CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	25500 Clawiter Road Industrial
LOCATION	25500 Clawiter Road Hayward, California
CLIENT	Dermody Properties
PROJECT NUMBER	916-2-1
DATE	August 26, 2020

GEOTECHNICAL

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Client Address	**11900 NE 1st Street, Suite 300 Bellevue, Washington**
Project Number	**916-2-1**
Date	**August 26, 2020**

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Dermody Properties for the 25500 Clawiter Road Industrial project in Hayward, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A Conceptual Site Plan titled, "Scheme 4G," prepared by Ware Malcomb, dated July 23, 2020.
- As built drawings, including, plumbing, electrical, mechanical and civil plans, dated 1996.
- A geotechnical report prepared by Harza, dated March 29, 1996.

1.1 PROJECT DESCRIPTION

The project will include redeveloping the approximately 20¼-acre site for a new industrial facility. The new facility will include two buildings. Industrial Building 1 will total approximately 227,500 square feet including about 2,500 square feet of office space. Industrial Building 2 will be 123,600 square feet with 3,100 square feet of office space. We anticipate the buildings will be single-story with 50 to 52 feet typical bay spacing, interior clear height of 36 to 38 feet, and of concrete tilt-up construction. Loading docks will be located along the south side of Industrial Building 1 and east side of Industrial Building 2. At-grade auto and trailer parking and drive aisles will surround both buildings. Detention basins will be located at the northern and eastern margins of the site. Appurtenant utilities, landscaping, and other improvements necessary for overall site development will also be constructed. Existing site improvements will be demolished prior to new construction.

Cuts and fills up to about 3 to 4 feet are anticipated across the site to rework undocumented fills beneath the building pads. Structural loads have not been provided at this time; however, structural loads are expected to be typical for similar structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated July 14, 2020 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of seven borings drilled on August 4 and 5, 2020 with truck-mounted hollow-stem auger drilling equipment and seven Cone Penetration Tests (CPTs) advanced on June 30, 2020. The borings were drilled to depths of approximately 30 to 42½ feet; the CPTs were advanced to depths of approximately 50 to 150 feet below existing site grades. Seismic shear wave velocity measurements were collected from CPT-1. All of our borings were advanced adjacent to our CPTs, with the exception of EB-4, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses, consolidation tests, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located approximately 2 miles east of the San Francisco Bay. Based on our site exploration and review of recent geologic maps of the area (Graymer 2000; Helley and Graymer, 1997), the site is underlain by Holocene age fan or basin (Qhaf and Qhb). The fan deposits (Qhaf) are generally described by Graymer (2000) as medium dense to dense, gravelly sand or sandy gravel, grading upward to sandy or silty clay. It may contain localized layers, lenses and stringers of silt and sand. The basin deposits (Qhb) are generally very fine silty clays and clays deposited near the distal edge of alluvial fans and adjacent to Bay Mud, which may extend partially onto the western or southern edge of the site. The young sediments are generally underlain by older alluvial fan deposits collectively referred to Older Bay Mud or Old Bay Clay. These older alluvial soils generally consist of clays, sands, silts and localized gravel layers.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7 percent for UCERF2 to about 7.0 percent for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Total Length)	3.5	5.6
Calaveras	11.1	17.9
San Andreas (1906)	14.9	24.0
Monte Vista-Shannon	15.3	24.6

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND AND SURFACE DESCRIPTION

The site consists of two parcels totaling approximately 20½ acres and is located in an industrial park area. The site is currently occupied by approximately 225,000 square feet of existing office and industrial buildings and was previously used as Berkeley Farms production and distribution facility. The site is relatively level but graded to drain to storm drainage facilities. Building 1 is proposed to be located on the south side of the property while Building 2 is proposed to be located on the eastern side of the property.

Surface pavements at Borings EB-1 and EB-2 generally consisted of 3 inches of asphalt concrete over 6 inches of aggregate base overlying subgrade. Surface pavements at Borings EB-3, EB-4, and EB-5 generally consisted of 9 to 10 inches of Portland Cement Concrete (PCC) over 4 inches of aggregate base overlying subgrade. Based on visual observations, the existing pavements are in fair to poor shape with some significant cracking and minor alligator cracking in both the asphalt and concrete pavements.

3.2 SUBSURFACE CONDITIONS

Undocumented fill was encountered in borings EB-6 and EB-7 to a depth of 1½ to 3½ feet below existing ground surface. Boring EB-4 was advanced adjacent to an existing underground storage tank along the southern edge of the project site and encountered undocumented fill to a depth of approximately 16 feet below existing grade. The fill consisted of sandy lean clay and clayey sand. Below the surface pavements and fill, our borings generally encountered medium stiff to hard lean clay with variable amounts of sand, medium dense to dense sand with interbedded with variable amounts of silt to the maximum depth explored of 43 feet below the existing ground surface.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The results of the surficial PI test indicated a PI of 29, indicating moderate to high expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 3 to 22 percent moisture. In our opinion, we estimate this corresponds to about 5 percent below to 5 percent above the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our borings at depths ranging from 13 to 19 feet below current grades. Pore pressure dissipation testing was conducted at CPT-3 through CPT-7.

Groundwater was inferred based on pore pressure dissipation testing performed and estimated to be at depths ranging from about 9 to 21½ feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Table 2: Depth to Groundwater

Boring/CPT Number	Date Drilled	Depth to Groundwater (feet)	Depth of Boring/CPT (feet)
EB-1	8/4/2020	13	35
EB-2	8/4/2020	18	30
EB-3	8/4/2020	19	41½
EB-4	8/5/2020	18	30
EB-5	8/5/2020	13	30
EB-6	8/5/2020	14	42½
EB-7	8/5/2020	14½	41½
CPT-1	7/30/2020	14	150.6
CPT-2	7/30/2020	17	50.7
CPT-3	7/30/2020	11.2	50.7
CPT-4	7/30/2020	8.9	50.7
CPT-5	7/30/2020	18.5	50.7
CPT-6	7/30/2020	16.7	50.7
CPT-7	7/30/2020	21.3	50.7

Based on depth to groundwater maps, the historic high groundwater is mapped at about 10 feet below the ground surface (CGS, Hayward 7.5-Minute Quadrangle, 2003). In general, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. For our analysis, a design groundwater level of 10 feet below existing ground surface was selected based on the depth to groundwater maps, depth of groundwater encountered in our borings and our previous experience in the area.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration was estimated for analysis as allowed in the 2019 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.852g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Hayward, 2003). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design groundwater depth of 10 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), Andrus and Stokoe (2000), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the “Estimated Ground Shaking” section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 through CPT-7) are presented on Figures 4A through 4G of this report and summarized on Table 3 below. Calculations for these CPTs are attached as Appendix C.

Table 3: Summary of Seismic Settlements

CPT Number	Estimated Dry Sand Settlement (inches)	Estimated Liquefaction Settlement (inches)	Total Seismic Settlement (inches)
CPT-1	0.25	0.36	0.61
CPT-2	0.0	0.24	0.32
CPT-3	0.3	0.4	0.7
CPT-4	0.1	0.52	0.62
CPT-5	0.0	0.1	0.1
CPT-6	0.0	0.78	0.78
CPT-7	0.0	0.7	0.7

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from approximately ¼- to ¾ inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, total differential seismic settlements are anticipated to be less than about ½ inch between foundation elements, estimated to be about 40 to 60 feet.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 10-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of loose and medium dense, unsaturated sandy soils above the design groundwater depth of 10 feet based on the work by Tokimatsu and Seed (1984). Our analyses indicate that the in-situ, loose and unsaturated sandy soils could experience up to ⅓-inch of movement after strong seismic shaking.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 2 miles inland from the San Francisco Bay shoreline, and is approximately 26 to 35 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area of minimal flood hazard. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Presence of undocumented fill
- Potential for seismic and liquefaction-induced settlements
- Shallow groundwater
- Potential for wet, unstable trench bottoms
- Presence of highly expansive soils

5.1.1 Presence of Undocumented Fill

Potential concerns that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fill. As previously discussed, undocumented fill was encountered in borings EB-6 and EB-7 to depths ranging from 1½ to 3½ feet below existing ground surface. Boring EB-4 was advanced adjacent to an existing underground storage tank along the southern edge of the project site and encountered undocumented fill to a depth of approximately 16 feet below existing grade. The lateral limit of the deeper undocumented fill is not known but may extend to within the proposed building footprint. Undocumented fills are expected to vary in thickness and consistency across the site. The fill consisted of sandy lean clay with variable amounts of sand and clayey sand with gravel. All fills should be completely removed from within building areas. Detailed grading recommendations are presented in Section 6.3 below. Depending on the actual limits of undocumented fill and quantity, alternative mitigation measures may also be feasible in lieu of over-excavation and replacing with engineered fill.

5.1.2 Potential for Seismic and Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for settlement of loose, unsaturated sand layers and liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that seismic and liquefaction-induced settlement on the order of ¼- to ¾-inch could occur, resulting in differential settlement up to ½ inch. Based on our review of the estimated foundation loads, it should be feasible to support the proposed buildings on shallow foundations; however, the building foundations will need to be designed to tolerate total and differential settlement due to static loads and liquefaction-induced settlement. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.3 Shallow Groundwater

Shallow groundwater has been measured at the site at depths ranging from approximately 13 to 19 feet below the existing ground surface in our borings, and inferred at shallower depths in our CPTs. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some areas of the site, depending on the depth of the trenches. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.4 Wet, Unstable Trench Bottoms

The proposed utility excavations may extend into saturated clay and sand with varying strength. Due to the high moisture content of this material, it will likely be unstable. The contractor should anticipate that in addition to dewatering, it may be necessary to remove an additional approximately 12 to 18 inches of native soil beneath the trench bottoms and replace it with a bridging layer, such as crushed rock to avoid delays. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

5.1.5 Expansive Soils

Highly expansive surficial soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition, and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

6.3 REMOVAL OF EXISTING FILLS

All fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Site C materials.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with the OSHA soil classification.

6.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.6 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below.

Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 to 18 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) and Portland Cement Concrete (PCC) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections. AC/AB grindings may not be reused within the building areas. Laboratory testing will be required to confirm the grindings meet project specifications. The grinding operation for the AC may leave significant oversize chunks and won’t meet the Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements of this report, it may be used as select fill within the building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.7.3 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7.4 Non-Expansive Fill Using Chemical Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of chemical treatment required.

6.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 4: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

6.8.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

6.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Locally, seasonal high groundwater is mapped at a depth of 10 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.

- The site has a known geotechnical hazard consisting of soils subject to liquefaction; therefore, stormwater infiltration facilities may not be feasible.

- In our opinion, infiltration locations within 15 feet of the buildings would create a geotechnical hazard.

- Infiltration devices should be located at least 100 feet away from septic tanks and underground storage tanks with hazardous materials, as well as any other potential underground sources of pollution.

- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

- Local Water District policies or guidelines may limit locations where infiltration may occur, require greater separation from seasonal high groundwater, or require greater setbacks from potential sources of pollution.

6.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.11.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.11.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.11.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.12 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

We developed site-specific design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20 and 21 and Supplement No. 1 of ASCE 7-16.

7.1 SITE LOCATION AND PROVIDED DATA FOR 2019 CBC SEISMIC DESIGN

The project is located at latitude 37.633825° and longitude -122.117991°, which is based on Google Earth (WGS84) coordinates at the center of the site at 25500 Clawiter Road in Hayward, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 SITE CLASSIFICATION – CHAPTER 20 OF ASCE 7-16

Code-based site classification and ground motion attenuation relationships are based on the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile, or V_{S30}.

Shear wave velocity (V_S) measurements were performed while advancing CPT-1, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 751 feet per second (or 229 meters per second). Based on the conditions encountered in our borings and in accordance with Table 20.3-1 of ASCE 7-16, we recommend the site be classified as Soil Classification D, which is described as a “stiff soil” profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as “determined” for the purposes of estimating the seismic design parameters from the code outlined below. Our site-specific ground motion hazard analysis considered a V_{S30} of 229 m/s (751 ft/s).

7.3 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum

Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were determined using the ATC Hazards by Location website (https://hazards.atcouncil.org).

The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 5.

In accordance with CBC Section 1613.2.5, Risk Category I, II, or III structures with mapped spectral response acceleration parameter at the 1-second period (S_1) equal to or greater than 0.75, are assigned Seismic Design Category E. In accordance with Section 11.4.8 of ASCE 7-16, structures on Site Class D sites with mapped 1-second period spectral acceleration (S_1) values greater than or equal to 0.2 require a site-specific ground motion hazard analysis be performed in accordance with Section 21.2 of ASCE 7-16. We also assumed that the Structural Engineer will not take the “Exceptions” listed in Section 11.4.8 of ASCE 7-16. ***Design site-specific seismic parameters are presented in Table 6, Section 7.5. The values in Table 5 should not be used for design. Values summarized in Table 5 are only used to determine Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis (Section 7.4 to follow).***

Table 5: Site Class D: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.633825°
Site Longitude	-122.117991°
Risk Category	II*
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.721
1-second Period Mapped Spectral Acceleration[1], S_1	0.653
Short-Period Site Coefficient – Fa	1
Long-Period Site Coefficient – Fv	**null
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.721
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	**null
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.147
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	**null
Long-Period Transition – T_L	8
Mapped MCEG Peak Ground Acceleration – PGA	0.724
Site Coefficient – F_{PGA}	1.1
MCEG Mapped Adjusted for Site Effects – PGA_M	0.796

**Assumed, to be confirmed by Structural Engineer*
***null per Section 11 of ASCE 7-16*

7.4 SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

Following Section 11.4.8 of ASCE 7-16, we performed a ground motion hazards analysis (GMHA) in accordance with Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project.

Our analyses were performed using the USGS interface Unified Hazard Tool (UHT) based on the UCERF 3 Data Set, Building Seismic Safety Council (BSSC) Scenario Catalog 2014 event set (BSSC 2014), and the 2014 National Seismic Hazard Maps – Source Parameters (NSHMP deterministic event set). Additionally, we utilized the USGS program Response Spectra Plotter with combined models (Combined: WUS 2014 (4.1)).

Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation West 2 (NGA-West 2) relationships: Boore-Atkinson (2013), Campbell-Bozognia (2013), Chiou-Youngs (2013), and Abrahamson-Silva (2013). Rotation factors (scale factors) were determined as specified in ASCE 7-16 Chapter 21, Section 21.2, to calculate the maximum rotated component of ground motions (ASCE, 2016).

7.4.1 Probabilistic MCE_R

We performed a probabilistic seismic hazard analysis (PSHA) in accordance with ASCE 7-16 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50-year period (2,475-year return period). The probabilistic MCE spectrum was multiplied by Risk Coefficients (C_R) to determine the probabilistic MCE_R. We used Risk Coefficients (C_{RS} and C_{R1}) of 0.925 and 0.912, respectively, based on ASCE 7-16 Section 21.2.1.1 - Method 1 and the ATC website. Risk coefficients for the various periods are presented in Table 6, Column 3.

The resulting probabilistic MCE_R for site class D are presented on Figure 5 (red line). Spectral ordinates are tabulated in Table 6, Column 6.

7.4.2 Deterministic MCE_R

We performed deterministic seismic hazard analyses in accordance with ASCE 7-16 Section 21.2.2 and ASCE 7-16 Supplement No. 1. The deterministic MCE_R acceleration response spectrum is calculated as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. The largest deterministic ground motion for all periods resulted from a M_w 7.58 earthquake on the Hayward Fault (RC+HN+HS+HE segments), located at a distance of approximately 6.77 km from the site.

In accordance with Supplement No.1 of ASCE 7-16, when the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than $1.5F_a$ then the largest 84th percentile rotated response spectrum (Table 6, Column 4) shall be scaled by a single factor such that the maximum response spectral acceleration equals 1.5F_a. For Site Classes A, B, C and D, F_a is determined using Table 11.4.1 with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0. When the largest spectral response acceleration of the probabilistic ground motion response of 21.2.1 is less than 1.2F_a, the deterministic ground motion response spectrum does not need to be calculated.

As the largest probabilistic spectral response acceleration was determined to be 2.694 which is greater than 1.2F_a, where F_a is taken as 1.000 from Table 11.4-1 in ASCE 7-16 Supplement No.1, the 84th percentile rotated response spectrum was calculated as part of the deterministic analyses. The maximum spectral acceleration from the 84th percentile rotated response spectrum was then compared to 1.5F_a to determine if a scale factor needed to be applied. The deterministic MCE spectrum are tabulated in Table 6, Column 5. The deterministic MCE_R is presented graphically on Figure 5 (blue line).

