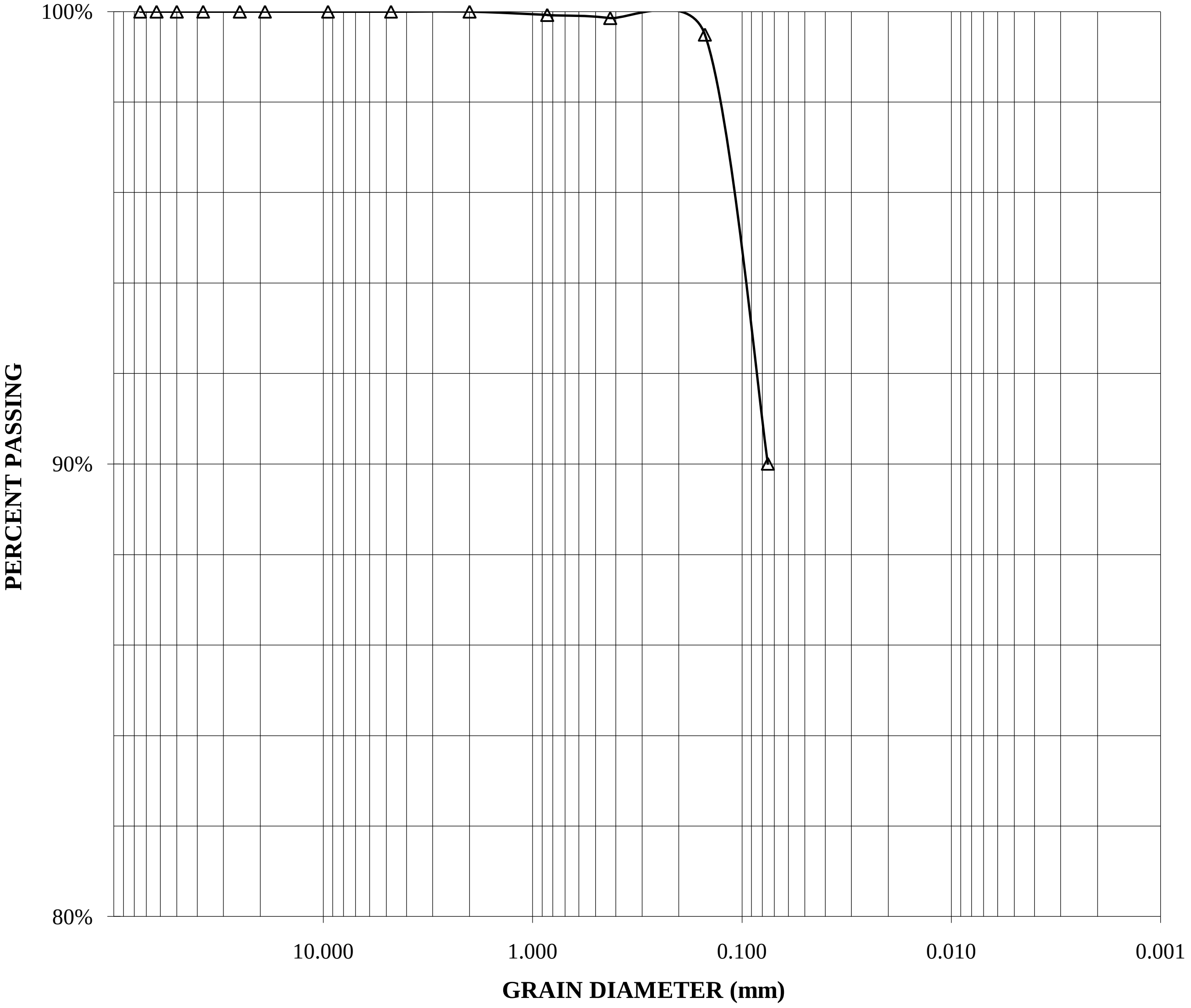
GRAIN SIZE DISTRIBUTION

511 Ohlone Parkway

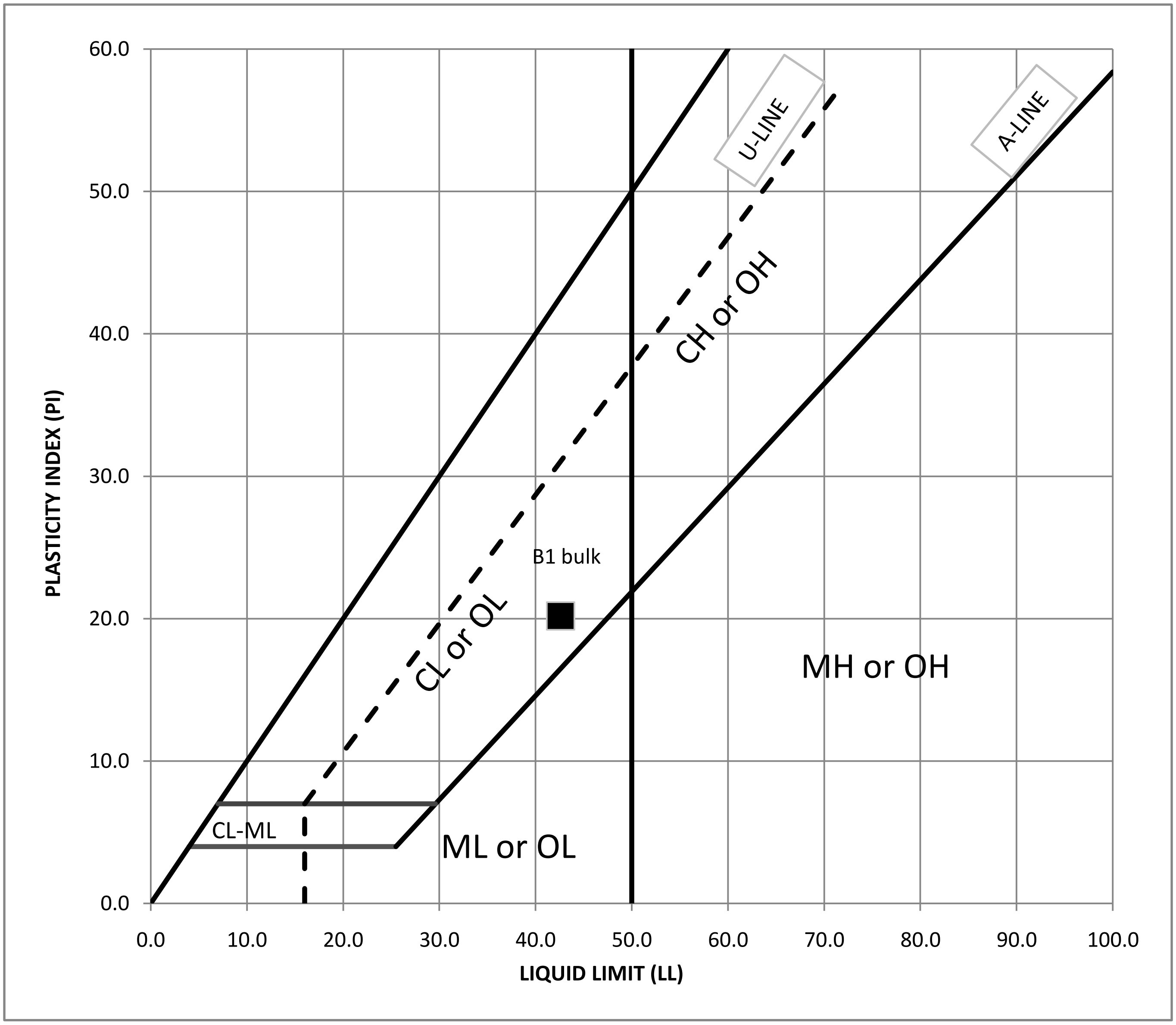
FIGURE

C-6

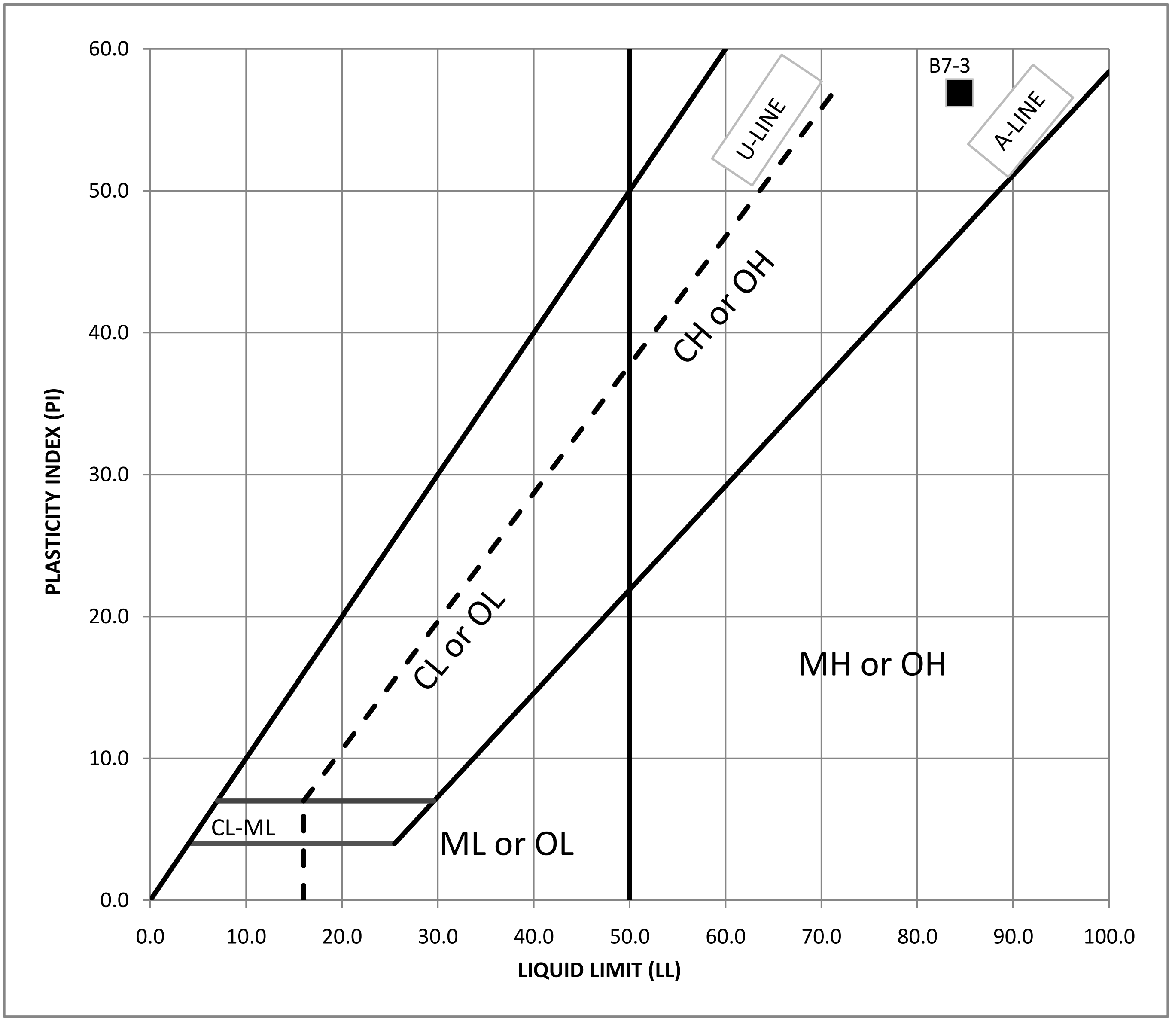
BORING:	B11-4	PERCENT	PERCENT
DEPTH (ft):	6.5	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	CL	100.0%	90.0%



BUTANO	**GRAIN SIZE DISTRIBUTION**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-7



BUTANO	**ATTERBERG LIMITS**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-8



BUTANO	**ATTERBERG LIMITS**	FIGURE
GEOTECHNICAL ENGINEERING, INC.	511 Ohlone Parkway	C-9

APPENDIX B:
SLOPE STABILITY ANALYSES RESULTS

Slope Stability Analysis

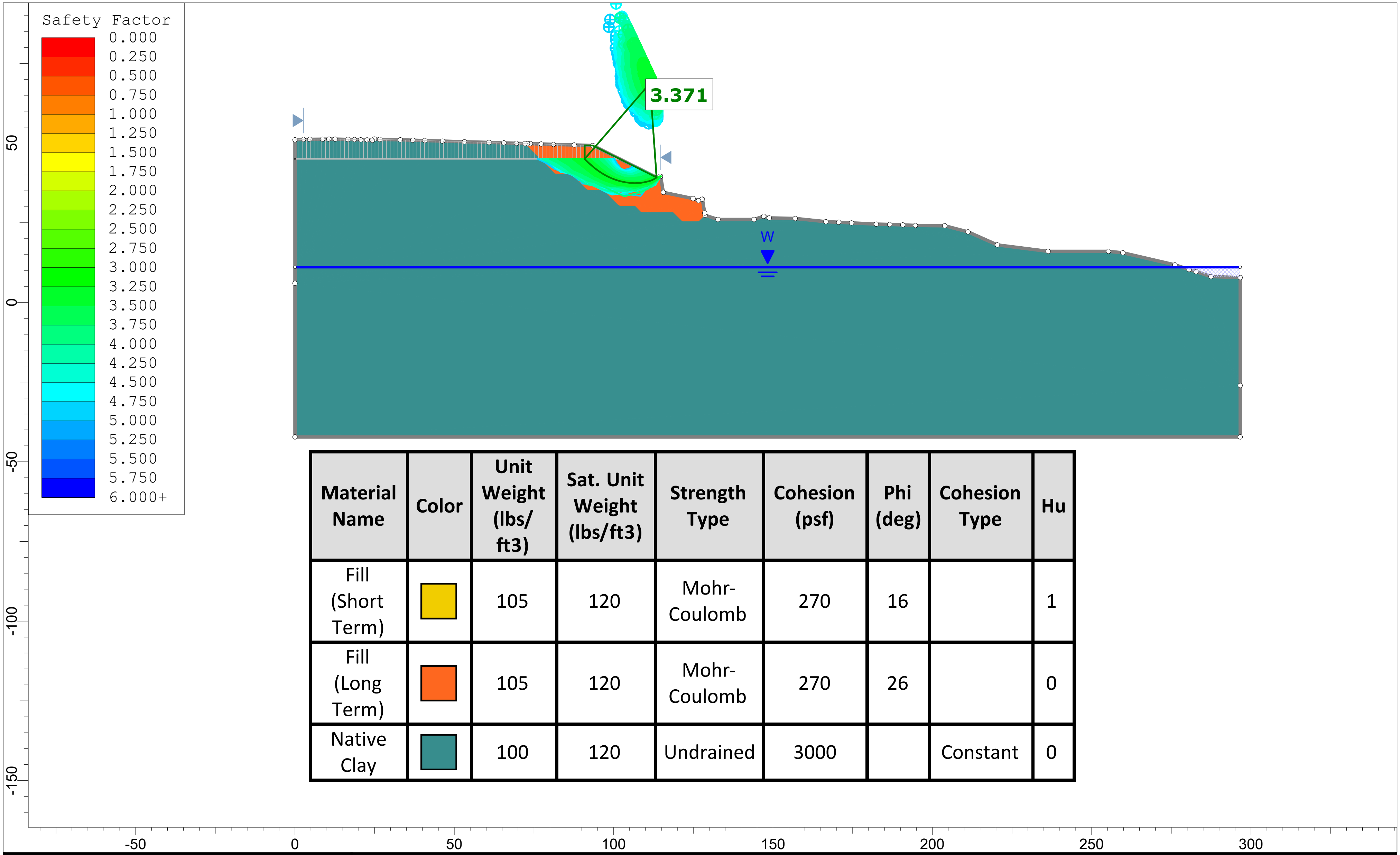
We analyzed various cross sections throughout the planned project site as shown on Figures 1 through 7 of this section. Two of the five sections analyzed, the MSE Wall Section and Section B, consisted of placing significant portions of fill on the existing grades. The remaining sections either involved cutting soil away from the slopes or adding minor amounts of fill. Therefore, we performed slope stability analyses on the MSE Sections and Section B utilizing the slope stability software SLIDE developed by Rocscience. Section B also includes a 2:1 (horizontal:vertical) above the five foot terraced retaining walls. Therefore, 2 stability analyses were performed on this section, a global analysis and an analysis focused on the 2:1 slope.

Slope stability analyses results indicate a factor of safety against soil movement. Under static conditions a factor of Safety above 1.5 is considered appropriate. Under seismic conditions, a factor of safety less than 1.0 indicates some movement may be observed during a strong seismic event. We utilized the procedures outlined by Bray and Travasarou, 2007 to determine the amount of deformation. The results of our slope stability and slope deformation analyses are presented in the following pages and summarized below:

	Static F.S.	Seismic F.S.	Seismic Deformation[1]
Section B (Global)	2.00	0.75	~4 – 8-in
Section B (2:1)	3.37	1.30	0.0-in (F.S. > 1.0)
Section MSE Wall	1.75	0.66	~6 – 12-in

Notes:

1. Predicted deformations are distributed throughout the landslide mass.



Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Short Term)	🟨	105	120	Mohr-Coulomb	270	16		1
Fill (Long Term)	🟧	105	120	Mohr-Coulomb	270	26		0
Native Clay	🟦	100	120	Undrained	3000		Constant	0

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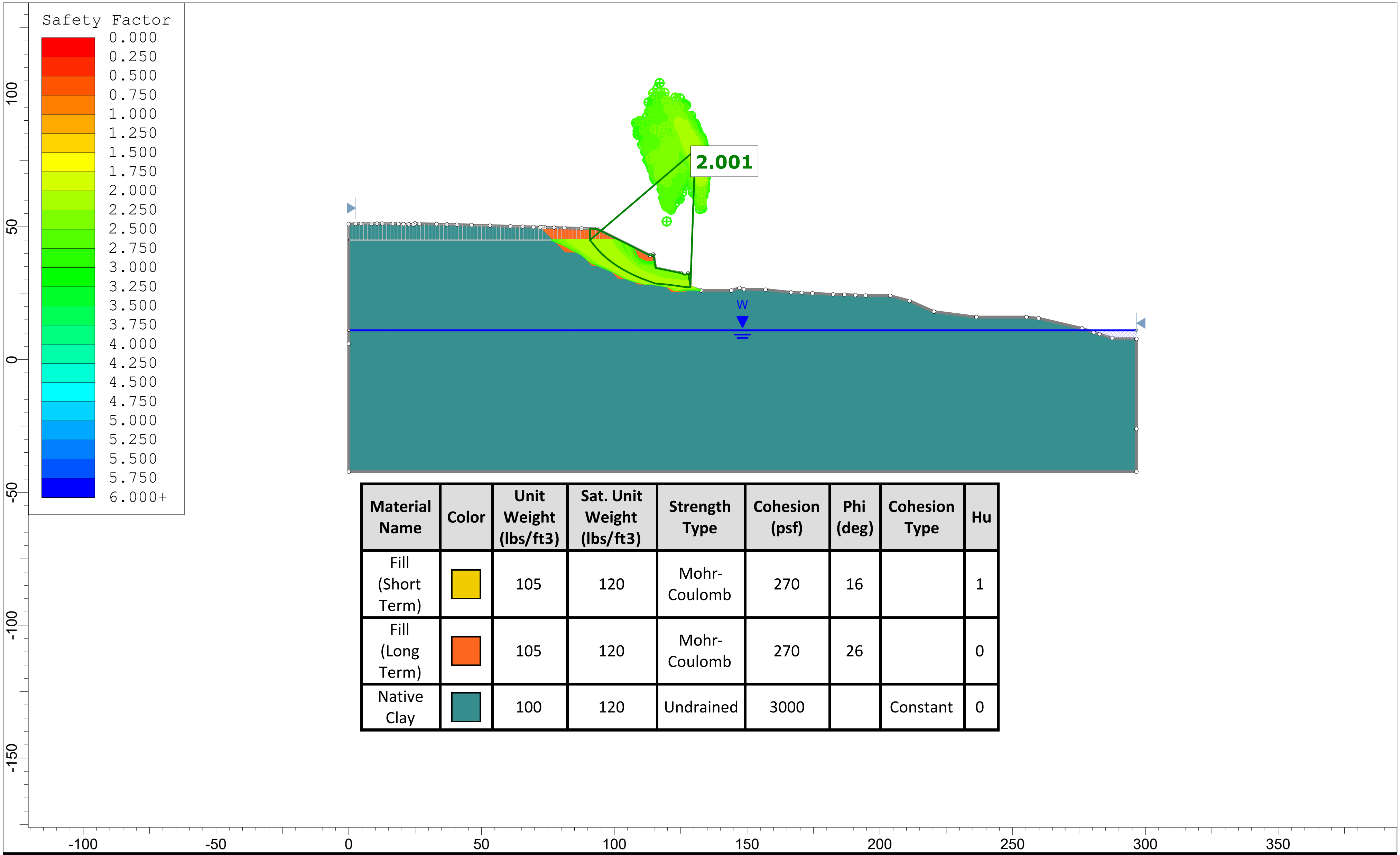
SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	SectionBB (2to1).slmd



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Short Term)	🟨	105	120	Mohr-Coulomb	270	16		1
Fill (Long Term)	🟧	105	120	Mohr-Coulomb	270	26		0
Native Clay	🟦	100	120	Undrained	3000		Constant	0

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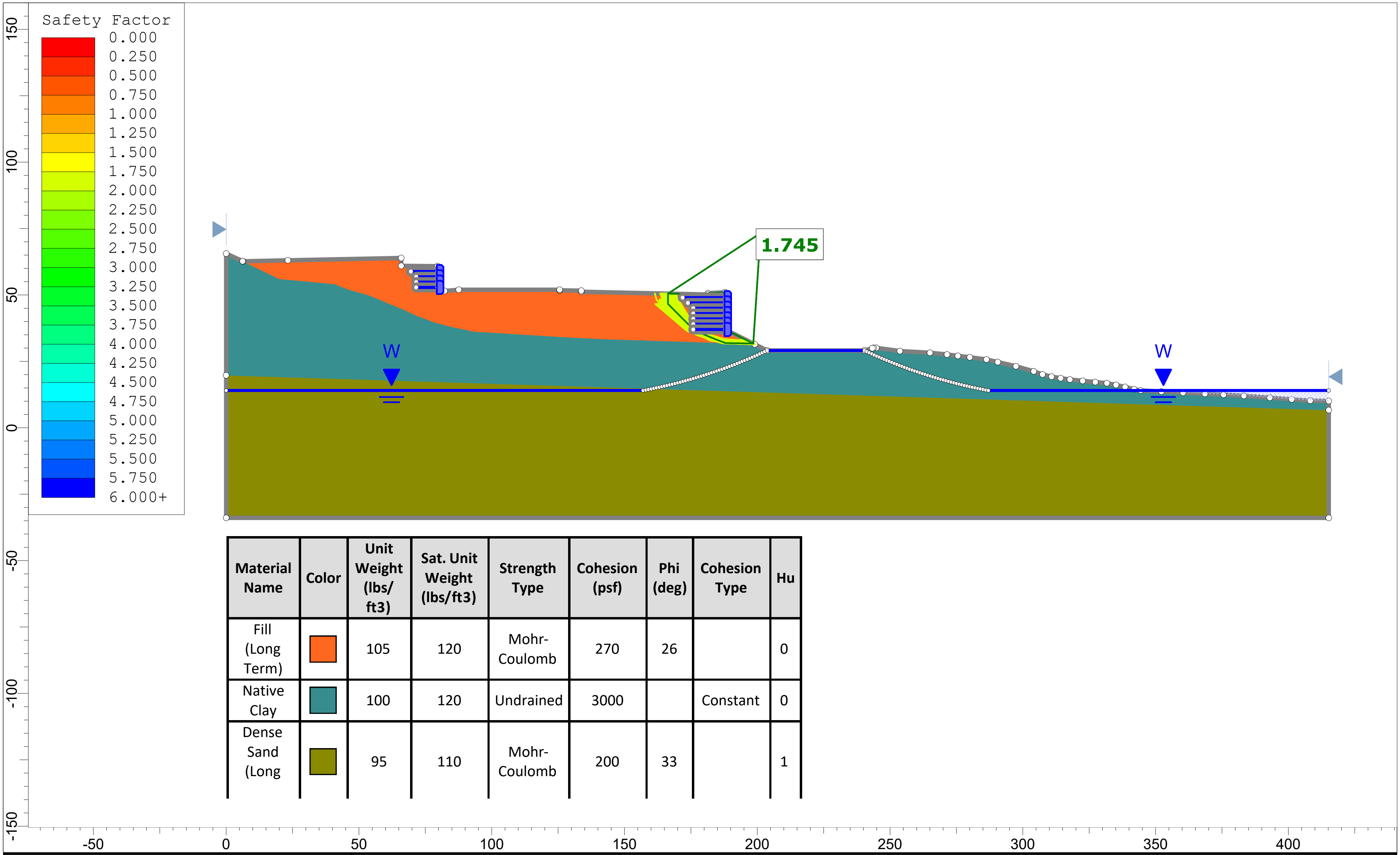
SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	SectionBB (2to1).slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Long Term)	🟧	105	120	Mohr-Coulomb	270	26		0
Native Clay	🟦	100	120	Undrained	3000		Constant	0
Dense Sand (Long	🟫	95	110	Mohr-Coulomb	200	33		1