7.4.3 Site-Specific MCE_R

The site-specific MCE_R is defined by ASCE 7-16 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. Spectral ordinates for the site-specific MCE_R are tabulated in Table 6, Column 7 and shown graphically on Figure 5 (dashed black line).

Table 6: Development of Site-Specific MCE_R Spectrum

Period (seconds)	CBC General Spectrum (g)	Risk Coefficient	Det. 84th Percentile Rotated	Deterministic MCE_R (g)	Probabilistic MCE_R (g)	Site-Specific MCE_R (g)
0.000	0.459	0.925	0.938	0.938	1.003	0.938
0.050	0.640	0.925	0.958	0.958	1.327	0.958
0.100	0.822	0.925	1.386	1.386	1.652	1.386
0.150	1.003	0.925	1.709	1.709	1.932	1.709
0.190	1.147	0.925	1.848	1.848	2.156	1.848
0.200	1.147	0.925	1.882	1.882	2.212	1.882
0.250	1.147	0.924	2.015	2.015	2.394	2.015
0.300	1.147	0.923	2.094	2.094	2.575	2.094
0.400	1.147	0.922	2.190	2.190	2.634	2.190
0.500	1.147	0.920	2.207	2.207	2.694	2.207
0.750	1.147	0.916	1.910	1.910	2.372	1.910
0.949	1.147	0.913	1.741	1.741	2.144	1.741
1.000	1.088	0.912	1.698	1.698	2.086	1.698
2.000	0.544	0.912	0.979	0.979	1.202	0.979
3.000	0.363	0.912	0.668	0.668	0.804	0.668
4.000	0.272	0.912	0.476	0.476	0.579	0.476
5.000	0.218	0.912	0.366	0.366	0.443	0.366

7.4.4 Design Response Spectrum

The Design Response Spectrum (DRS) is defined in ASCE 7-16 Section 21.3 as:

- two-thirds of the site-specific MCE_R, but
- not less than 80% of the general design response spectrum

Spectral accelerations corresponding to two-thirds of the MCE_R are tabulated in Table 7, Column 2. Ordinates corresponding to 80% of the general Site Class D response spectrum are tabulated below in Table 7, Column 3. Ordinates of the site-specific DRS are tabulated in Table 7, Column 4. Development of the site-specific DRS is presented graphically on Figure 6 (dashed black line).

Table 7: Development of Site-Specific Design Response Spectrum

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC Site Class C Spectrum (g)	Design Response Spectrum (g)
0.000	0.625	0.367	0.625
0.050	0.639	0.512	0.639
0.100	0.924	0.657	0.924
0.150	1.139	0.803	1.139
0.190	1.232	0.918	1.232
0.200	1.255	0.918	1.255
0.250	1.343	0.918	1.343
0.300	1.396	0.918	1.396
0.400	1.460	0.918	1.460
0.500	1.471	0.918	1.471
0.750	1.273	0.918	1.273
0.949	1.161	0.917	1.161
1.000	1.132	0.871	1.132
2.000	0.653	0.435	0.653
3.000	0.445	0.290	0.445
4.000	0.317	0.218	0.317
5.000	0.244	0.174	0.244

7.5 DESIGN ACCELERATION PARAMETERS

Design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-16. S_{DS} is defined as the design spectral acceleration at 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 seconds, inclusive. S_{D1} is defined as the maximum value of the product, TS_a, for periods from 1 to 2 seconds for sites with vs,30 > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 seconds for sites with vs,30 ≤ 1,200 ft/s ($v_{s,30}$ ≤365.76 m/s).

Site-specific MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as:

- 1.5 times the S_{DS} and S_{D1} values, respectively, but
- not less than 80% of the code-based values presented in Table 5

Recommended design acceleration parameters are summarized in Table 8

When using the Equivalent Lateral Force Procedure, ASCE 7-16 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of S_{D1}/T in Eq. 12.8-3 and $S_{D1}T_L/T^2$ in Eq. 12.8-4. The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 7, Column 4.

Table 8: Site-Specific Design Acceleration Parameters

Parameter	Value
S_{DS}	1.324
S_{D1}	1.335
S_{MS}	1.986
S_{M1}	2.003

7.6 SITE-SPECIFIC MCE_G PEAK GROUND ACCELERATION

We calculated the Site-Specific MCE_G Peak Ground Acceleration (PGA_M) in accordance with ASCE 7-16 Section 21.5. The Site-Specific PGA_M is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 0.985g. The deterministic PGA is considered the greater of the largest 84th percentile deterministic geometric mean PGA (0.852g) or one-half of the tabulated F_{PGA} value from ASCE 7-16 Table 11.8.1 with the value of PGA taken as 0.5g. For the site, F_{PGA} is 1.100 and one-half of the F_{PGA} is 0.55g; therefore, the deterministic PGA is 0.852g. Additionally, the Site-Specific PGA_M may not be less than 80% of the mapped PGA_M determined from ASCE 7-16 Equation 11.8-1. The mapped PGA_M for the site is 0.796g; 80% of PGA_M is 0.637g.

Based on the above, the recommended Site-Specific PGA_M for the site is 0.852g.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Spread Footings

Spread footings should bear entirely on natural, undisturbed soil, or engineered fill, be at least 12 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of moderately expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 9: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	100 to 150 kips
Exterior Isolated Column Footing	50 to 75 kips
Perimeter Strip Footing	4 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be less than ½ inch, resulting in up to ¼ inch of post-construction differential settlement between adjacent foundation elements. We estimate total static and seismic settlement at CPT-1, CPT-3, CPT-6 and CPT-7 to be on the order of ¾ and

1¼ inches, resulting in a total differential settlement of up to ¾ inches between foundation elements, assumed to be on the order of 40 to 60 feet. We recommend the structures be supported on conventional shallow foundations.

As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.35 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 350 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Spread Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 29, the proposed slabs-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick should have a minimum compressive strength of 3,500 psi. The warehouse slab should also be supported on at least 6 inches of non-expansive, crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base. Due to the high plasticity of the surficial soils, an additional 6 inches of non-expansive fill (NEF) should underlie the upper granular base. All base and sub-base materials should be placed and compacted in accordance with the “Compaction” section of this report. If there will be areas within the warehouse that are moisture sensitive, such as equipment and elevator rooms, a vapor barrier may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the “Interior Slabs Moisture Protection Considerations” section for vapor barrier construction. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer’s recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1”	100
¾”	90 – 100
No. 4	0 - 10

The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer’s requirements prior to installation.

9.4 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian loading only should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the “Vehicular Pavements” section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and an assumed R-value of 5. The design R-value was chosen based on engineering judgement considering the proposed pavement areas and potential variable surface conditions following site grading. We have also included pavement structural section alternatives for chemical-treated (lime/cement) subgrade soil with an estimated design

R-value of 50 for your consideration. If it is desired to chemical-treat, we recommend that the upper 12 inches of subgrade soil be treated. Additional testing will need to be performed to determine the appropriate lime/cement percentage to be mixed with the subgrade soil.

Table 10: Asphalt Concrete Pavement Recommendations (Untreated Subgrade)

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0
7.0	4.0	16.0	20.0
7.5	4.5	17.0	21.5
8.0	5.0	18.0	23.0
8.5	5.0	20.0	25.0
9.0	5.5	21.0	26.5
9.5	6.0	22.0	28.0
10.0	6.5	23.0	29.5
10.5	6.5	25.0	31.5
11.0	7.0	26.0	33.0

*Caltrans Class 2 aggregate base; minimum R-value of 78.

Table 11: Asphalt Concrete Pavement Recommendations (Chemical-Treated Subgrade)

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0/4.5	2.5	4.0	6.5
5.0/5.5	3.0	4.0	7.0
6.0	3.5	4.0	7.5
6.5	4.0	4.0	8.0
7.0	4.0	4.5	8.5
7.5	4.5	5.0	9.5
8.0	5.0	5.0	10.0

Table 11 continues

Table 11: Asphalt Concrete Pavement Recommendations (Chemical-Treated Subgrade) Continued

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
8.5	5.0	6.5	11.5
9.0	5.5	6.5	12.0
9.5	6.0	7.0	13.0
10.0	6.5	7.5	14.0
10.5	6.5	8.5	15.0
11.0	7.0	8.5	15.5

*Caltrans Class 2 aggregate base with minimum R-value of 78; minimum chemical-treated subgrade R-value assumed to be 50

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be using the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development. PCC alternatives for chemical-treated (lime/cement) subgrade are also provided in the tables below.

Table 12: PCC Pavement Recommendations (Untreated Subgrade)

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.5
130	6.0

Table 13: PCC Pavement Recommendations (Chemical-Treated Subgrade)

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.0
150	5.5

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

10.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

10.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 14: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls or loading dock walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

11.3 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

11.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Dermody Properties specifically to support the design of the 25500 Clawiter Road Industrial project in Hayward, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Dermody Properties may have provided Cornerstone with plans, reports and other documents prepared by others. Dermody Properties understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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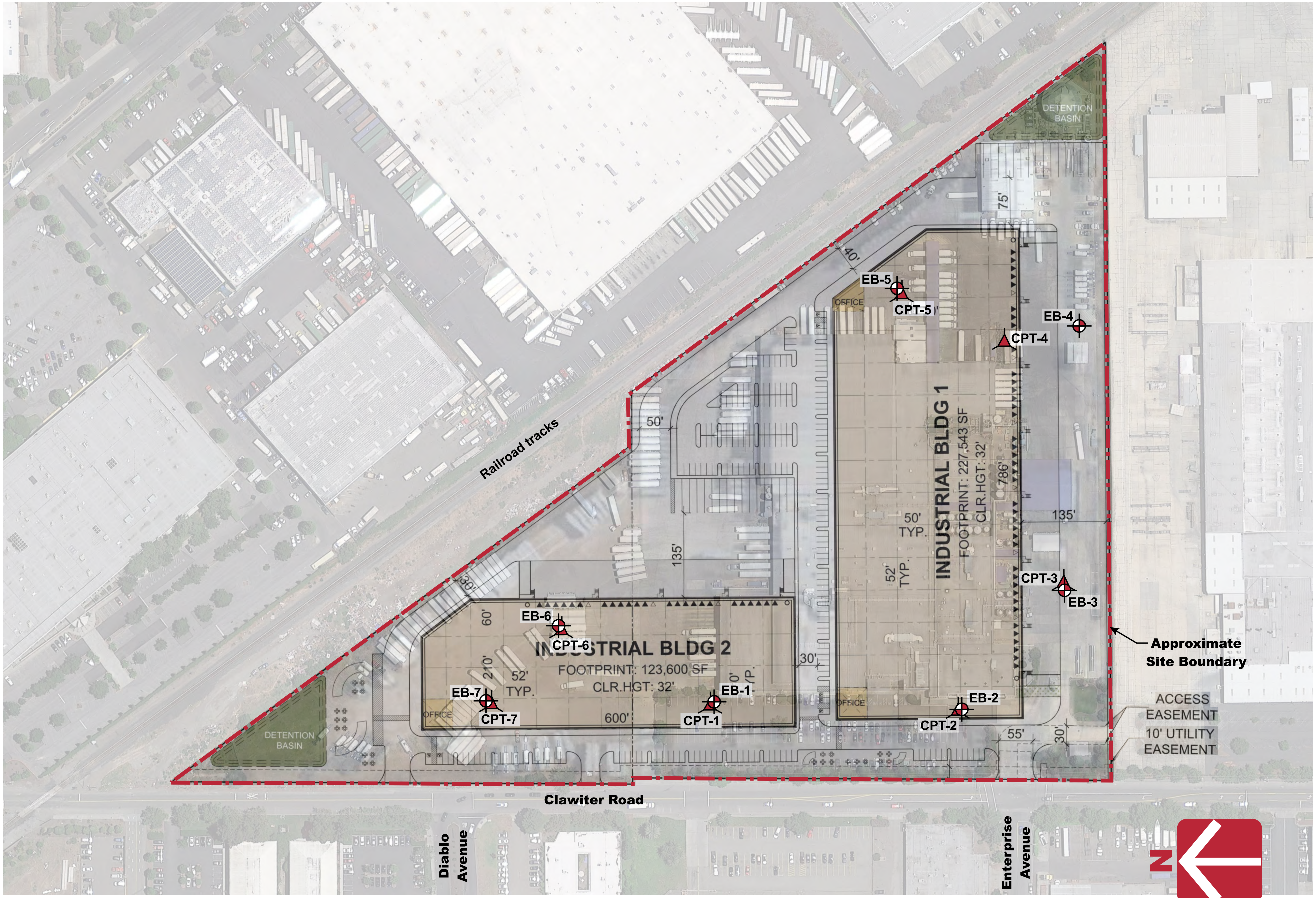
Vicinity Map

25500 Clawiter Road Industrial
Hayward, CA

Project Number	916-2-1
Figure Number	Figure 1
Date	August 2020
Drawn By	RRN



Legend

⊕ Approximate location of exploratory boring (EB)

▲ Approximate location of cone penetration test (CPT)

0 150 300

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 06/20/2019
Overlay by Ware Malcomb, Scheme: 4g - Conceptual Site Plan - Sheet 1, dated 07/22/2020

Project Number 916-2-1

Figure Number Figure 2

Date August 2020

Drawn By RRN

Site Plan

25500 Clawiter Road Industrial
Hayward, CA

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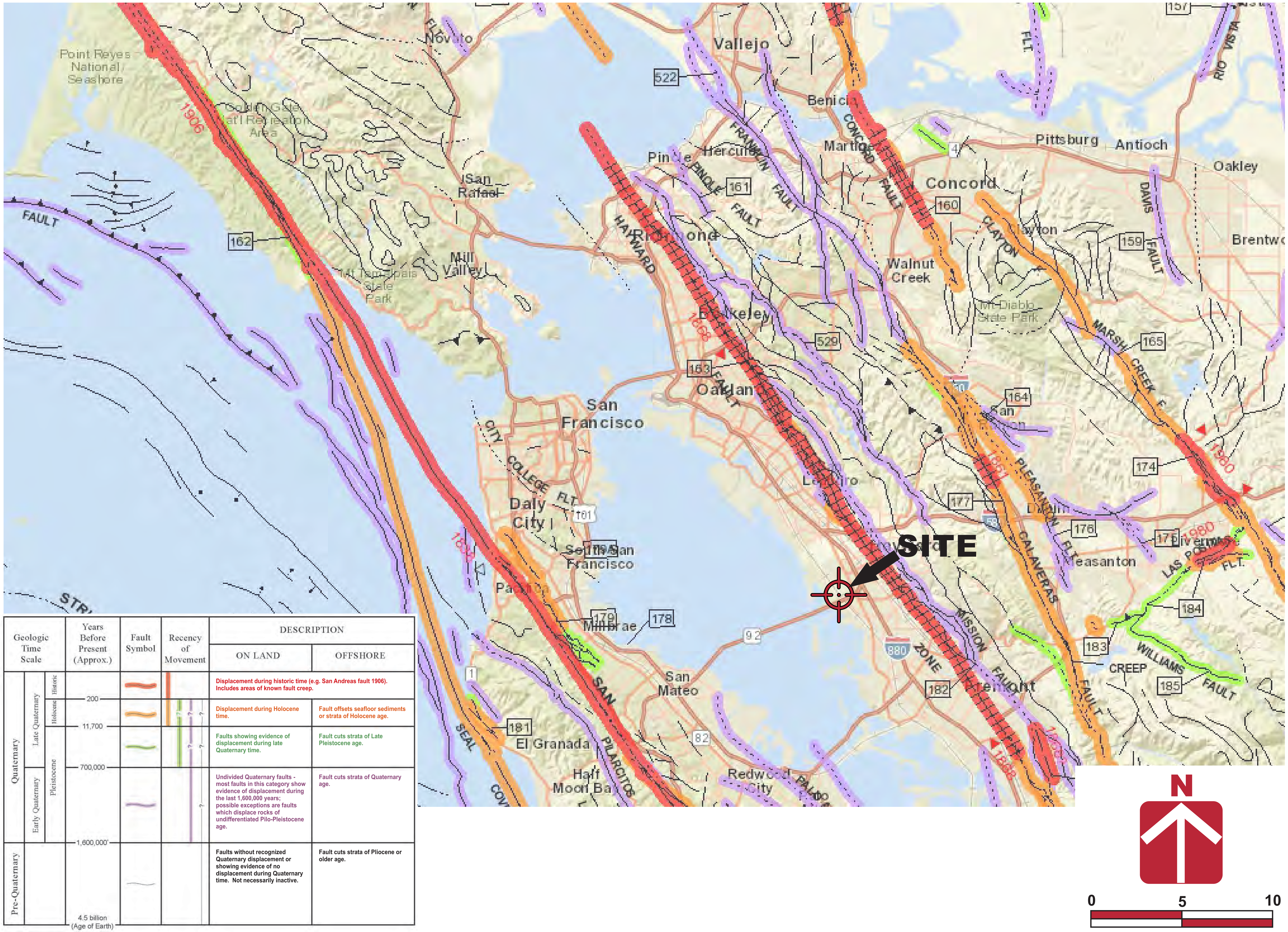


Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000' 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Regional Fault Map

25500 Clawiter Road Industrial
Hayward, CA

Project Number
916-2-1

Figure Number
Figure 3

Date
August 2020

Drawn By
RRN

CORNERSTONE
EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title **25450 Clawiter Road**

Project No. **916-2-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.58**

PGA (Amax) **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **13**

Design Water Depth (feet) **10**

Ave. Unit Weight Above GW (pcf) **121**

Ave. Unit Weight Below GW (pcf) **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.25 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.37 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.6 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 1000.0

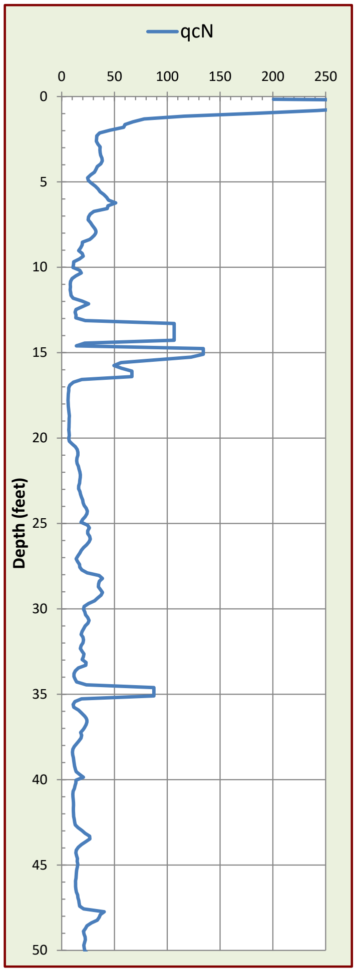
LDI[1] **Corrected for Distance** 0.00 (4 < L/H < 40)

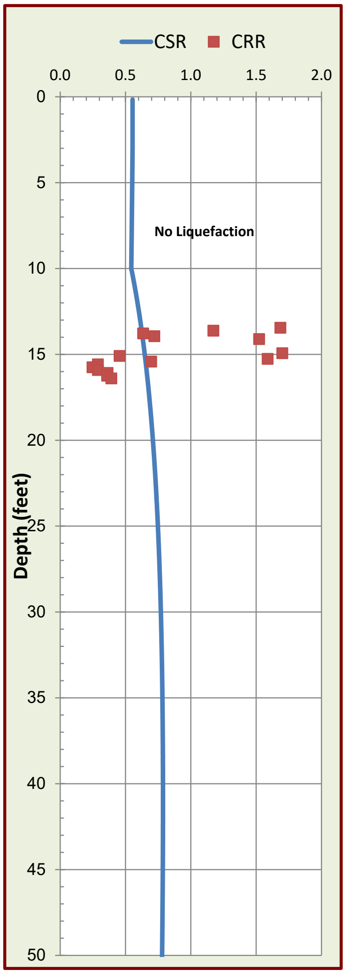
EXPECTED RANGE OF DISPLACEMENT

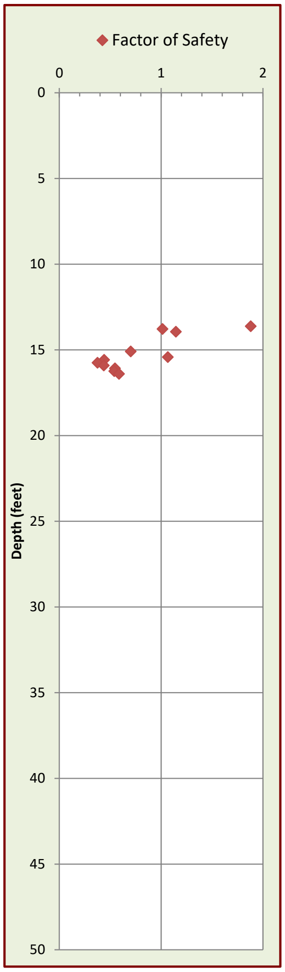
0.0 **to** **0.0** **feet**

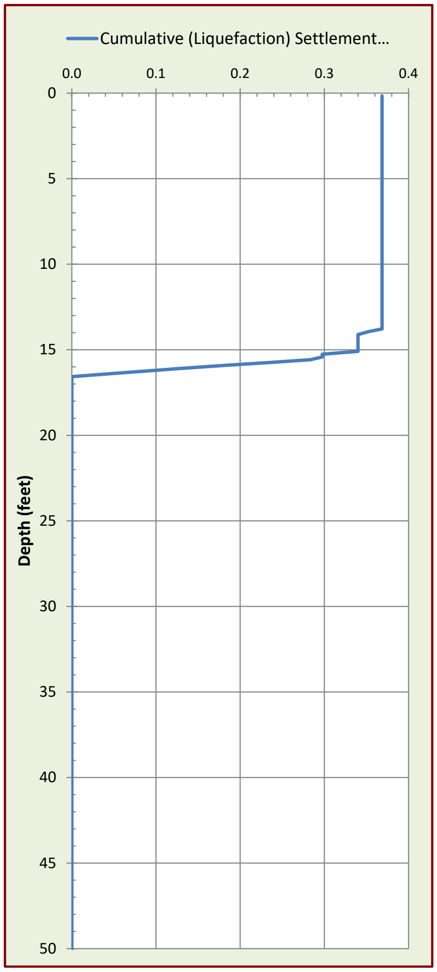
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

FIGURE **4B**

CPT NO. **2**

PROJECT/CPT DATA

Project Title	**25450 Clawiter Road**
Project No.	**916-2-1**
Project Manager	**SCO**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.58**	
PGA (Amax)	**0.852**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**18**
Design Water Depth (feet)	**10**
Ave. Unit Weight Above GW (pcf)	**121**
Ave. Unit Weight Below GW (pcf)	**121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.24 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 1000.0

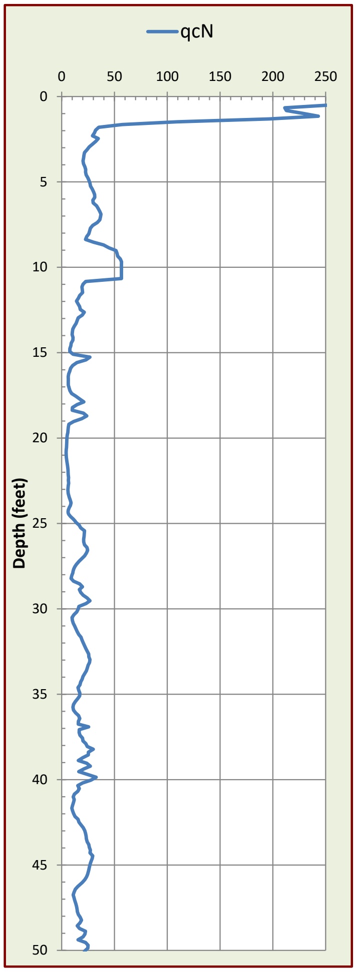
LDI^1**Corrected for Distance** 0.00 (4 < L/H < 40)

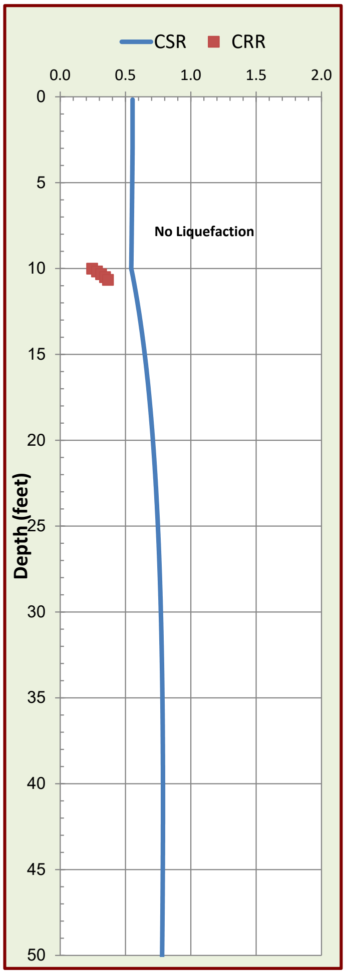
EXPECTED RANGE OF DISPLACEMENT

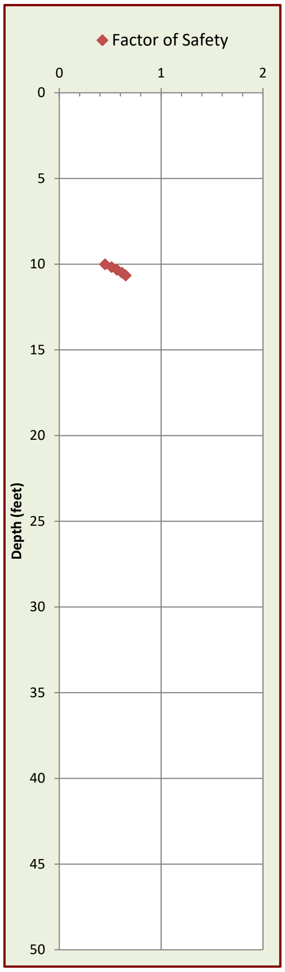
0.0 **to** **0.0** **feet**

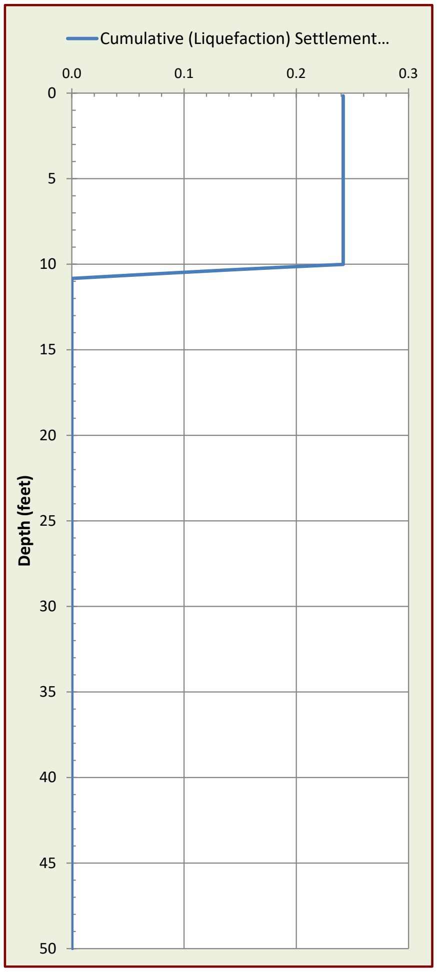
^{1}Not Valid for L/H Values < 4 and > 40.

^{2}LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title **25450 Clawiter Road**

Project No. **916-2-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.58**

PGA (Amax) **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **11.2**

Design Water Depth (feet) **10**

Ave. Unit Weight Above GW (pcf) **121**

Ave. Unit Weight Below GW (pcf) **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.33 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.40 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.7 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 1000.0

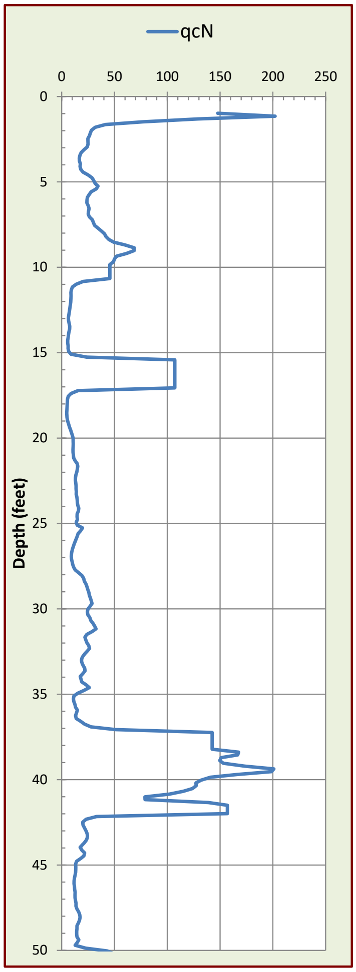
LDI[1] **Corrected for Distance** 0.00 (4 < L/H < 40)

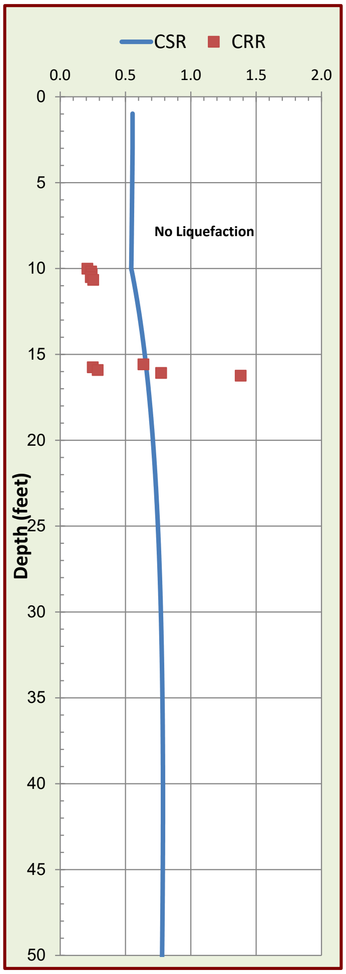
EXPECTED RANGE OF DISPLACEMENT

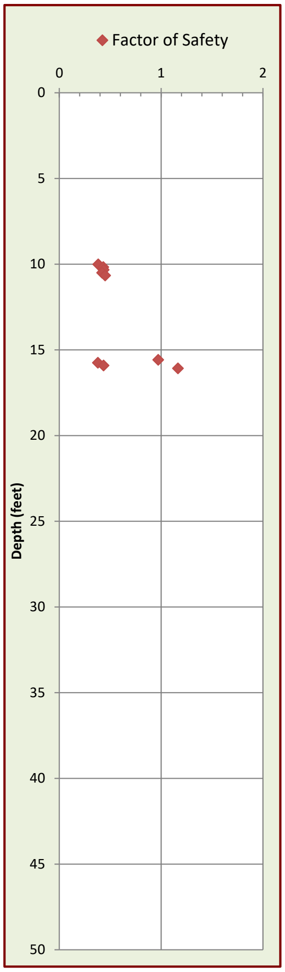
0.0 **to** **0.0** **feet**

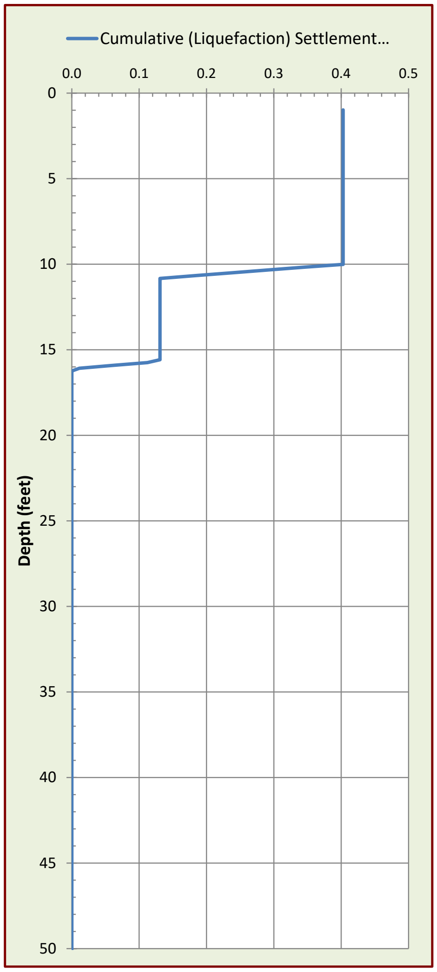
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

FIGURE **4D**

CPT NO. **4**

PROJECT/CPT DATA

Project Title **25450 Clawiter Road**

Project No. **916-2-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.58**

PGA (Amax) **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **8.9**

Design Water Depth (feet) **10**

Ave. Unit Weight Above GW (pcf) **121**

Ave. Unit Weight Below GW (pcf) **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.10 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.52 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.6 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 1000.0