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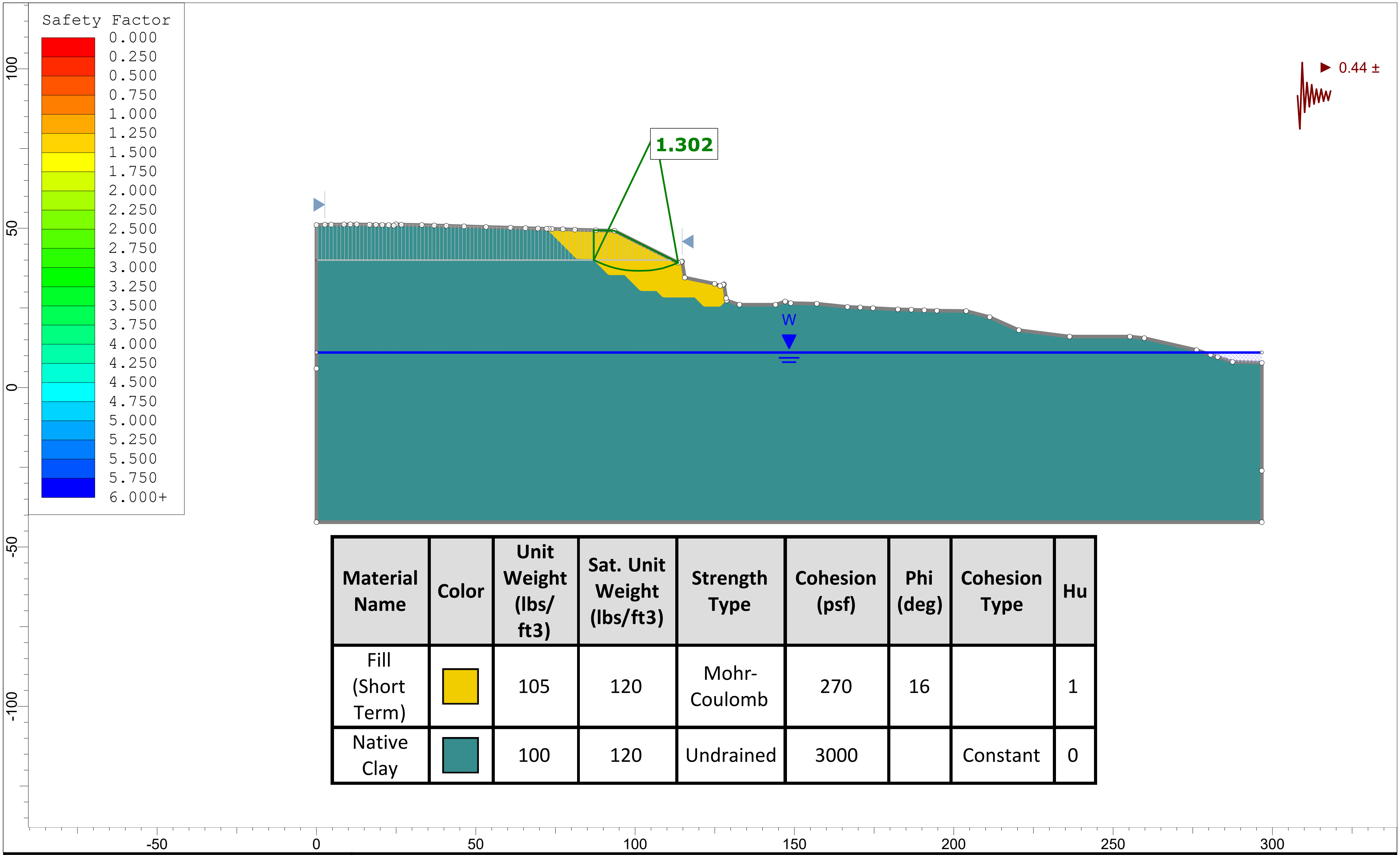
SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	Section MSE.slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Short Term)	🟨	105	120	Mohr-Coulomb	270	16		1
Native Clay	🟦	100	120	Undrained	3000		Constant	0

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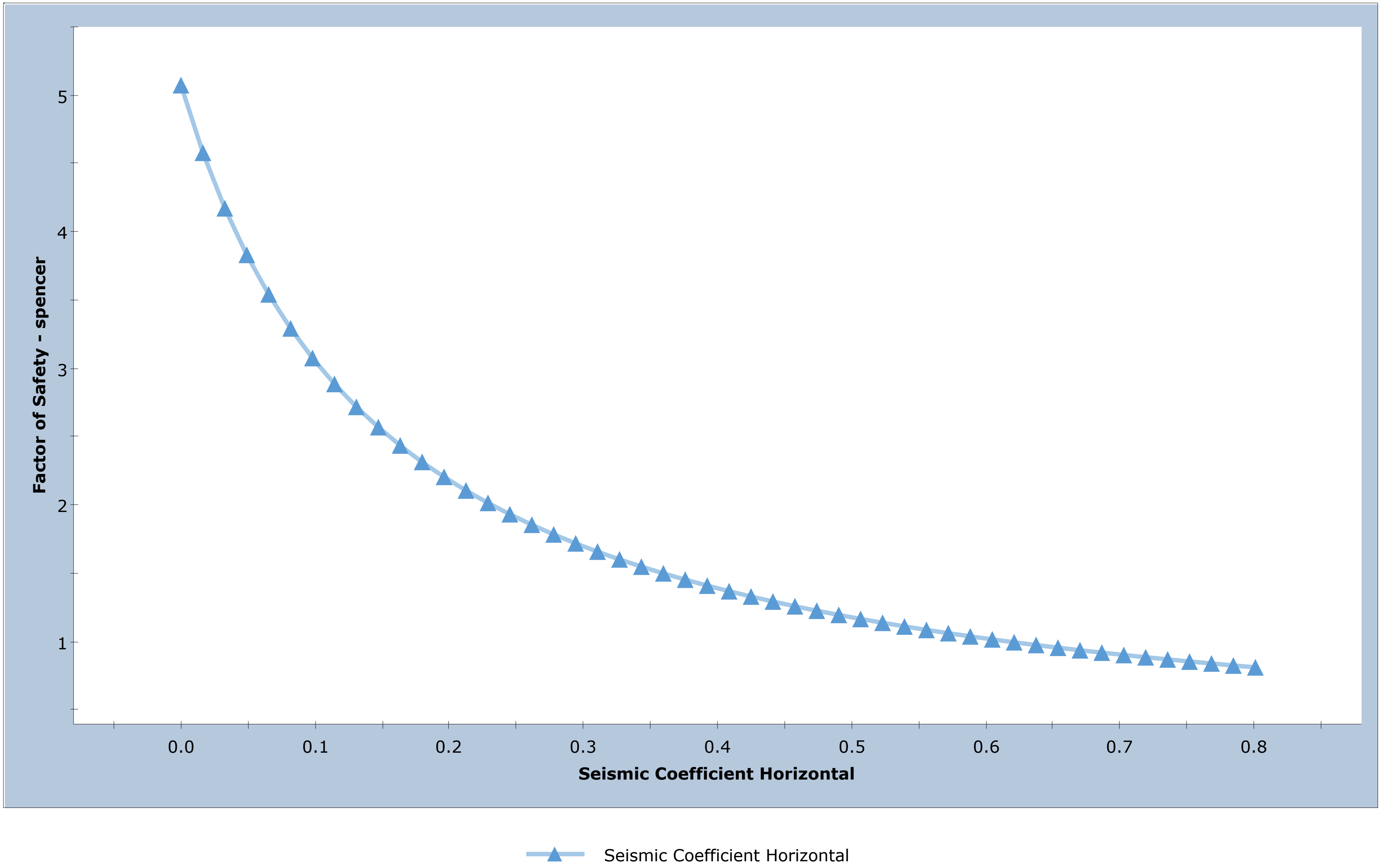
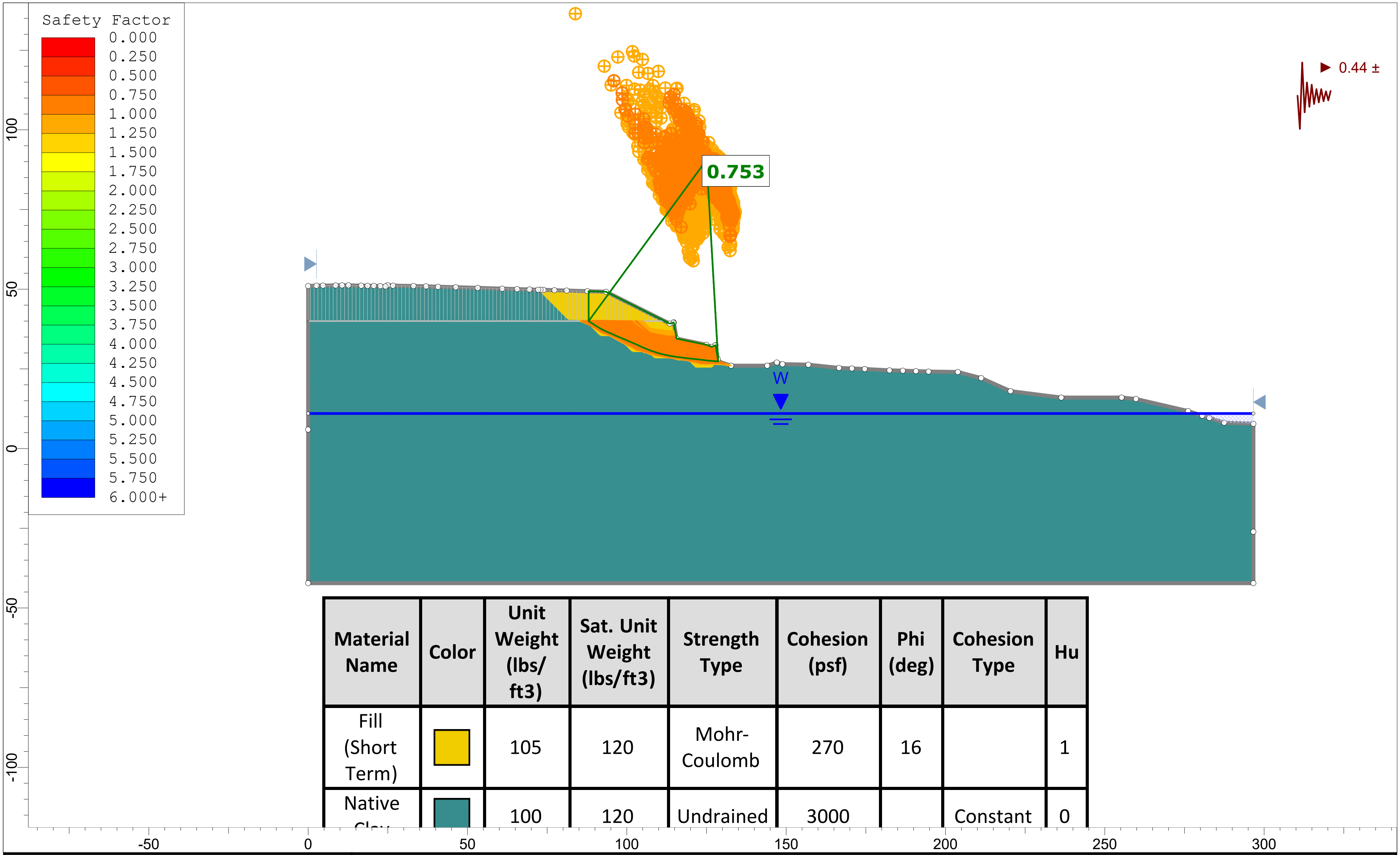
SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	SectionBB (2to1).slmd



rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Group 1	*Scenario* Master Scenario	
	Drawn By	*Company*	
SLIDEINTERPRET 9.017	*Date* 3/2/2021, 3:56:53 PM	*File Name* SectionBB (2to1).slmd	

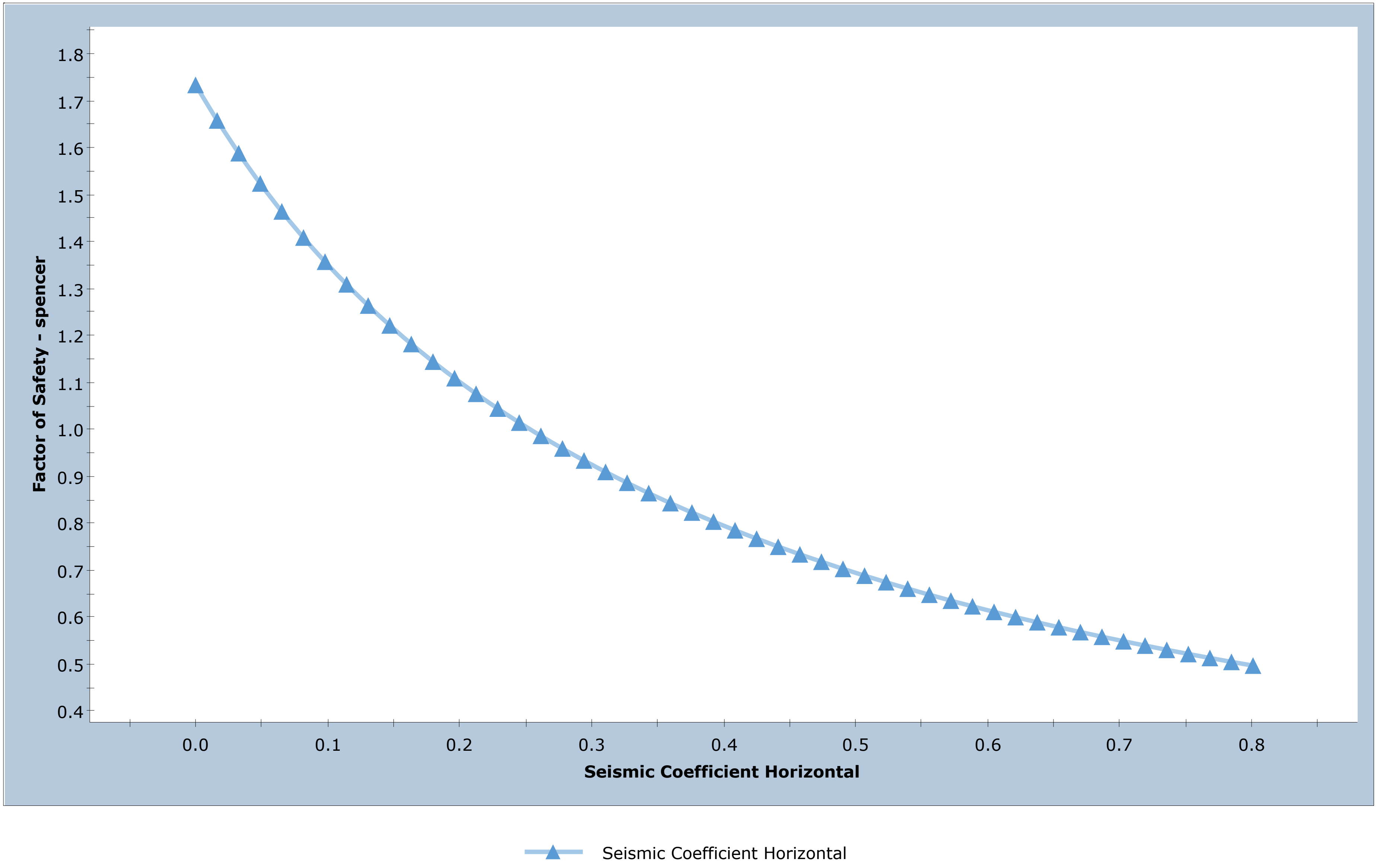


Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Short Term)	🟨	105	120	Mohr-Coulomb	270	16		1
Native	🟦	100	120	Undrained	3000		Constant	0