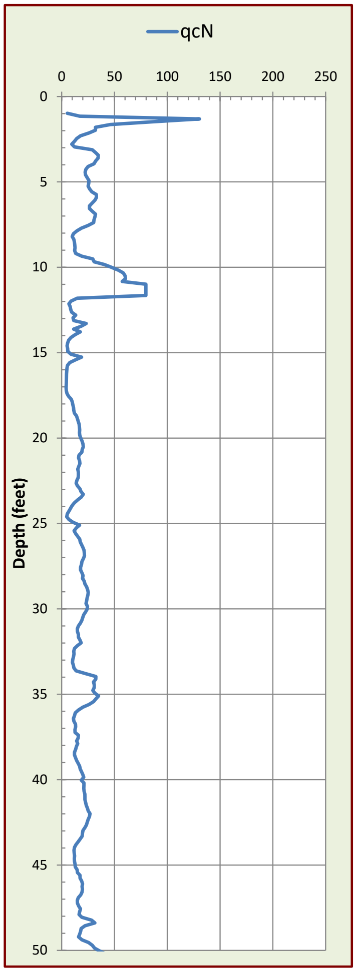
LDI^1 Corrected for Distance 0.00 (4 < L/H < 40)

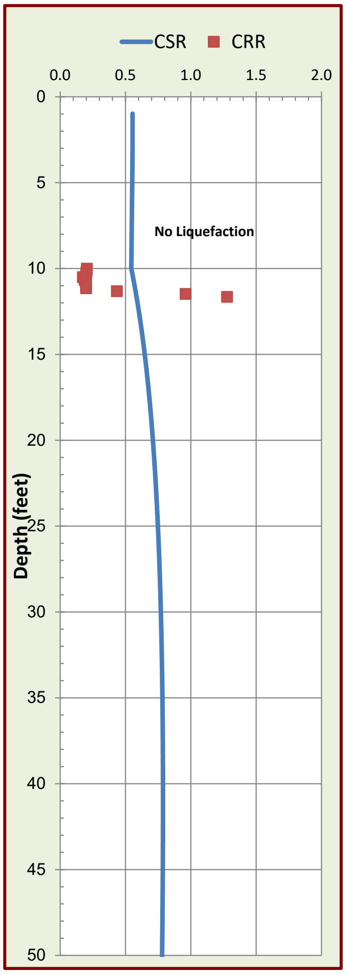
EXPECTED RANGE OF DISPLACEMENT

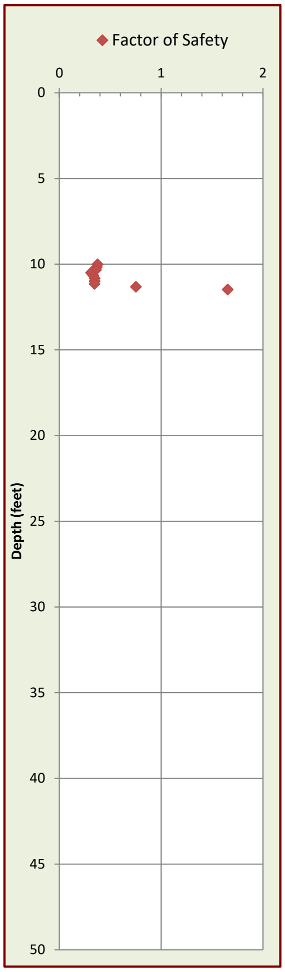
0.0 **to** **0.0** **feet**

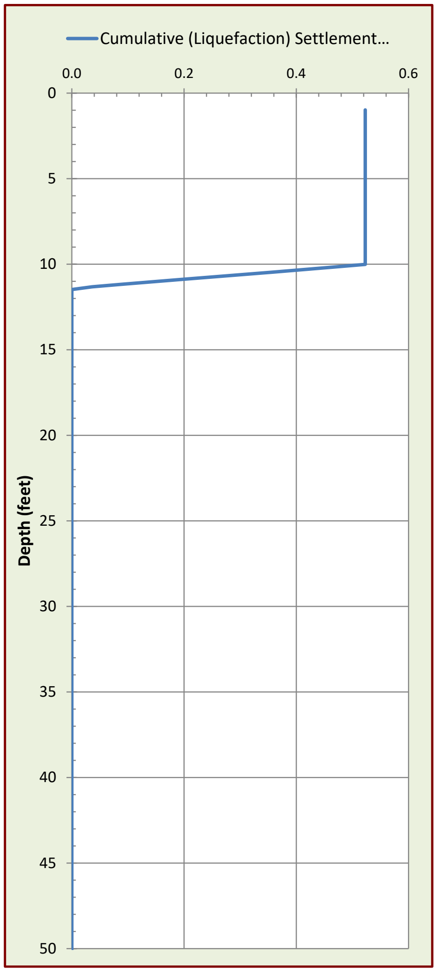
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









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EARTH GROUP

FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title: **25450 Clawiter Road**

Project No.: **916-2-1**

Project Manager: **SCO**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**

Earthquake Magnitude (Mw): **7.58**

PGA (Amax): **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **18.5**

Design Water Depth (feet): **10**

Ave. Unit Weight Above GW (pcf): **121**

Ave. Unit Weight Below GW (pcf): **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.06 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 1000.0

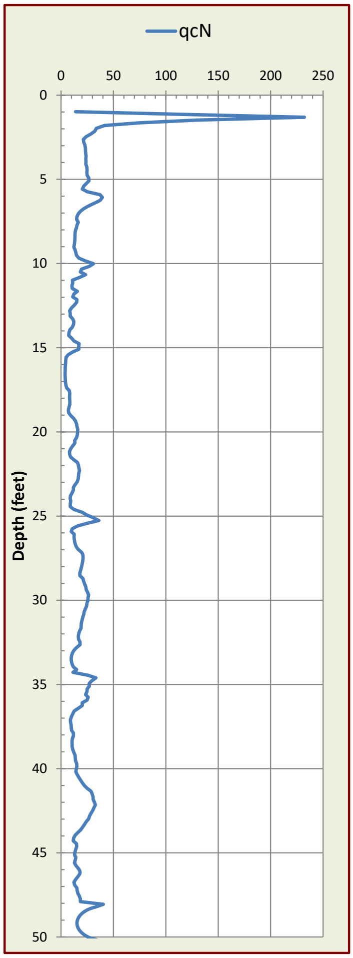
LDI[1] **Corrected for Distance** 0.00 (4 < L/H < 40)

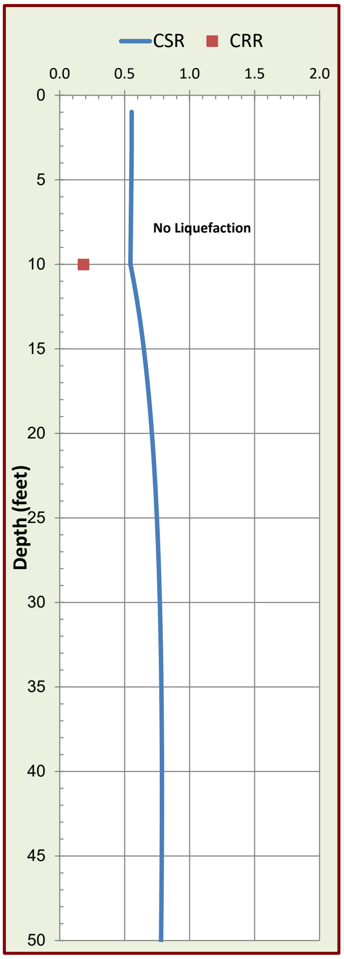
EXPECTED RANGE OF DISPLACEMENT

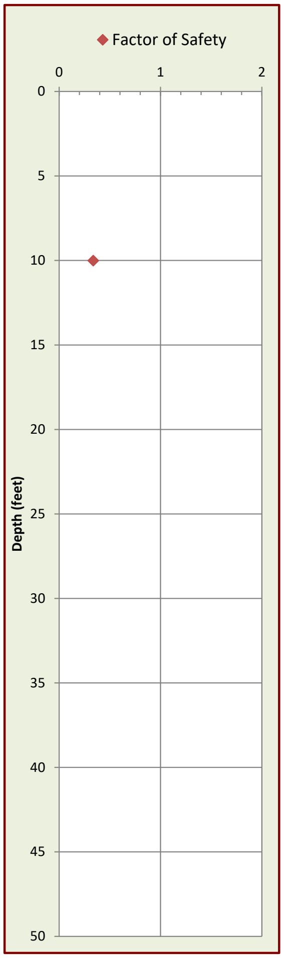
0.0 **to** **0.0** **feet**

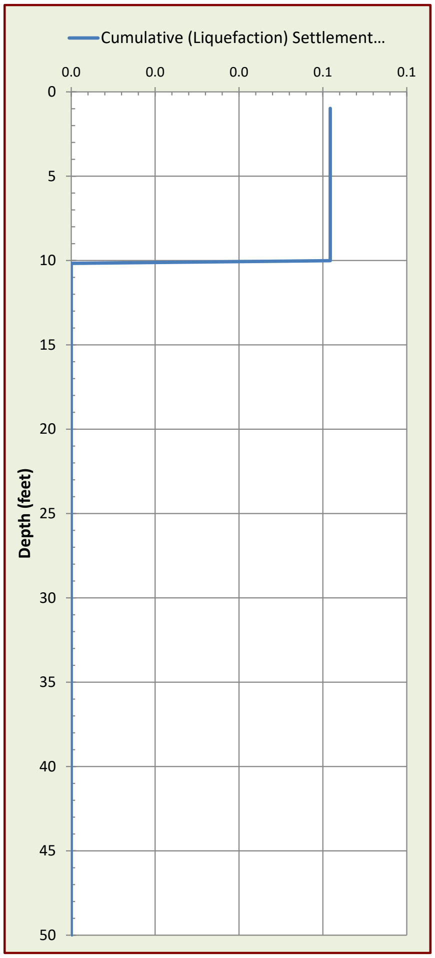
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

FIGURE | **4F**

CPT NO. | **6**

PROJECT/CPT DATA

Project Title **25450 Clawiter Road**

Project No. **916-2-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.58**

PGA (Amax) **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **16.7**

Design Water Depth (feet) **10**

Ave. Unit Weight Above GW (pcf) **121**

Ave. Unit Weight Below GW (pcf) **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.78 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.8 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 1000.0

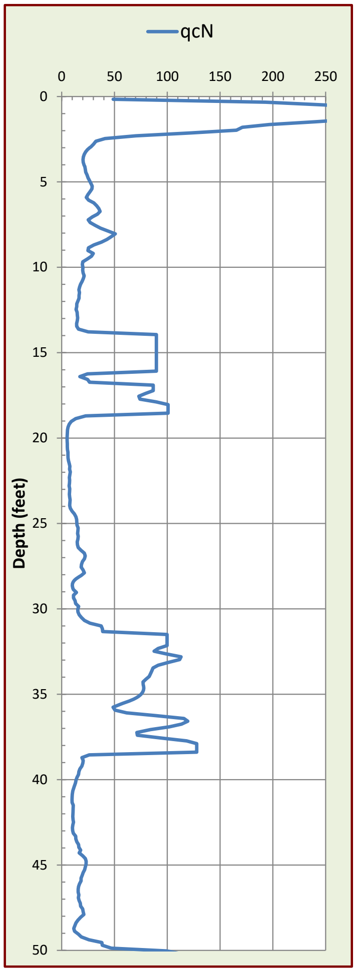
LDI^1**$_{\text{Corrected for Distance}}$** 0.00 (4 < L/H < 40)

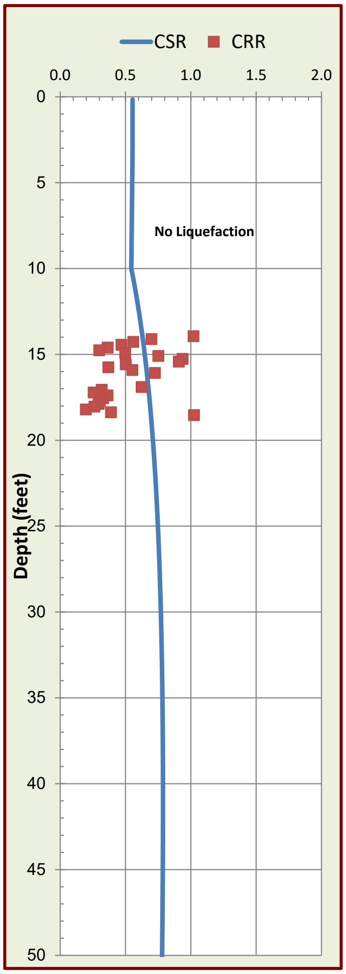
EXPECTED RANGE OF DISPLACEMENT

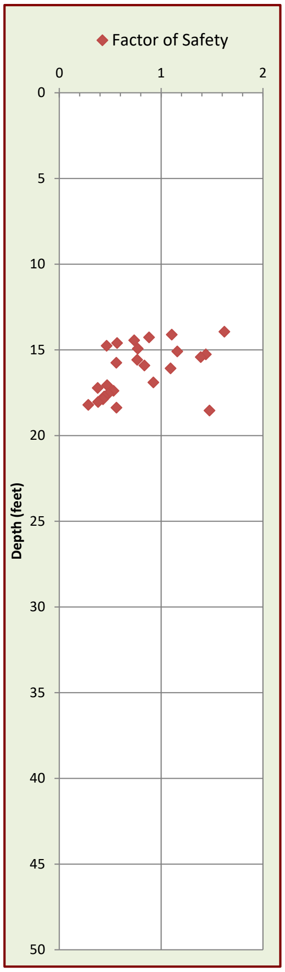
0.0 **to** **0.0** **feet**

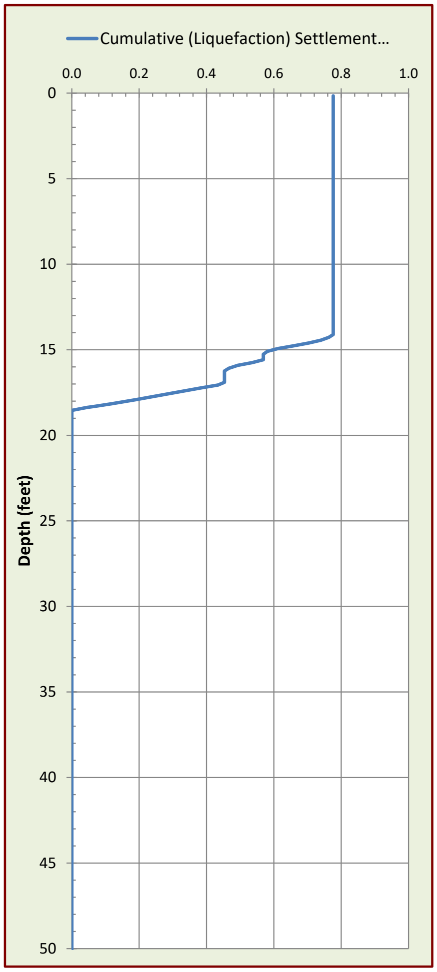
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

FIGURE **4G**

CPT NO. **7**

PROJECT/CPT DATA

Project Title: **25450 Clawiter Road**
Project No.: **916-2-1**
Project Manager: **SCO**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**
Earthquake Magnitude (Mw): **7.58**
PGA (Amax): **0.852** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **21.3**
Design Water Depth (feet): **10**
Ave. Unit Weight Above GW (pcf): **121**
Ave. Unit Weight Below GW (pcf): **121**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 10 **FEET**
0.00 **(Inches)**
LIQUEFACTION SETTLEMENT FROM 50 **FEET**
0.70 **(Inches)**