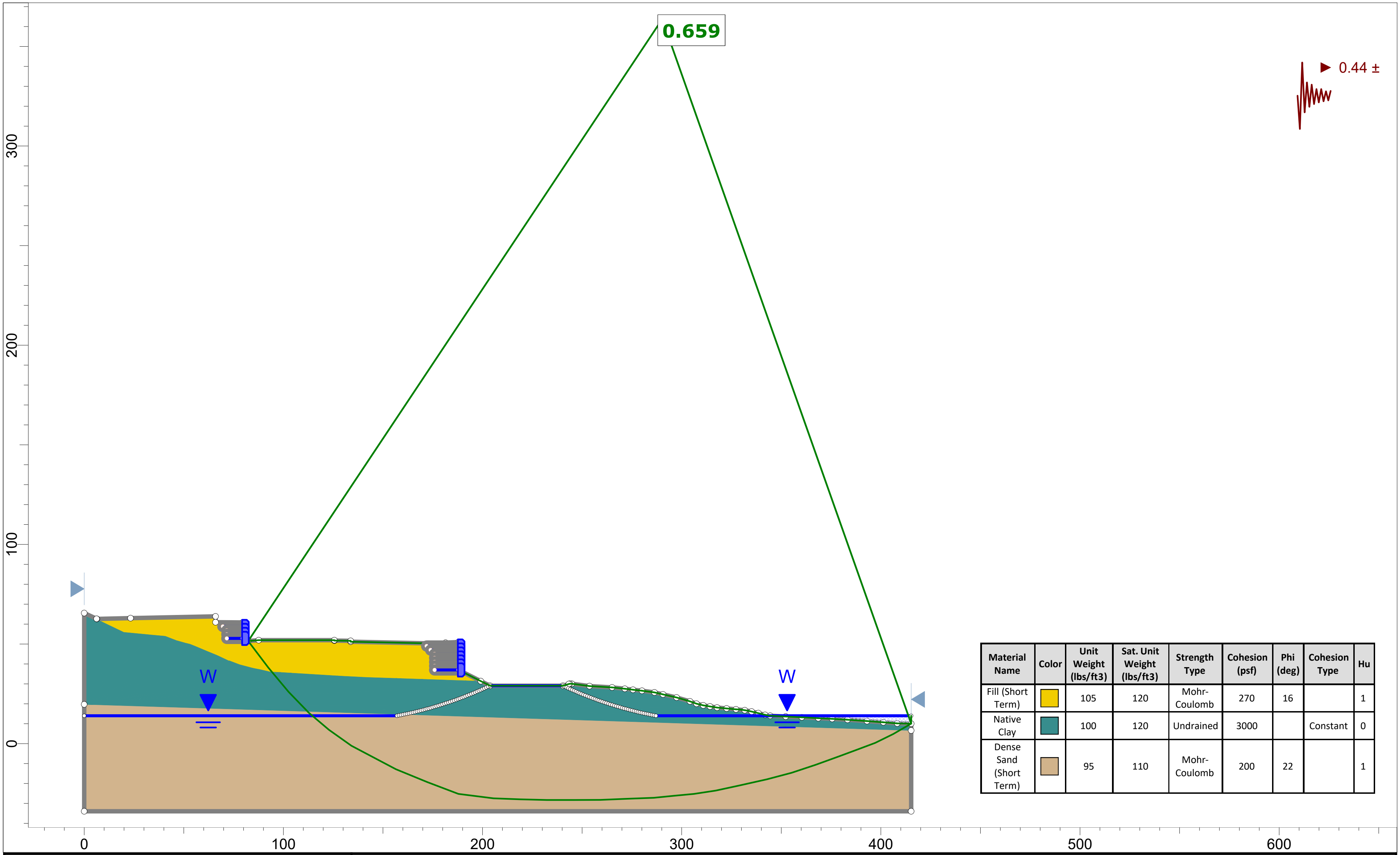
rocscience

SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	SectionBB (2to1).slmd



rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Group Group 1	*Scenario* Master Scenario	
	Drawn By	*Company*	
SLIDEINTERPRET 9.017	*Date* 3/2/2021, 3:56:53 PM	*File Name* SectionBB (2to1).slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Hu
Fill (Short Term)	🟨	105	120	Mohr-Coulomb	270	16		1
Native Clay	🟩	100	120	Undrained	3000		Constant	0
Dense Sand (Short Term)	🟫	95	110	Mohr-Coulomb	200	22		1

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SLIDEINTERPRET 9.017

Project
SLIDE - An Interactive Slope Stability Program

Group Group 1

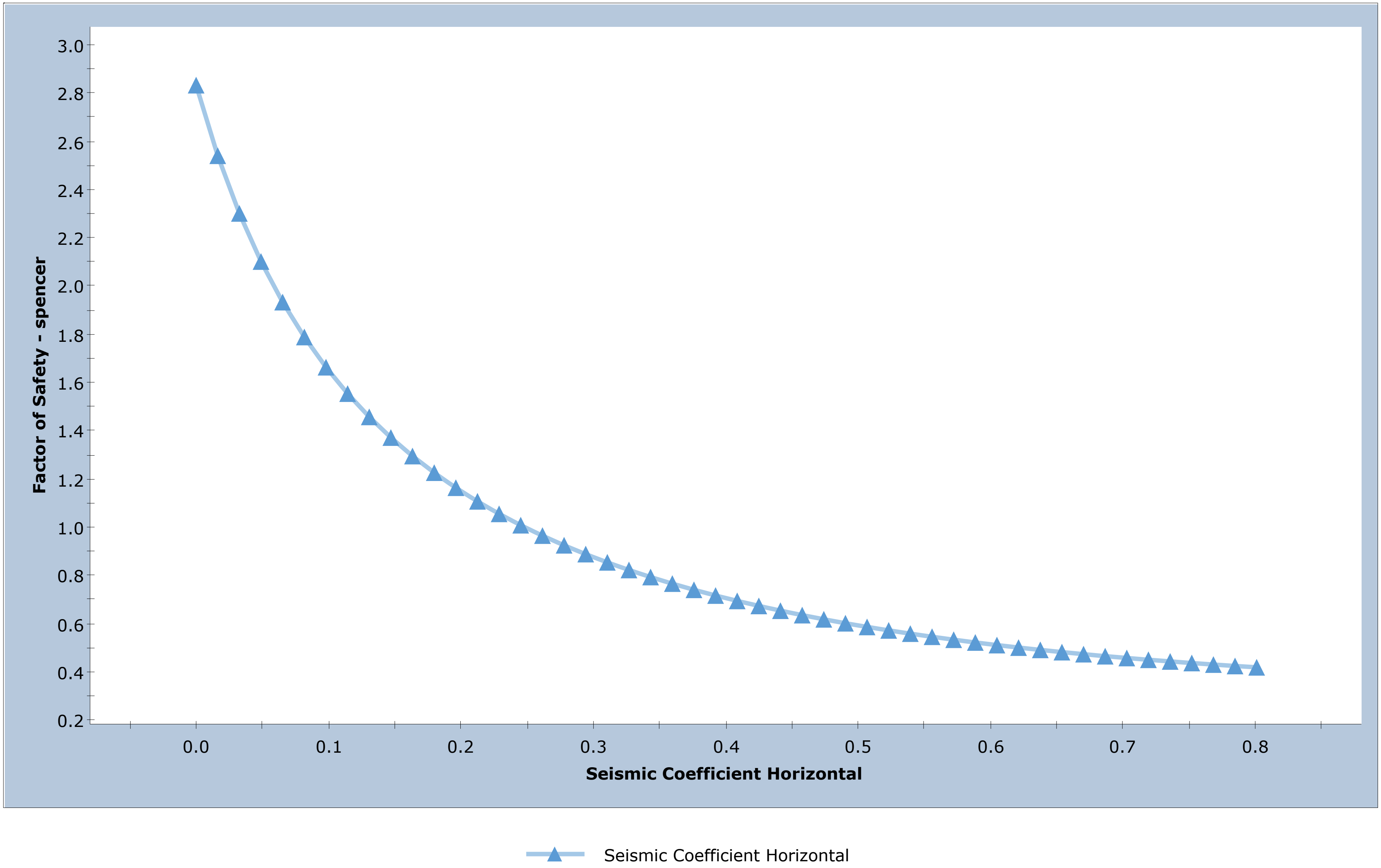
Scenario Master Scenario

Drawn By

Company

Date 3/2/2021, 3:56:53 PM

File Name Section MSE.slmd



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SLIDEINTERPRET 9.017

Project	SLIDE - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Master Scenario
Drawn By		*Company*	
Date	3/2/2021, 3:56:53 PM	*File Name*	Section MSE.slmd

SECTION B

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

by Jonathan D. Bray and Thaleia Travasarou

Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.260		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.10	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.15	seconds	
Moment Magnitude (M_W)	8.0		
Spectral Acceleration (Sa(1.5Ts))	1.5	g	Input the Spectral Acceleration at the base of the sliding mass assuming there is no material above it.

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	2.54	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	30.55	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.00		eq. (3)
D1	15.8	cm	calc. using eq. (7)
D2	30.5	cm	calc. using eq. (7)
D3	58.9	cm	calc. using eq. (7)
P(D>d_threshold)	1.00		eq. (7)

Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 8
7. Rigid slope is assumed for Ts < 0.05 s, i.e. Ts = 0.0. If Ts is just less than 0.05 s, set Ts = 0.050 s
8. When a value for D is not calculated, D is < 1cm
9. ky should be estimated with a slope stability program; the simplified equations shown below provide approximate values.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

MSE WALL SECTION

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

by Jonathan D. Bray and Thaleia Travasarou

Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.260		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.08	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.11	seconds	
Moment Magnitude (M_w)	8.0		
Spectral Acceleration (Sa(1.5Ts))	1.25	g	Input the Spectral Acceleration at the base of the sliding mass assuming there is no material above it.

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	2.54	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	19.98	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.00		eq. (3)
D1	10.4	cm	calc. using eq. (7)
D2	20.0	cm	calc. using eq. (7)
D3	38.5	cm	calc. using eq. (7)
P(D>d_threshold)	1.00		eq. (7)

Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 8
7. Rigid slope is assumed for Ts < 0.05 s, i.e. Ts = 0.0. If Ts is just less than 0.05 s, set Ts = 0.050 s
8. When a value for D is not calculated, D is < 1cm
9. ky should be estimated with a slope stability program; the simplified equations shown below provide approximate values.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

APPENDIX C:
RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

Due to the presence of sandy soil layers beneath the building site that are prone to liquefaction, we judge the site should be classified as “Site Class F” per the 2019 California Building Code. However, per section 20.3.1 of the ASCE 7-16, an equivalent linear site-specific response analysis (i.e., SHAKE, DeepSoil, etc.) is not required if the proposed structure has a fundamental period of less than 0.5 seconds. We anticipate the proposed structures will have fundamental periods less than 0.5-seconds; therefore, based on the harmonic mean of the blow counts we recommend classifying the site as a “Site Class D”.

The ASCE 7-16 mapped spectral acceleration parameters at a period of 0.2-second, S_S, and 1.0-second, S_1, at the project site are 2.47 g and 0.94 g, respectively. Per ASCE 7-16 Table 11.4-1 a Site-Specific Ground Motion shall be developed per Section 11.4.8 for S_S values greater than 1.0 g for Site Class E sites and all cases for Site Class F sites. Additionally, a Site-Specific Ground Motion Hazard Analysis shall be performed per ASCE 7-16 Section 11.4.8 if the S_1 value is greater than 0.2 g for Site Class D, greater than 1.0 g for Site Class E, and all cases for Site Class F. Therefore, per ASCE 7-16 Section 11.4.8, we performed a Site-Specific Ground Motion Hazard Analysis per ASCE 7-16 Section 21.2, as described in the sections below.

Probabilistic (MCE_R) Ground Motions: Method 1

A probabilistic acceleration response spectrum, corresponding to a 2% chance of exceedance in 50-years (2,475 return period) was generated utilizing the United States Geologic Survey (USGS) online Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/, accessed 2019) for a Site Class D soil profile (V_{S30} = 270 m/s) an the Dynamic: Conterminous U.S. 2014 (v4.2.0) model. The accelerations given were modified by the risk coefficients C_{RS} and $C_{R1,}$ 0.93 and 0.90, respectively. The accelerations were further converted to the probabilistic spectral response acceleration in the maximum horizontal response utilizing the procedures outlined by in ASCE 7-16. These modifications to the probabilistic spectra correspond to a response with a risk targeted level of 1% probability of collapse within a 50-year period. The resulting probabilistic MCE_R values and spectra are presented on Figures C-1 and C-2, respectively.

Deterministic (MCE_R) Ground Motions

A deterministic acceleration response spectrum was generated utilizing the NGA attenuation models outlined by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell & Borzognia (2014); and Chiou & Youngs (2014) NGA2 West models for a Site Class D (V_{S30} = 270 m/s). The geometric average of the 84th percentile spectral accelerations from the aforementioned attenuation relationships were modified for the probabilistic spectral response acceleration in the maximum horizontal direction, utilizing the procedures outlined in ASCE 7-16. The resulting deterministic MCE_R values and spectra are shown on Figures C-1 and C-2, respectively. The deterministic MCE_R spectra shall not be less than the Lower Limit Deterministic MCE_R Response Spectrum, as described in ASCE 7-16 Figure 21.2-1 which is tabulated and plotted on Figures C-1 and C-2, respectively.

Site Specific MCE_R

The site specific MCE_R spectral response acceleration at any period shall be taken as the lesser of the response accelerations from the probabilistic ground motions and the deterministic ground motions and is presented on Figure C-3. Additionally, per ASCE 7-16 Section 21.3, the design spectral response acceleration at any period is equal to $2/3^{rds}$ the MCE_R Response Spectrum, as shown on Figure C-3.

Per ASCE 7-16 Section 21.4, the MCE_R spectral response acceleration parameters shall be taken from the Site-Specific Spectrum defined as follows and are presented on Figure C-3:

- S_{DS} – The S_{DS} parameter shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period between 0.2 and 5.0-seconds. However, the values obtained shall not be less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.5.
- S_{D1} – The S_{D1} parameter shall be taken as the maximum value of the product, TS_a, for periods between 1.0 and 2.0-seconds for Site Class C and B sites; and periods between 1.0 and 5.0-seconds for Site Class D, E & F sites. However, the values obtained shall not be less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.5.
- S_{MS} – The S_{MS} parameter is equal to 1.5 times the S_{DS} value, but not less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.4.
- S_{M1} – The S_{M1} parameter is equal to 1.5 times the S_{D1} value, but not less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.4.

ASCE 7-16
SITE SPECIFIC RISK-TARGETED
MAXIMUM CONSIDERED EARTHQUAKE (MCE_R)

Project Name:	Hillcrest Residential Development	Latitude:	36.9105
Project Numb:	1680.023	Longitude:	-121.7719

General Seismic Parameters
ASCE 7-16 Section 11.4

Site Class:	D
S_S (g):	2.47
S_1 (g):	0.94
F_a:	1.20
F_v:	N/A
T_L (sec):	12.0
C_{RS}:	0.93
C_{R1}:	0.90

Minimum Design Spectra Parameters
ASCE 7-16 Section 21.3

Site Class:	D	S_{MS} (g):	2.47
S_S (g):	2.47	S_{M1} (g):	2.35
S_1 (g):	0.94	S_{DS} (g):	1.65
F_a:	1.00	S_{D1} (g):	1.57
F_v:	2.50	T_0 (sec):	0.19
		T_S (sec):	0.95

Deterministic MCE Screening
ASCE 7-16 (Sup #1) 21.2.3

Fa:	1.00
1.2 x Fa (g):	1.20
Max PSHA (g):	2.79
DSHA Rqd.:	**YES**

Min. Deterministic MCE
ASCE 7-16 (Sup #1) 21.2.2

Fa:	1.00
1.5 x Fa (g):	1.50
Max DSHA (g):	2.10
Min MCE Rqd.:	**NO**

Probabilistic MCE
ASCE 7-16 Section 21.2.1 - Method 1

Period (sec)	Sa_{RotD50} (g)	$Sa_{RotD100}/Sa_{RotD50}$	$Sa_{RotD100}$ (g)	C_R	Sa (g)
0.01	1.04	1.10	1.15	0.931	1.07
0.10	1.76	1.10	1.94	0.931	1.81
0.20	2.28	1.10	2.51	0.931	2.33
0.30	2.57	1.13	2.89	0.928	2.68
0.50	2.58	1.18	3.03	0.921	2.79
0.75	2.19	1.24	2.71	0.912	2.47
1.00	1.87	1.30	2.43	0.903	2.19
2.00	1.10	1.35	1.48	0.903	1.34
3.00	0.75	1.40	1.05	0.903	0.95
4.00	0.54	1.45	0.79	0.903	0.71
5.00	0.42	1.50	0.63	0.903	0.57

Deterministic MCE
NGA West2 2014 - 84th Percentile

Period (sec)	Sa_{RotD50} (g)	$Sa_{RotD100}/Sa_{RotD50}$	$Sa_{RotD100}$ (g)
0.01	0.73	1.10	0.81
0.02	0.74	1.10	0.81
0.03	0.75	1.10	0.82
0.05	0.82	1.10	0.90
0.08	0.97	1.10	1.06
0.10	1.12	1.10	1.23
0.15	1.35	1.10	1.48
0.20	1.51	1.10	1.67
0.25	1.66	1.11	1.85
0.30	1.77	1.13	1.99
0.40	1.83	1.15	2.10
0.50	1.78	1.18	2.09
0.75	1.49	1.24	1.85
1.00	1.26	1.30	1.64
1.50	0.88	1.33	1.17
2.00	0.65	1.35	0.88
3.00	0.41	1.40	0.57
4.00	0.27	1.45	0.38
5.00	0.18	1.50	0.27
7.50	0.08	1.50	0.12
10.00	0.04	1.50	0.07

Scaled Deterministic MCE
ASCE 7-16 (Sup #1) 21.2.2

Period (sec)	Sa (g)
0.01	0.58
0.02	0.58
0.03	0.59
0.05	0.64
0.08	0.76
0.10	0.88
0.15	1.06
0.20	1.19
0.25	1.32
0.30	1.42
0.40	1.50
0.50	1.49
0.75	1.32
1.00	1.17
1.50	0.83
2.00	0.63
3.00	0.41
4.00	0.27
5.00	0.20
7.50	0.09
10.00	0.05

Site Specific MCE_R
ASCE 7-16 Section 21.2.3

Period (sec)	Sa (g)
0.01	0.81
0.02	0.81
0.03	0.82
0.05	0.90
0.08	1.06
0.10	1.23
0.15	1.48
0.20	1.67
0.25	1.85
0.30	1.99
0.40	2.10
0.50	2.09
0.75	1.85
1.00	1.64
1.50	1.17
2.00	0.88
3.00	0.57
4.00	0.38
5.00	0.27
7.50	0.12
10.00	0.07

Site-Specific Design Spectrum
ASCE 7-16 Section 21.3

Period (sec)	Sa (g)
0.01	0.54
0.02	0.54
0.03	0.55
0.05	0.60
0.08	0.71
0.10	0.82
0.15	0.99
0.20	1.11
0.25	1.23
0.30	1.33
0.40	1.40
0.50	1.39
0.75	1.23
1.00	1.09
1.50	0.78
2.00	0.58
3.00	0.38
4.00	0.26
5.00	0.18
7.50	0.08
10.00	0.04