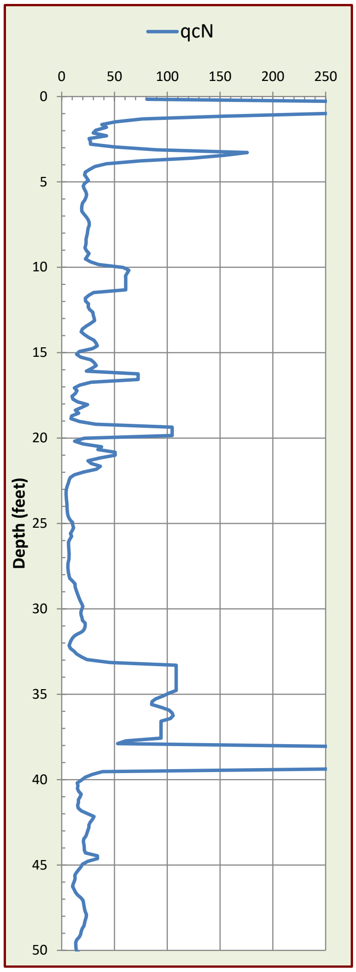
TOTAL SEISMIC SETTLEMENT 0.7 **INCHES**

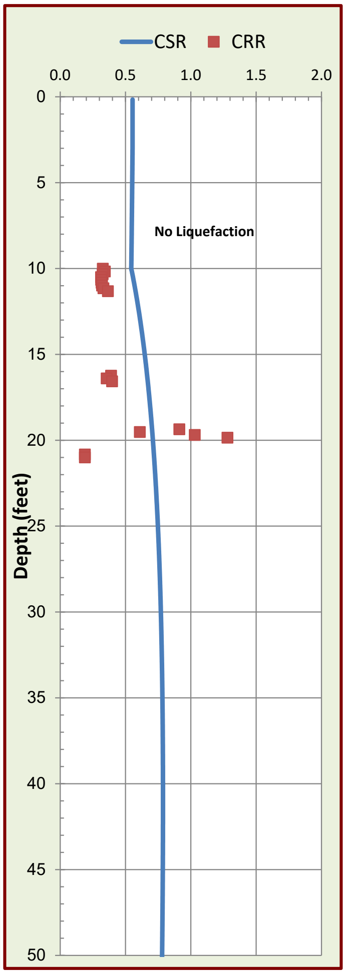
POTENTIAL LATERAL DISPLACEMENT

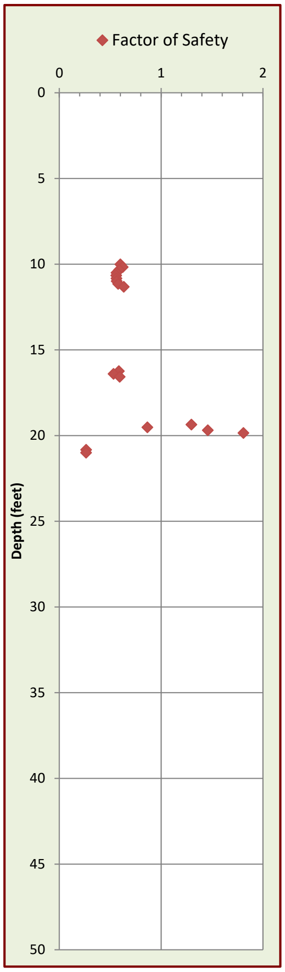
LDI^2 0.00 **L/H** 1000.0
LDI^1**Corrected for Distance** 0.00 (4 < L/H < 40)

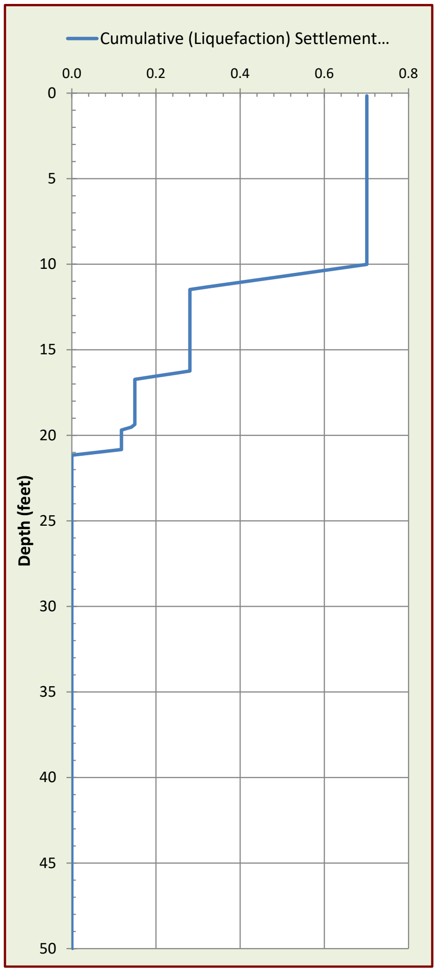
EXPECTED RANGE OF DISPLACEMENT
0.0 **to** **0.0** **feet**

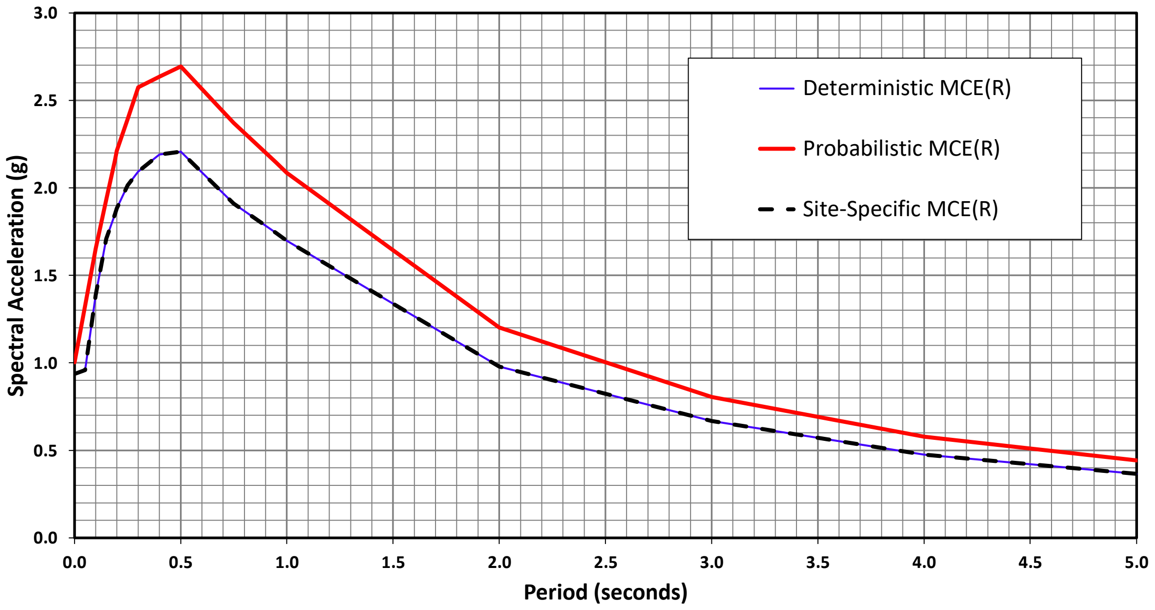
[1] Not Valid for L/H Values < 4 and > 40.
[2] LDI Values Only Summed to 2H Below Grade.











The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

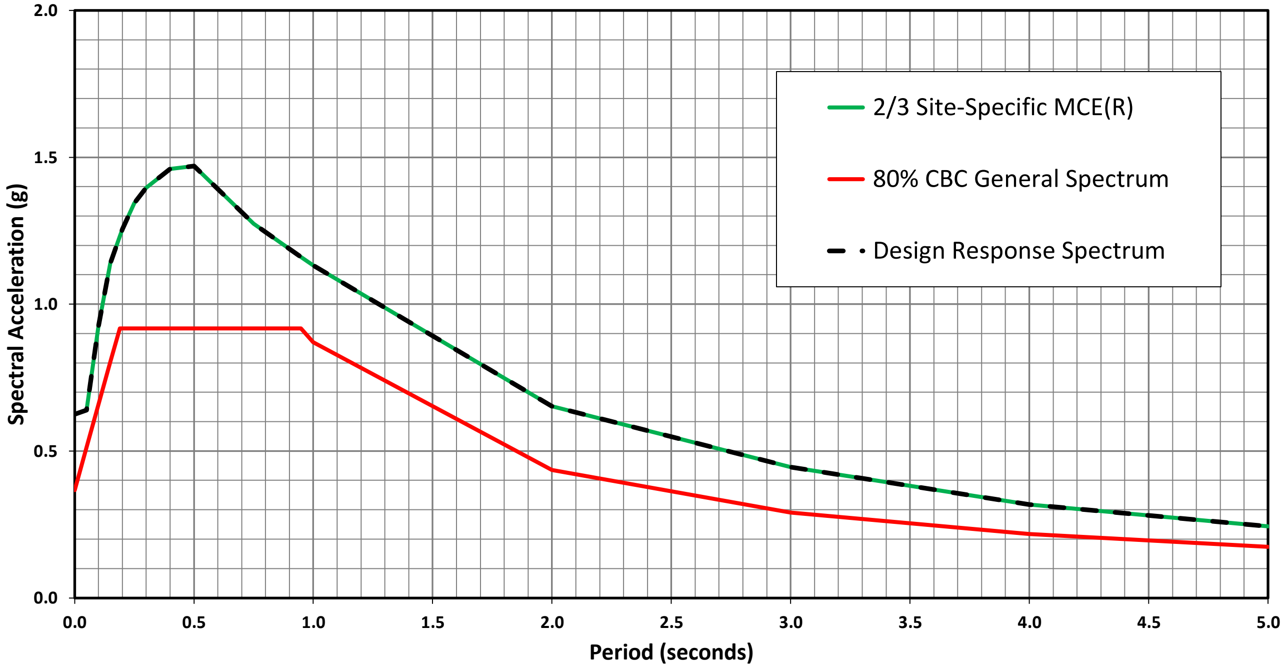
- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.938
0.05	0.958
0.10	1.386
0.15	1.709
0.19	1.848
0.20	1.882
0.25	2.015
0.30	2.094
0.40	2.190
0.50	2.207
0.75	1.910
0.95	1.741
1.00	1.698
2.00	0.979
3.00	0.668
4.00	0.476
5.00	0.366

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP	MCE_R RESPONSE SPECTRA	FIGURE 5	
	25500 Clawiter Industrial GI 25500 Clawiter Industrial Hayward, CA	PROJECT NO. 916-2-1	
		August 18, 2020	SCO



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.625
0.05	0.639
0.10	0.924
0.15	1.139
0.19	1.232
0.20	1.255
0.25	1.343
0.30	1.396
0.40	1.460
0.50	1.471
0.75	1.273
0.95	1.161
1.00	1.132
2.00	0.653
3.00	0.445
4.00	0.317
5.00	0.244

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	229
Site Latitude (degrees)	37.633825
Site Longitude (degrees)	-122.117991
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.85

Design Acceleration Parameters[1]	
S_{DS}	1.324
S_{D1}	1.335
S_{MS}	1.986
S_{M1}	2.003

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA

FIGURE 6

25500 Clawiter Industrial GI
25500 Clawiter Industrial
Hayward, CA

PROJECT NO. 916-2-1

August 18, 2020

SCO

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. Seven 8-inch-diameter exploratory borings were drilled on August 4 and 5, 2020 to depths ranging from 30 to 42½ feet. Seven CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on July 30, 2020, to depths ranging from approximately 50 to 150 feet. The approximate locations of exploratory borings and CPTs 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 and CPT locations were approximated using existing site boundaries and other site features as references. Boring and CPT elevations were not determined. The locations 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. 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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	⁘	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	⁘	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	○	Gravelly Silt
⁘	Poorly-Graded Gravelly Sand	■	Asphalt
⁂	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

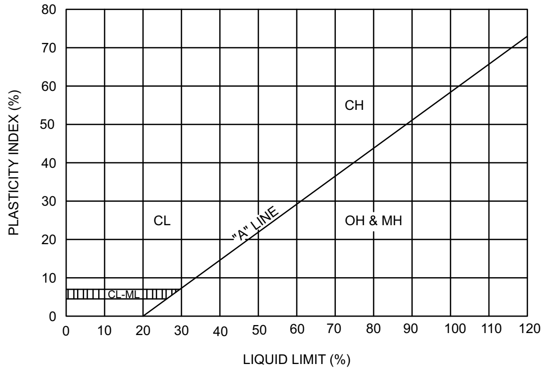
SAMPLER TYPES

☒	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 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA
DATE STARTED 8/4/20 **DATE COMPLETED** 8/4/20
GROUND ELEVATION **BORING DEPTH** 35 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 13 ft.
NOTES
▼ **AT END OF DRILLING** 13 ft.

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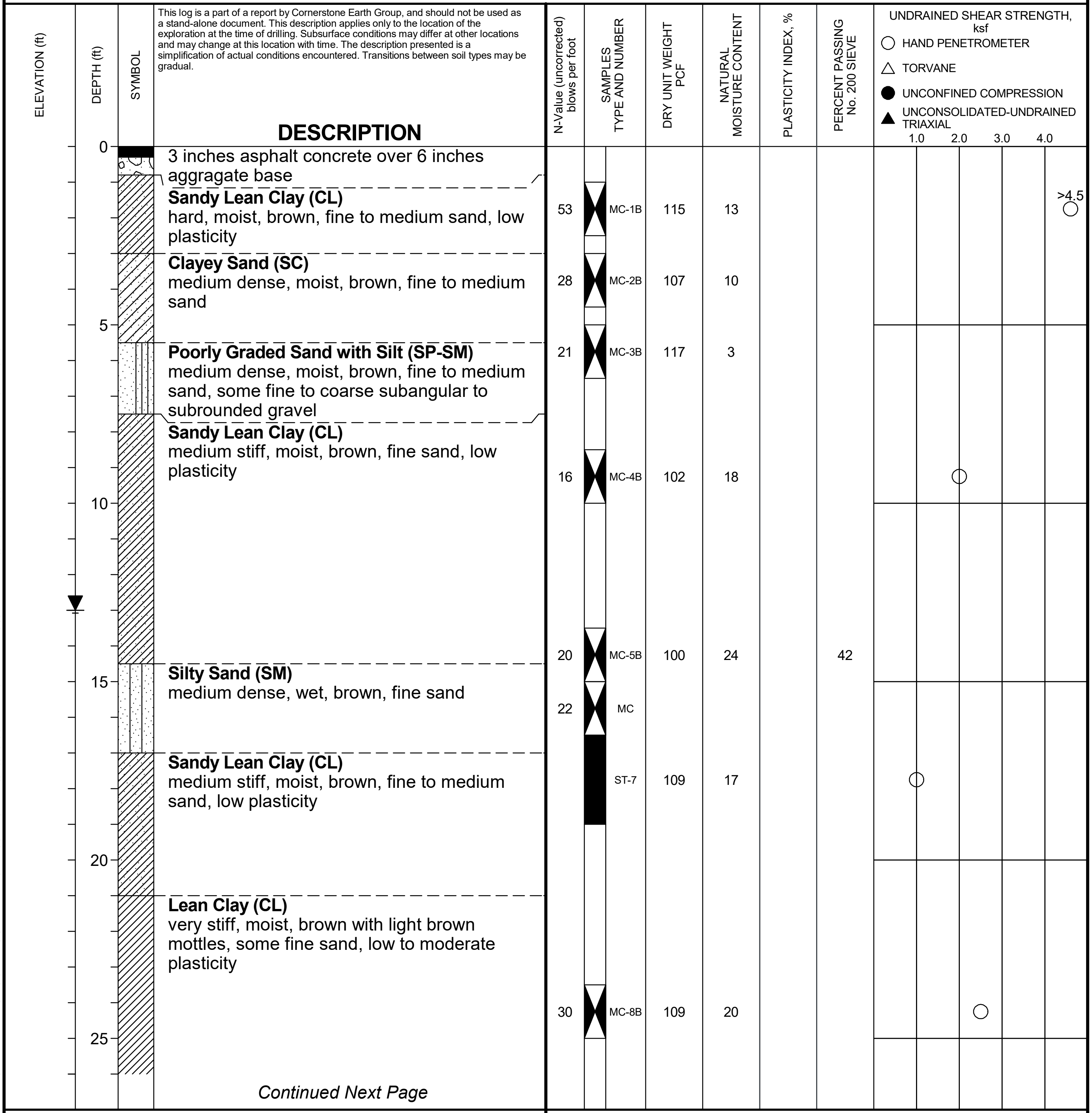

DESCRIPTION

3 inches asphalt concrete over 6 inches aggragate base

Sandy Lean Clay (CL)
hard, moist, brown, fine to medium sand, low plasticity

Clayey Sand (SC)
medium dense, moist, brown, fine to medium sand

Poorly Graded Sand with Silt (SP-SM)
medium dense, moist, brown, fine to medium sand, some fine to coarse subangular to subrounded gravel

Sandy Lean Clay (CL)
medium stiff, moist, brown, fine sand, low plasticity

Silty Sand (SM)
medium dense, wet, brown, fine sand

Sandy Lean Clay (CL)
medium stiff, moist, brown, fine to medium sand, low plasticity

Lean Clay (CL)
very stiff, moist, brown with light brown mottles, some fine sand, low to moderate plasticity

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
53	MC-1B	115	13		
28	MC-2B	107	10		
21	MC-3B	117	3		
16	MC-4B	102	18		
20	MC-5B	100	24		42
22	MC				
	ST-7	109	17		
30	MC-8B	109	20		

Continued Next Page

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA

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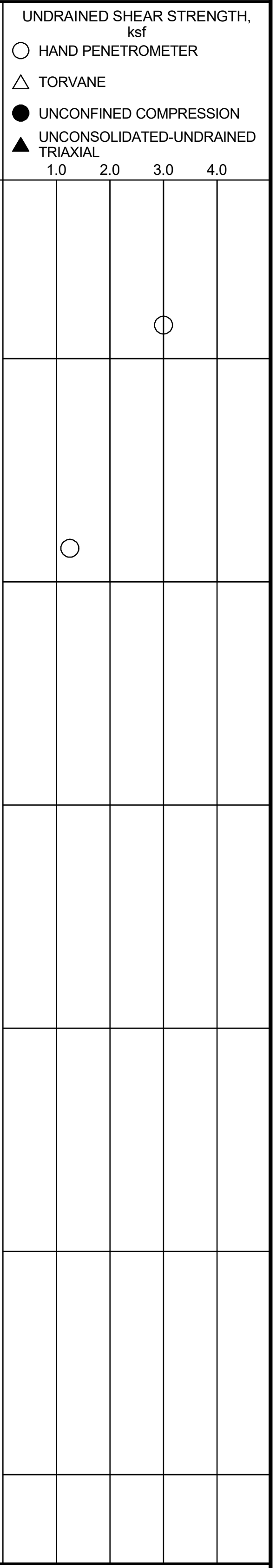
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
		▨	**Lean Clay (CL)** very stiff, moist, brown with light brown mottles, some fine sand, low to moderate plasticity	52	MC-9B	113	17		
	30	▨							
		▨	**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, low to moderate plasticity	45	MC-10B	107	23		
	35		Bottom of Boring at 35.0 feet.						
	40								
	45								
	50								
	55								



BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME 25550 Clawiter Road

PROJECT NUMBER 916-2-1

PROJECT LOCATION Hayward, CA

DATE STARTED 8/4/20 **DATE COMPLETED** 8/4/20

GROUND ELEVATION **BORING DEPTH** 30 ft.