80% General Response Spectrum
ASCE 7-16 Section 21.3

	Period (sec)	Sa (g)	80% Sa (g)
	0.01	0.71	0.57
	0.04	0.87	0.69
	0.07	1.02	0.82
	0.10	1.18	0.94
	0.13	1.34	1.07
	0.16	1.49	1.19
T_O =	0.19	1.65	1.32
T_S =	0.95	1.65	1.32
	1.26	1.24	0.99
	1.57	1.00	0.80
	1.89	0.83	0.67
	2.20	0.71	0.57
	2.51	0.63	0.50
	2.82	0.56	0.44
	3.13	0.50	0.40
	3.44	0.46	0.36
	3.75	0.42	0.33
	4.07	0.39	0.31
	4.38	0.36	0.29
	4.69	0.33	0.27
	5.00	0.31	0.25

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ASCE 7-16 MCEr CALCULATIONS

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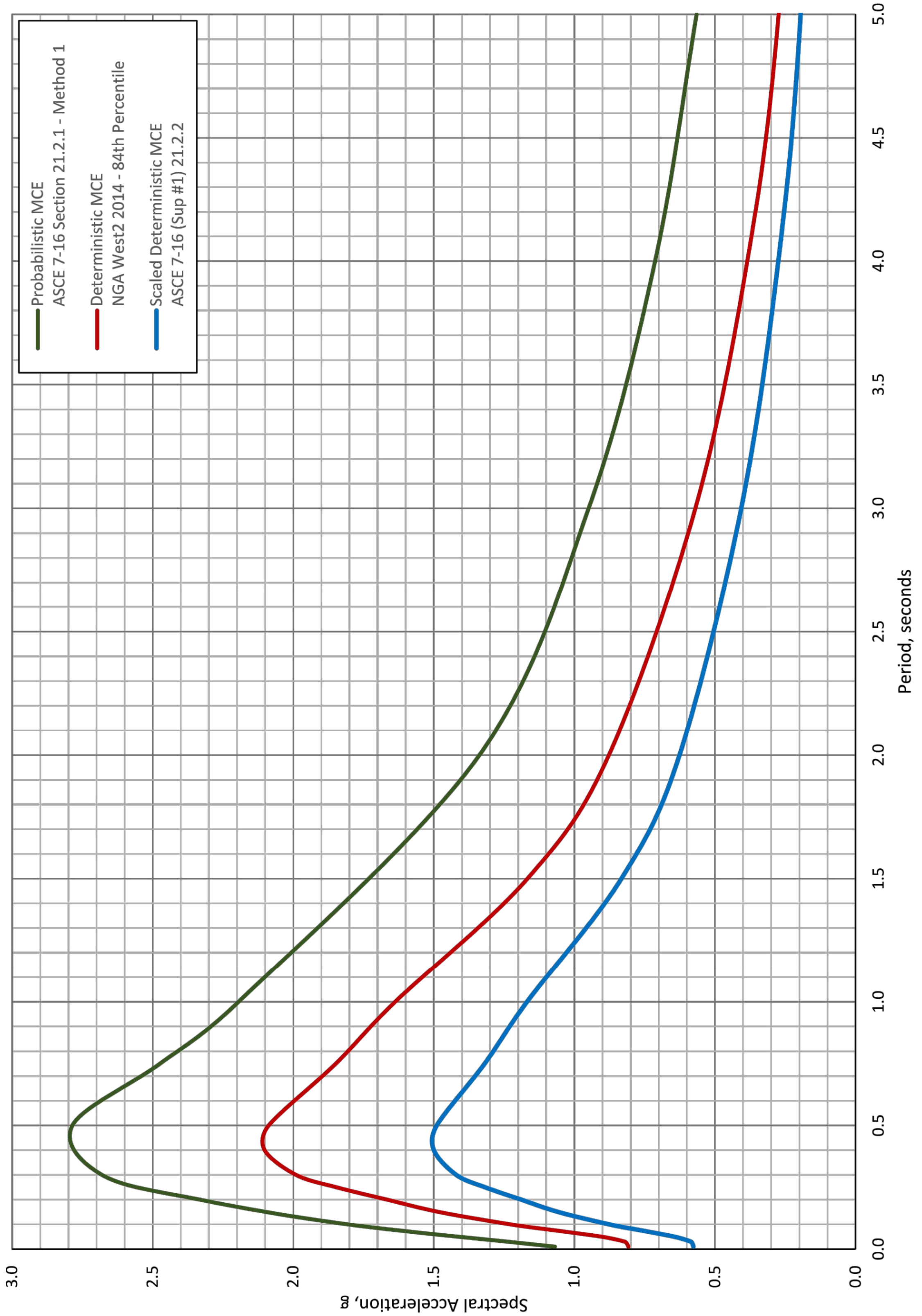
Watsonville, California

Drawn BSP
Checked

Project No. 2914.001 Date: 1/08/21

C-1
FIGURE

SITE SPECIFIC SPECTRA CANDIDATES



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ASCE 7-16 MCEr CANDIDATE SPECTRA

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Project No. 1680.023 Date: 1/09/21

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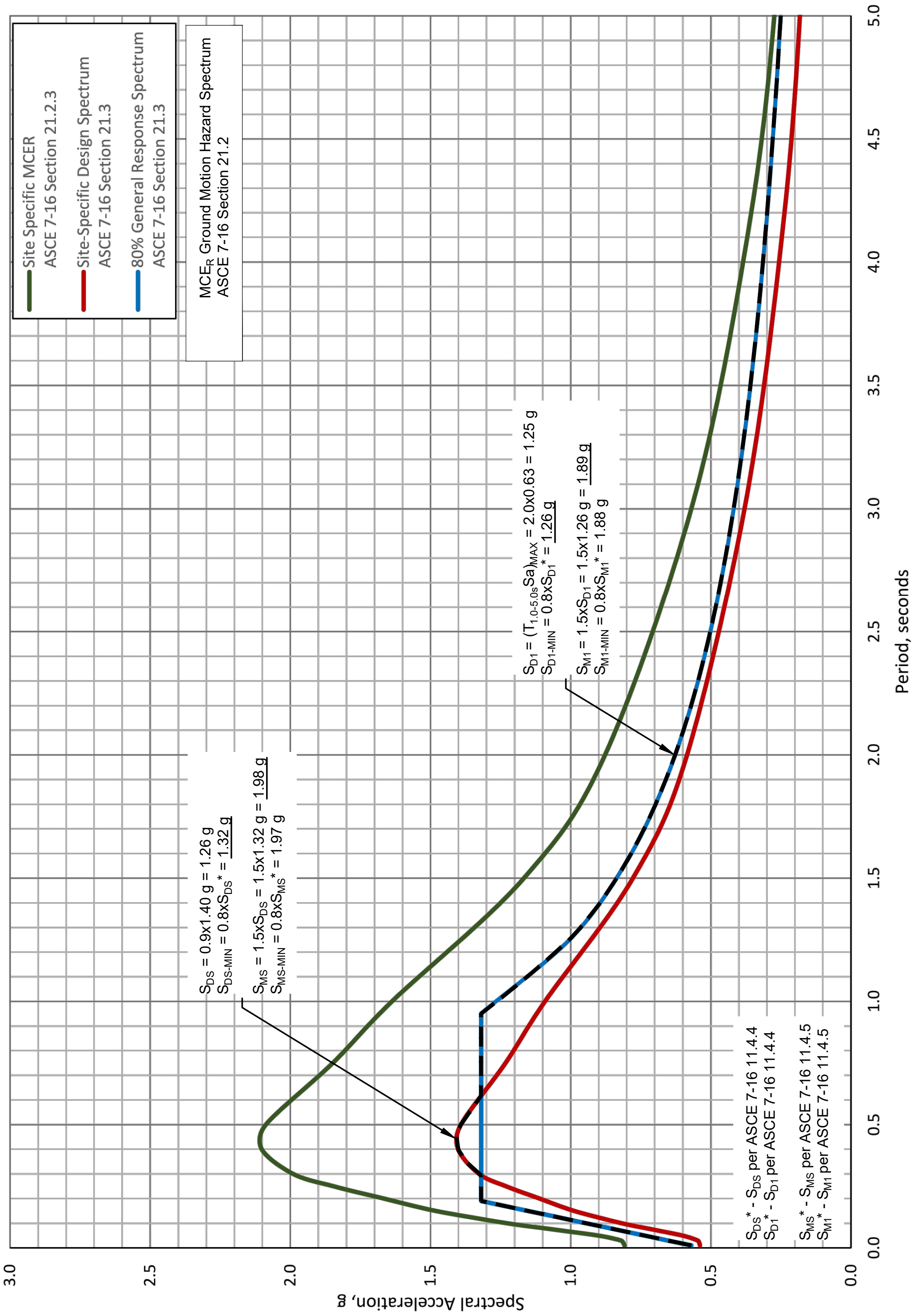
C-2
FIGURE

SITE SPECIFIC MCE_R



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ASCE 7-16 MCEr DESIGN SPECTRUM

Hillcrest Residential Subdivision
Watsonville, California

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Project No. 1680.023 Date: 1/09/21

C-3
FIGURE