DRILLING CONTRACTOR Exploartion Geoservices

LATITUDE **LONGITUDE**

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

GROUND WATER LEVELS:

LOGGED BY RAH

▽ **AT TIME OF DRILLING** 18 ft.

NOTES

▼ **AT END OF DRILLING** 18 ft.

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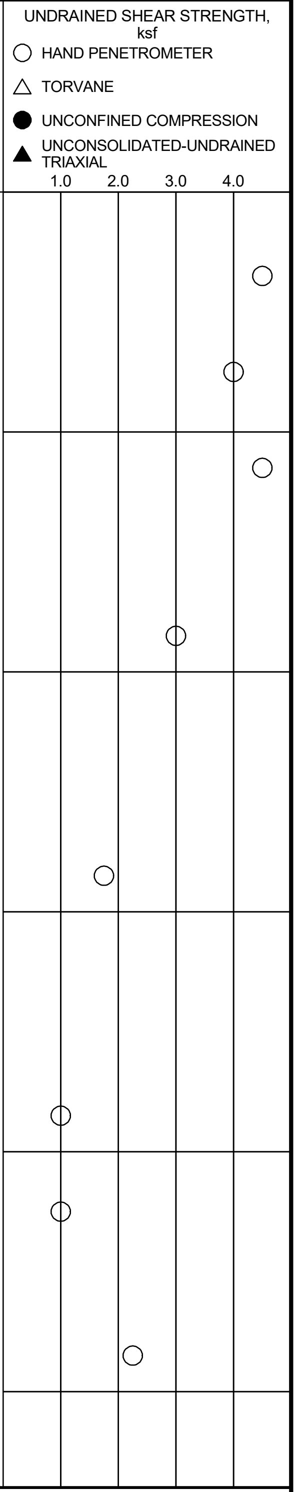
DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
0	3 inches asphalt concrete over 6 inches aggregate base						
	Lean Clay (CL) hard, moist, dark brown to brown, trace sand, moderate plasticity Liquid Limit = 46 Plastic Limit = 17	50	MC-1B	116	16		
		31	MC-2B	103	19	29	
5		56	MC-3B	109	18		
	Sandy Lean Clay (Cl) very stiff, moist, brown, fine to medium sand, low plasticity	40	MC-4	109	18		
10	**Clayey Sand (SC)** medium dense, moist, brown, fine sand	23	MC-5B	116	11		
	Silty Clay with Sand (CL-ML) stiff, moist, brown, fine sand, low plasticity	28	MC-6B	104	19		
15							
	Lean Clay (CL) medium stiff, moist, brown, some fine sand, low to moderate plasticity	14	MC-7B	94	27		
20			ST				
	Lean Clay (CL) very stiff, moist, brown, some fine sand, moderate plasticity	17	MC-9B	103	23		
25							



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CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 8/19/20 09:12 - P:\DRAFTING\GINT FILES\916-2-1 25550 CLAWITER ROAD.GPJ

BORING NUMBER EB-2

PROJECT NAME 25550 Clawiter Road

PROJECT NUMBER 916-2-1

PROJECT LOCATION Hayward, CA

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 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
		▨	**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity	9	MC-10B	94	28			○
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

BORING NUMBER EB-3

PAGE 1 OF 2

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA
DATE STARTED 8/4/20 **DATE COMPLETED** 8/4/20
GROUND ELEVATION **BORING DEPTH** 41.5 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 19 ft.
NOTES
▼ **AT END OF DRILLING** 19 ft.

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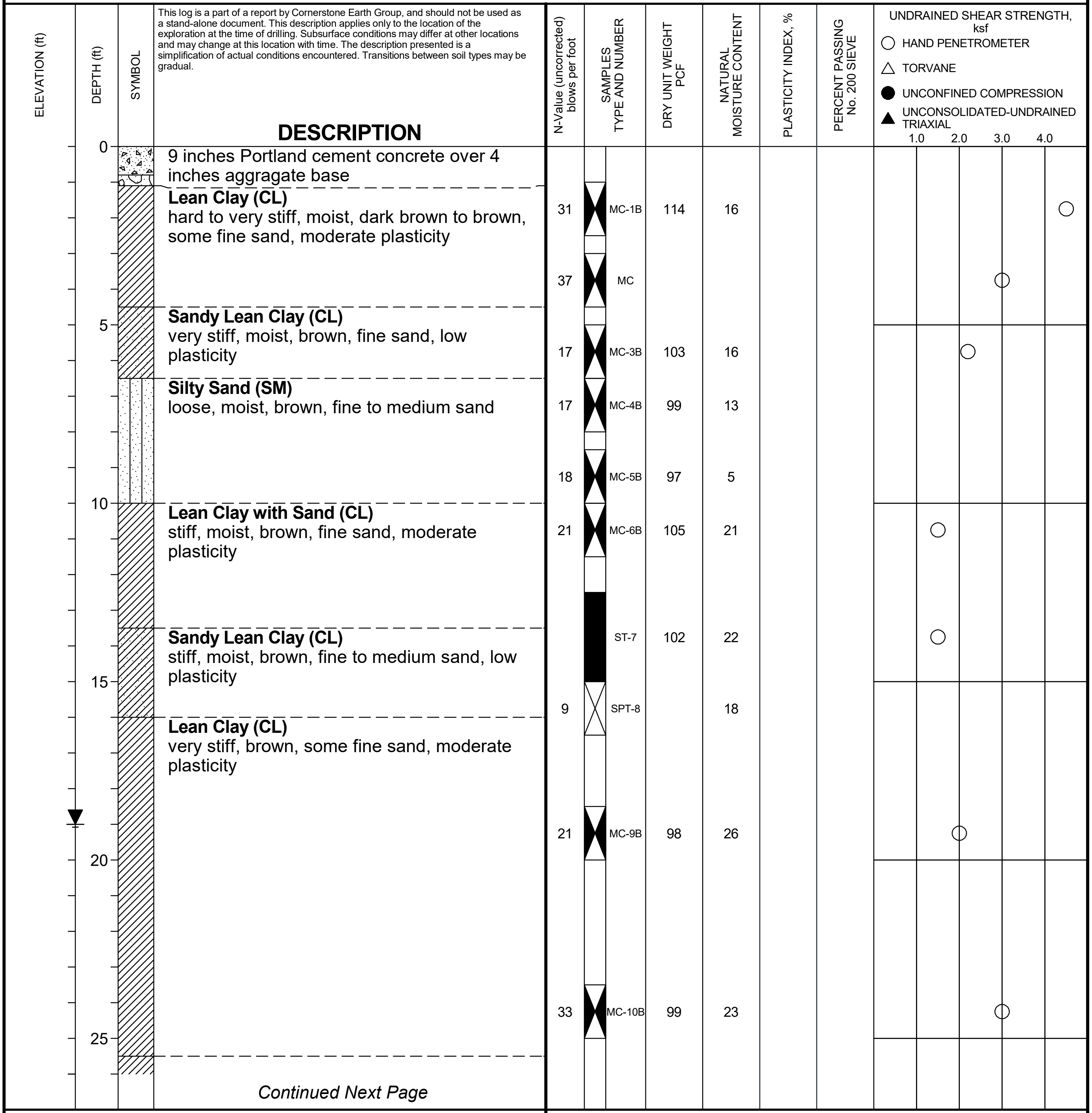

DESCRIPTION

9 inches Portland cement concrete over 4 inches aggragate base

Lean Clay (CL)
hard to very stiff, moist, dark brown to brown, some fine sand, moderate plasticity

Sandy Lean Clay (CL)
very stiff, moist, brown, fine sand, low plasticity

Silty Sand (SM)
loose, moist, brown, fine to medium sand

Lean Clay with Sand (CL)
stiff, moist, brown, fine sand, moderate plasticity

Sandy Lean Clay (CL)
stiff, moist, brown, fine to medium sand, low plasticity

Lean Clay (CL)
very stiff, brown, some fine sand, moderate plasticity

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	114	16		
37	MC				
17	MC-3B	103	16		
17	MC-4B	99	13		
18	MC-5B	97	5		
21	MC-6B	105	21		
	ST-7	102	22		
9	SPT-8		18		
21	MC-9B	98	26		
33	MC-10B	99	23		

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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, 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
		▨	**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity	52	MC-11B	104	21			○
		▨	**Lean Clay (CL)** hard, brown, some fine sand, moderate plasticity	71	MC-12B	107	22			○
	30									
		▨	**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, low plasticity	42	MC-13B	109	18			○
	35									
			Poorly Graded Sand with Silt (SP) medium dense, wet, brown fine to coarse sand	28	SPT-14		15		8	
	40			29	SPT					
			Bottom of Boring at 41.5 feet.							
	45									
	50									
	55									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-4

PAGE 1 OF 2

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA
DATE STARTED 8/5/20 **DATE COMPLETED** 8/5/20
GROUND ELEVATION **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
∇ **AT TIME OF DRILLING** 18 ft.
NOTES
▼ **AT END OF DRILLING** 18 ft.

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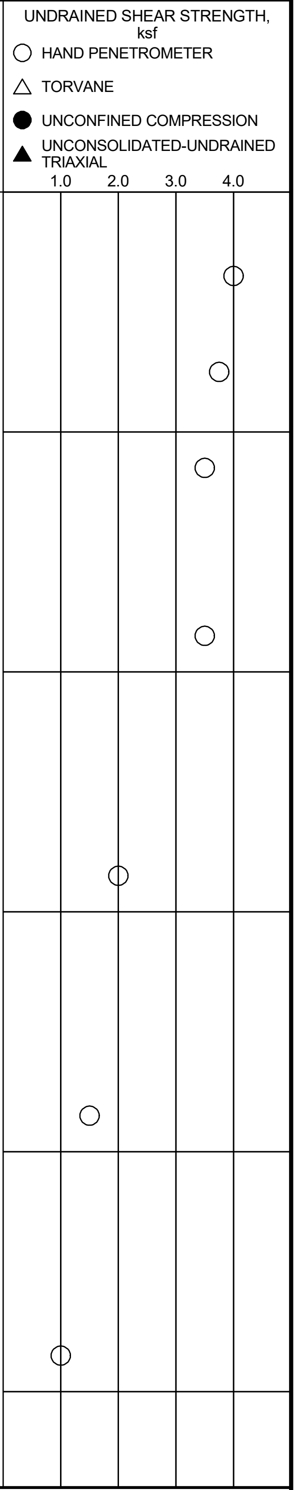
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0		10 inches Portland cement concrete over 4 inches aggragate base						
			Sandy Lean Clay (CL) [Fill] very stiff, moist, brown to dark brown with gray mottled, fine to medium sand, fine to medium subangular gravel, low plasticity	45	MC-1B	123	12		
				43	MC-2B	122	13		
	5			43	MC-3B	121	15		
				40	MC-4C	128	13		
	10								
			Lean Clay with Sand (CL) [Fill] stiff, moist, gray and brown mottled, fine sand, moderate plasticity						
				26	MC-5B	107	19		
	15								
			Lean Clay (CL) stiff, moist, brown, some fine sand, moderate plasticity						
		▼							
				23	MC-6B	97	26		
	20								
			Lean Clay with Sand (CL) stiff, moist, brown, fine sand, moderate plasticity						
				16	MC-7B	111	21		
	25								



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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-4

PAGE 2 OF 2

PROJECT NAME 25550 Clawiter Road

PROJECT NUMBER 916-2-1

PROJECT LOCATION Hayward, 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)
		▨	**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, moderate plasticity							
	30	▨	**Lean Clay (CL)** stiff, moist, brown, some fine sand, moderate plasticity	32	MC-8B	109	19			○ 2.0
			Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

BORING NUMBER EB-5

PAGE 1 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA

DATE STARTED 8/5/20 **DATE COMPLETED** 8/5/20
GROUND ELEVATION ______ **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE ______ **LONGITUDE** ______
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 13 ft.
NOTES ______
▼ **AT END OF DRILLING** 13 ft.

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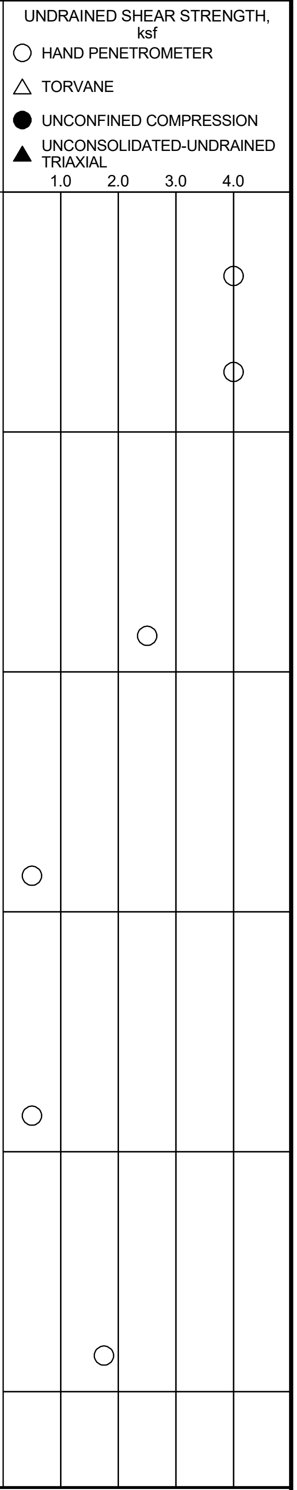
ELEVATION (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	10 inches Portland cement concrete over 4 inches aggragate base						
	Lean Clay (CL) hard, moist, dark brown to brown, some fine sand gravel, moderate plasticity	40	MC-1B	109	17		
		45	MC-2B	116	15		
	Clayey Sand (SC) medium dense, brown, moist, fine to medium sand	33	MC-3B	116	10		
	Lean Clay with Sand (CL) very stiff, brown, moist, fine to medium sand, trace coarse sand, low to moderate plasticity	26	MC-4B	108	19		
	Sandy Lean Clay (CL) medium stiff, moist, brown, fine sand, low plasticity	25	MC-5B	97	25		
			ST				
	Poorly Graded Sand (SP) medium dense, moist, brown, fine to coarse sand	19	MC-6C	98	24		
	Lean Clay (CL) medium stiff, moist, brown, some fine sand, low plasticity						
	Lean Clay with Sand (CL) stiff, moist, brown, fine to medium sand, low plasticity	23	MC-7B	111	20		



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BORING NUMBER EB-5

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CORNERSTONE EARTH GROUP

PROJECT NAME 25550 Clawiter Road

PROJECT NUMBER 916-2-1

PROJECT LOCATION Hayward, 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
		▨	**Lean Clay with Sand (CL)** stiff, moist, brown, fine to medium sand, low plasticity							
		▨	becomes very stiff	46	MC-8B	113	18			○
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

BORING NUMBER EB-6

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA
DATE STARTED 8/5/20 **DATE COMPLETED** 8/5/20
GROUND ELEVATION **BORING DEPTH** 43 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 14 ft.
NOTES
▼ **AT END OF DRILLING** 14 ft.

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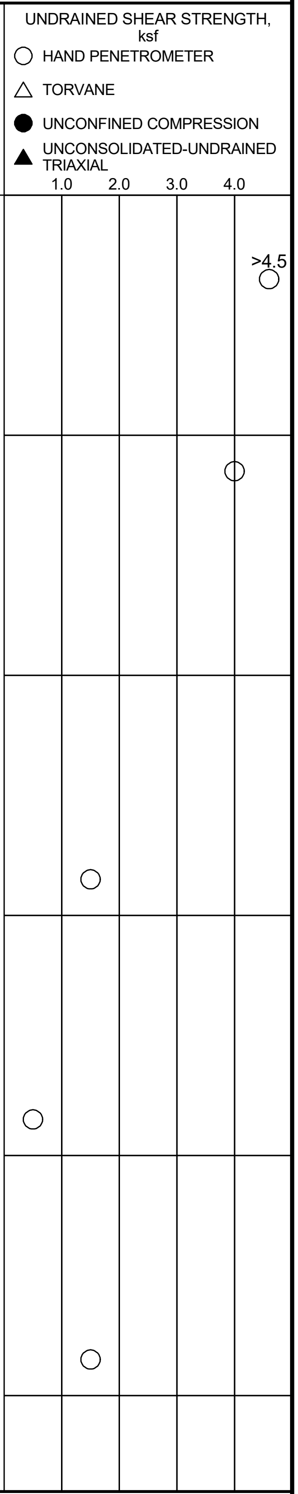
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown, fine to coarse sand, fine gravel						
		Lean Clay (CL) hard, moist, dark brown, some fine sand, moderate plasticity	38	MC-1B	104	19		
	5	**Lean Clay with Sand (CL)** hard, moist, brown, fine sand, moderate plasticity	42	MC-2B	113	14		
	10	**Clayey Sand (SC)** medium dense, moist, light brown, fine sand	23	MC-3B	105	12		
		Lean Clay with Sand (CL) stiff, moist, brown, fine sand, low plasticity	39	MC-4B	109	12		
	15	**Silty Sand (SM)** medium dense, wet, brown, fine to medium sand	11 14	SPT-5 SPT-6		24 24		23
	20	**Lean Clay with Sand (CL)** medium stiff, moist, brown, fine sand, low plasticity	17	MC-7B	100	24		
	25	**Lean Clay (CL)** stiff, moist, gray brown, trace fine sand, moderate plasticity	27	MC-8B	99	25		



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BORING NUMBER EB-6

PAGE 2 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME 25550 Clawiter Road

PROJECT NUMBER 916-2-1

PROJECT LOCATION Hayward, CA

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)	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 ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
		▨								
		▨	**Lean Clay with Sand** stiff, moist, brown, fine sand, low plasticity							
		▨		25	⧓ MC-9B	107	25			○
	30	░	**Silty Sand (SP-SM)** dense, moist, brown, fine to medium sand							
		░		45	⧖ SPT-10		20		36	
	35	░	**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown, fine to medium sand	29	⧖ SPT-11		23		12	
		░	**Silty Sand (SP-SM)** dense, moist, brown, fine to medium sand	43	⧖ SPT-12		20		13	
		░		59	○ NR					
	40	░								
		░		43	○ NR					
		▨	**Lean Clay with Sand (CL)** stiff, moist, brown, fine to medium sand, moderate plasticity Bottom of Boring at 43.0 feet.	30	⧖ SPT-13B		25			○
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-7

PAGE 1 OF 2

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA
DATE STARTED 8/5/20 **DATE COMPLETED** 8/5/20
GROUND ELEVATION **BORING DEPTH** 41.5 ft.
DRILLING CONTRACTOR Exploartion Geoservices
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
∇ **AT TIME OF DRILLING** 14.5 ft.
NOTES
▼ **AT END OF DRILLING** 14.5 ft.

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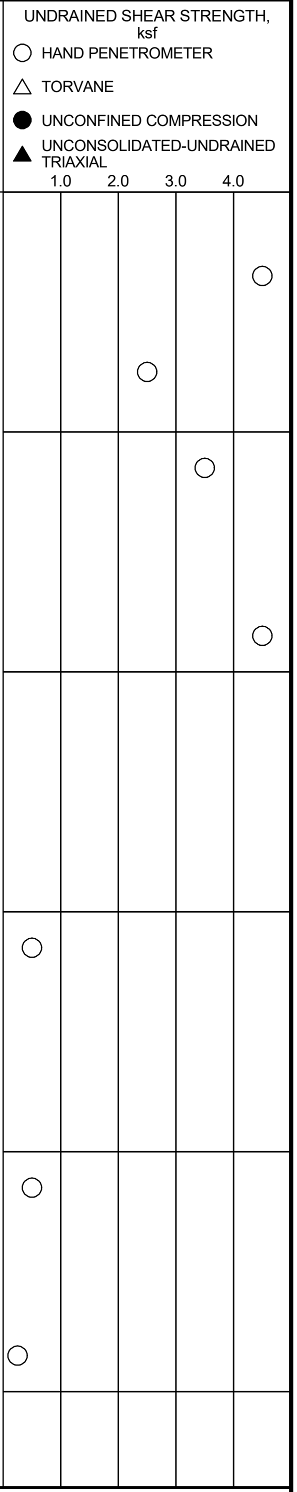
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0		2 inches gravel **Clayey Sand (SC) [Fill]** dense, moist, brown, fine to coarse sand, fine to coarse subangular gravel	63	MC-1B	119	14		
			Lean Clay (CL) very stiff, moist, dark brown to brown, some fine sand, low plasticity	54	MC-2B	100	22		
	5			45	MC-3B	104	22		
			Lean Clay with Sand (CL) hard, moist, brown, fine sand, low plasticity	31	MC-4B	113	16		
	10								
			Clayey Sand (SC) medium dense, moist, brown, fine sand Liquid Limit = 29 Plastic Limit = 14	31	MC-5	111	15	15	
	▼		**Sandy Lean Clay (CL)** medium stiff, moist, brown, fine sand, low plasticity	35	MC-6B	103	20		
	15			21	SPT-7		23		68
			Silty Sand (SM) medium dense, moist, brown, fine sand	37	MC-8B	109	20		
	20		**Sandy Lean Clay (CL)** medium stiff, moist, brown, fine sand, low plasticity	19	SPT				
			Lean Clay with Sand (CL) soft, moist, brown, fine sand, low plasticity	19	MC-10B	106	21		
	25								



Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-7

PROJECT NAME 25550 Clawiter Road
PROJECT NUMBER 916-2-1
PROJECT LOCATION Hayward, CA

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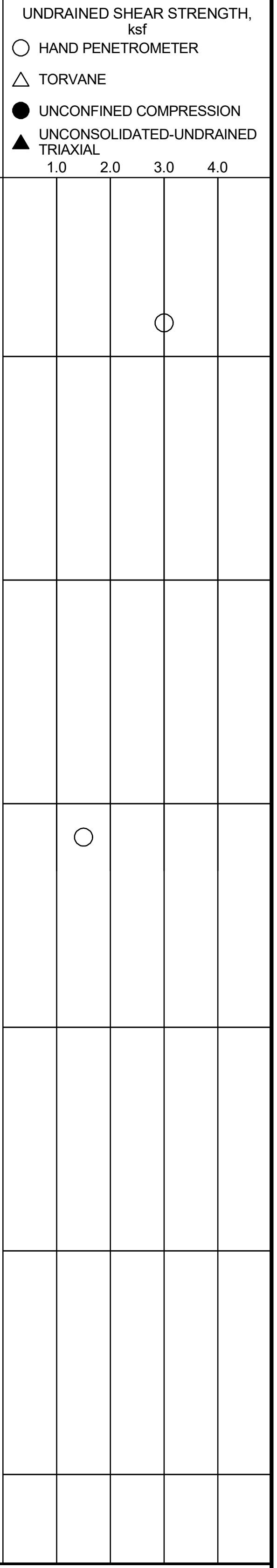
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
		▨	**Lean Clay (CL)** very stiff, moist, gray brown, some fine sand, moderate plasticity	38	MC-11B	109	19		
	30								
			Silty Sand (SM) dense, moist, brown, fine sand	40	SPT-12		20		34
	35			43	SPT-13		23		
			becomes very dense	50/6"	NR				
	40	▨	**Lean Clay with Sand (CL)** stiff, moist, fine sand, low to moderate plasticity		SPT-14		24		
			Bottom of Boring at 41.5 feet.						
	45								
	50								
	55								



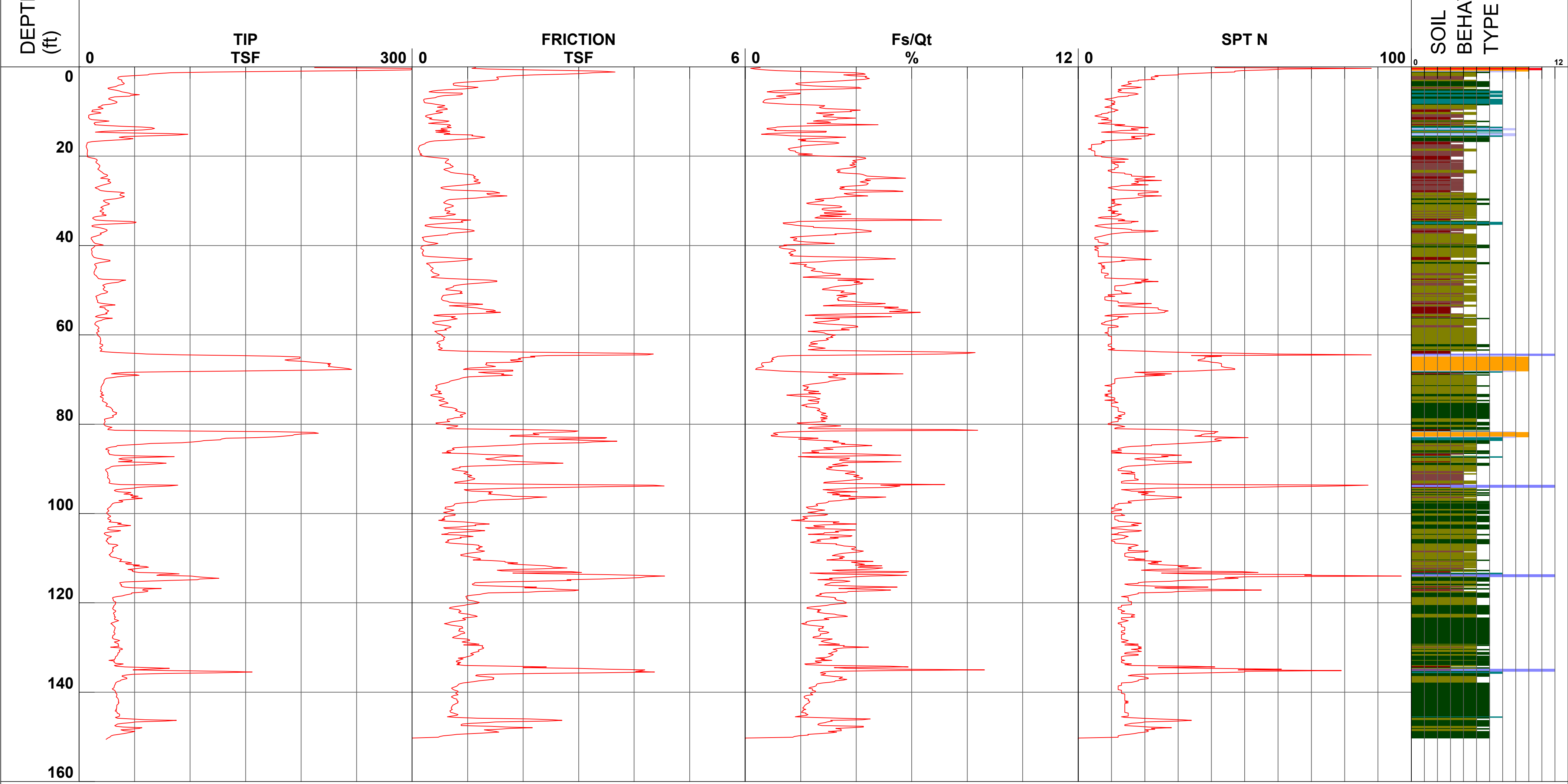
Middle Earth
GEO TESTING INC.

Cornerstone Earth Group

Project 25550 Clawiter Rd
Job Number 916-2-1
Hole Number CPT-01
EST GW Depth During Test 14.00 ft
Operator JM-ZG
Cone Number DDG1530
Date and Time 7/30/2020 7:30:26 AM
Filename SDF(942).cpt
GPS
Maximum Depth 150.59 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 (*)

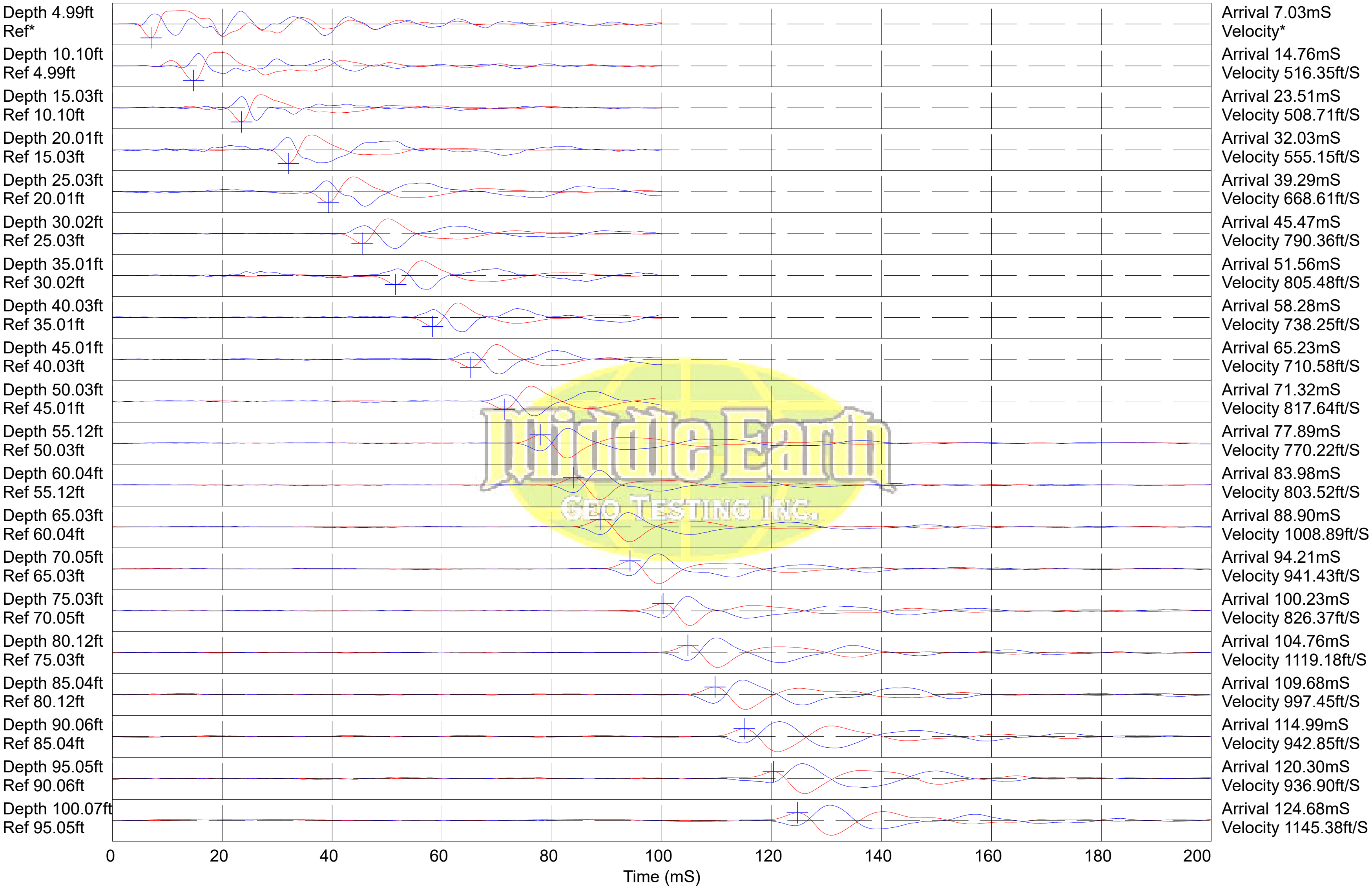
Cone Size 15cm squared

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

CPT-01

Cornerstone Earth Group

25550 Clawiter Rd



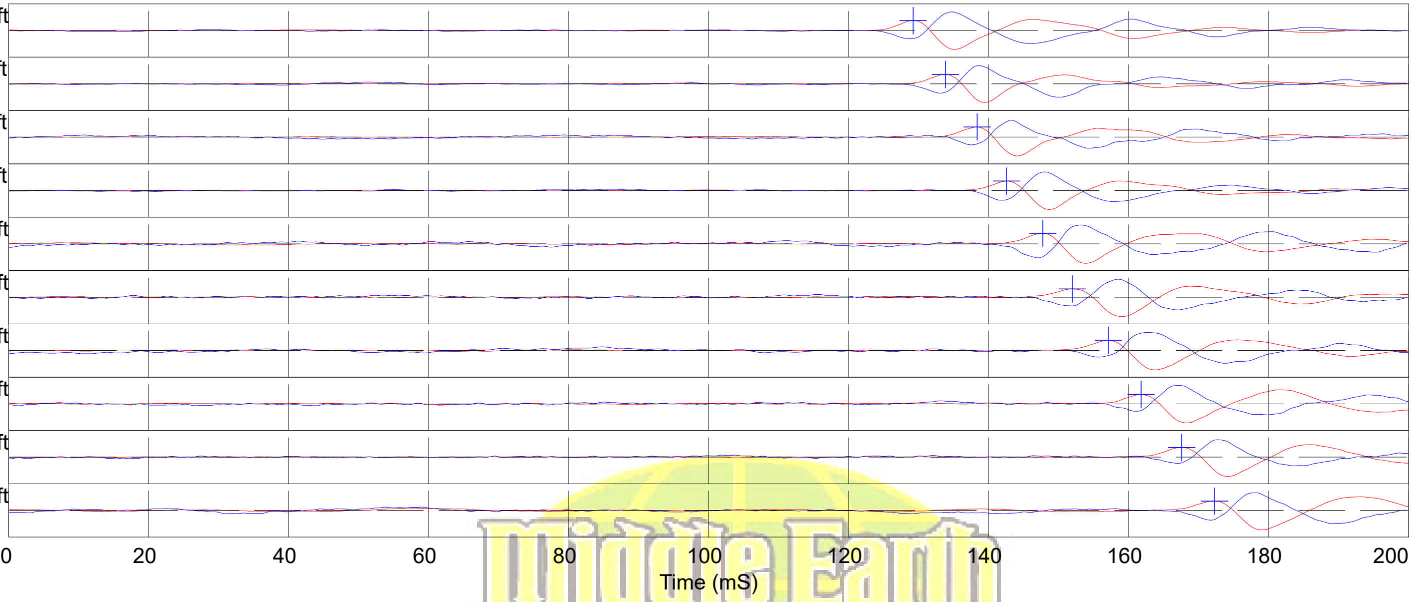
Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

CPT-01

Cornerstone Earth Group

25550 Clawiter Rd



Depth 105.05ft	Ref 100.07ft	Arrival 129.21mS	Velocity 1098.85ft/S
Depth 110.07ft	Ref 105.05ft	Arrival 133.82mS	Velocity 1087.49ft/S
Depth 114.99ft	Ref 110.07ft	Arrival 138.35mS	Velocity 1084.68ft/S
Depth 119.98ft	Ref 114.99ft	Arrival 142.57mS	Velocity 1180.70ft/S
Depth 125.00ft	Ref 119.98ft	Arrival 147.72mS	Velocity 972.47ft/S
Depth 130.02ft	Ref 125.00ft	Arrival 151.94mS	Velocity 1188.68ft/S
Depth 135.01ft	Ref 130.02ft	Arrival 157.10mS	Velocity 966.28ft/S
Depth 140.03ft	Ref 135.01ft	Arrival 161.79mS	Velocity 1069.97ft/S
Depth 145.01ft	Ref 140.03ft	Arrival 167.57mS	Velocity 861.93ft/S
Depth 150.03ft	Ref 145.01ft	Arrival 172.25mS	Velocity 1070.10ft/S

Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:

Cornerstone Earth Group

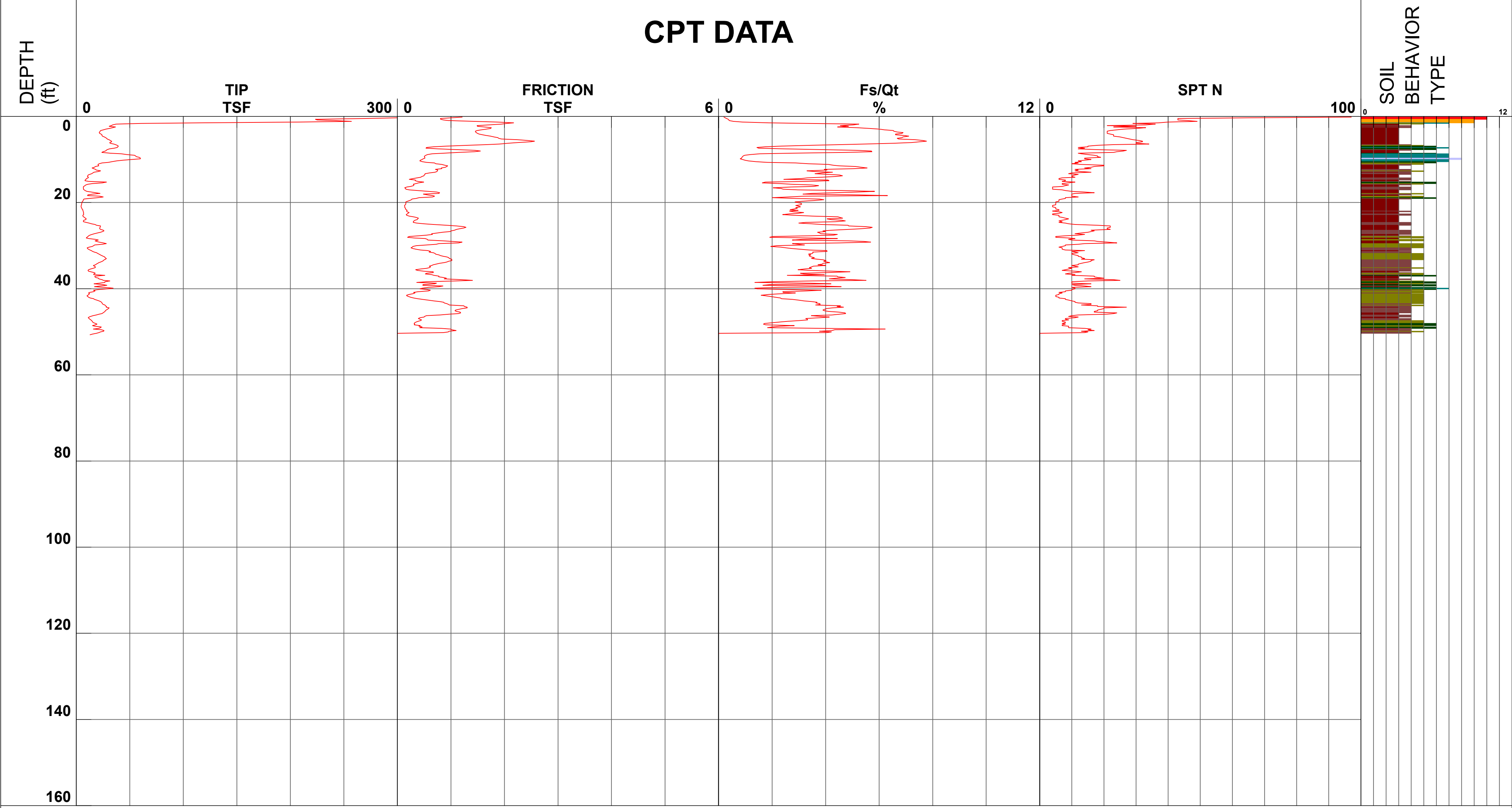
Project 25550 Clawiter Rd
Job Number 916-2-1
Hole Number CPT-02
EST GW Depth During Test 17.00 ft

Operator JM-ZG
Cone Number DDG1530
Date and Time 7/30/2020 11:16:01 AM

Filename SDF(943).cpt
GPS
Maximum Depth 50.69 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 15cm squared

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

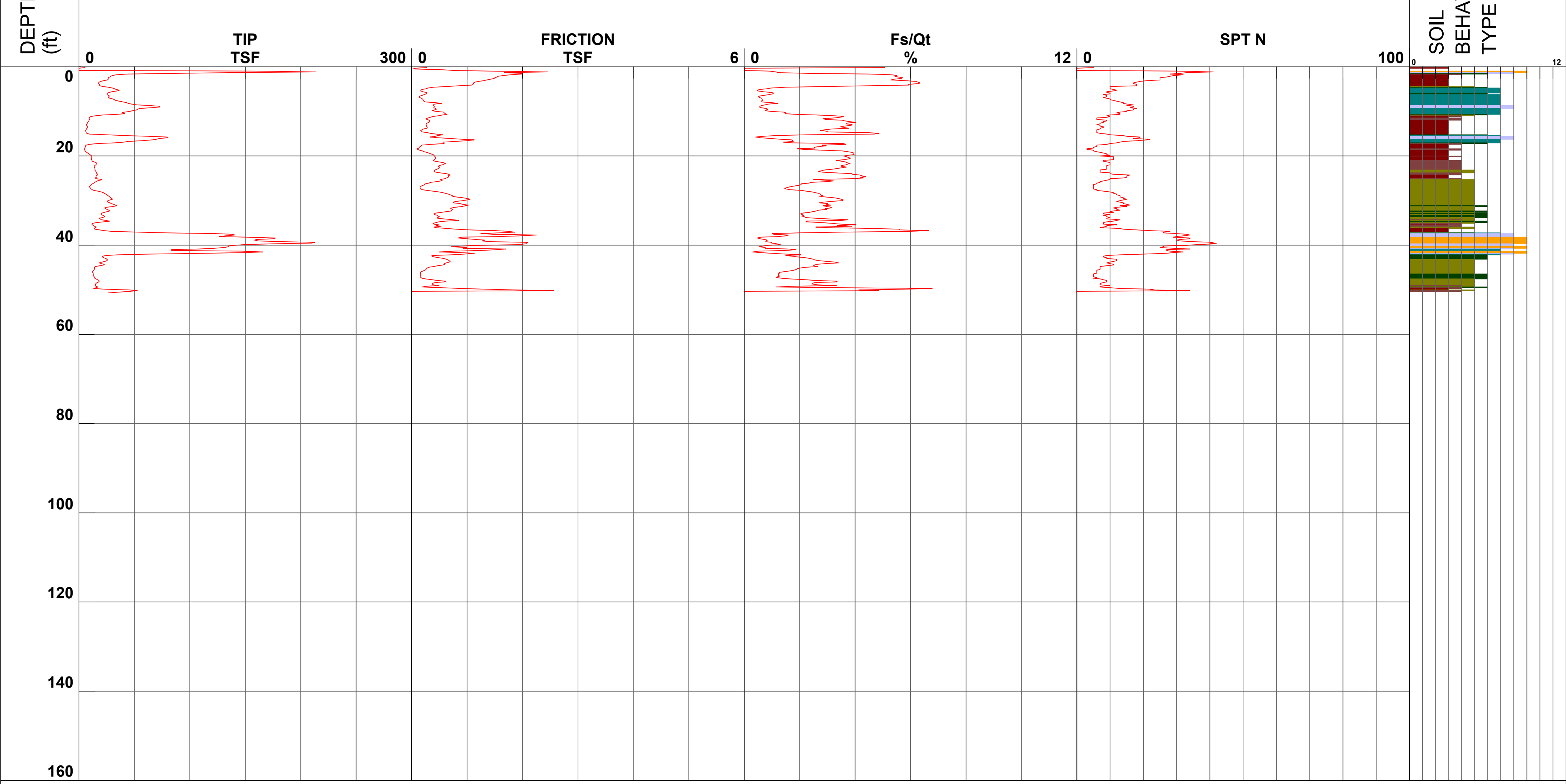
Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project 25550 Clawiter Rd
Job Number 916-2-1
Hole Number CPT-03
EST GW Depth During Test 15.00 ft
Operator JM-ZG
Cone Number DDG1530
Date and Time 7/30/2020 4:16:06 PM
Filename SDF(949).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8

CPT DATA



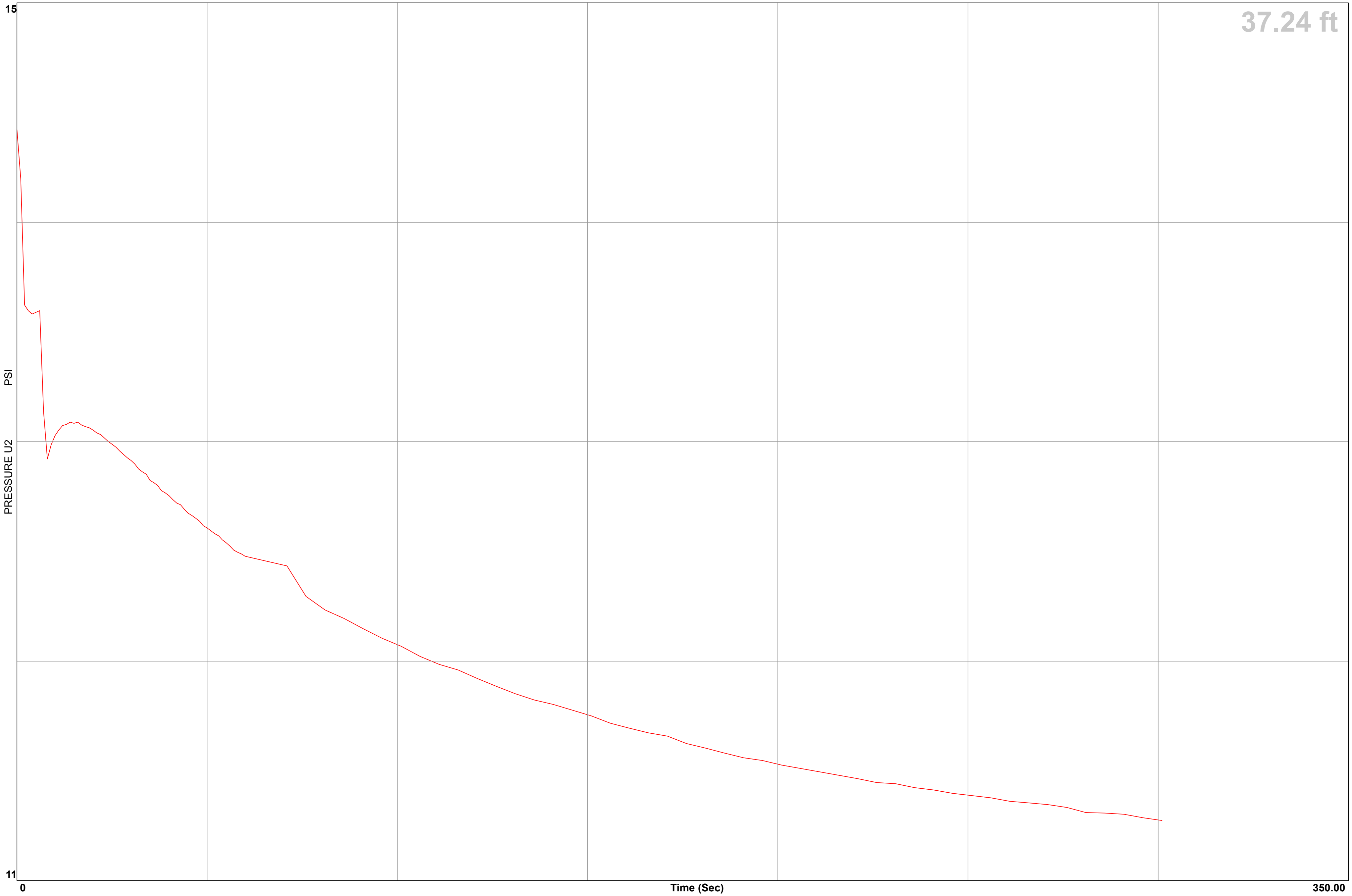
- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	25550 Clawiter Rd	**Operator**	JM-ZG		
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-03	**Date and Time**	7/30/2020 4:16:06 PM		
Equilized Pressure	11.2	**EST GW Depth During Test**	11.2		

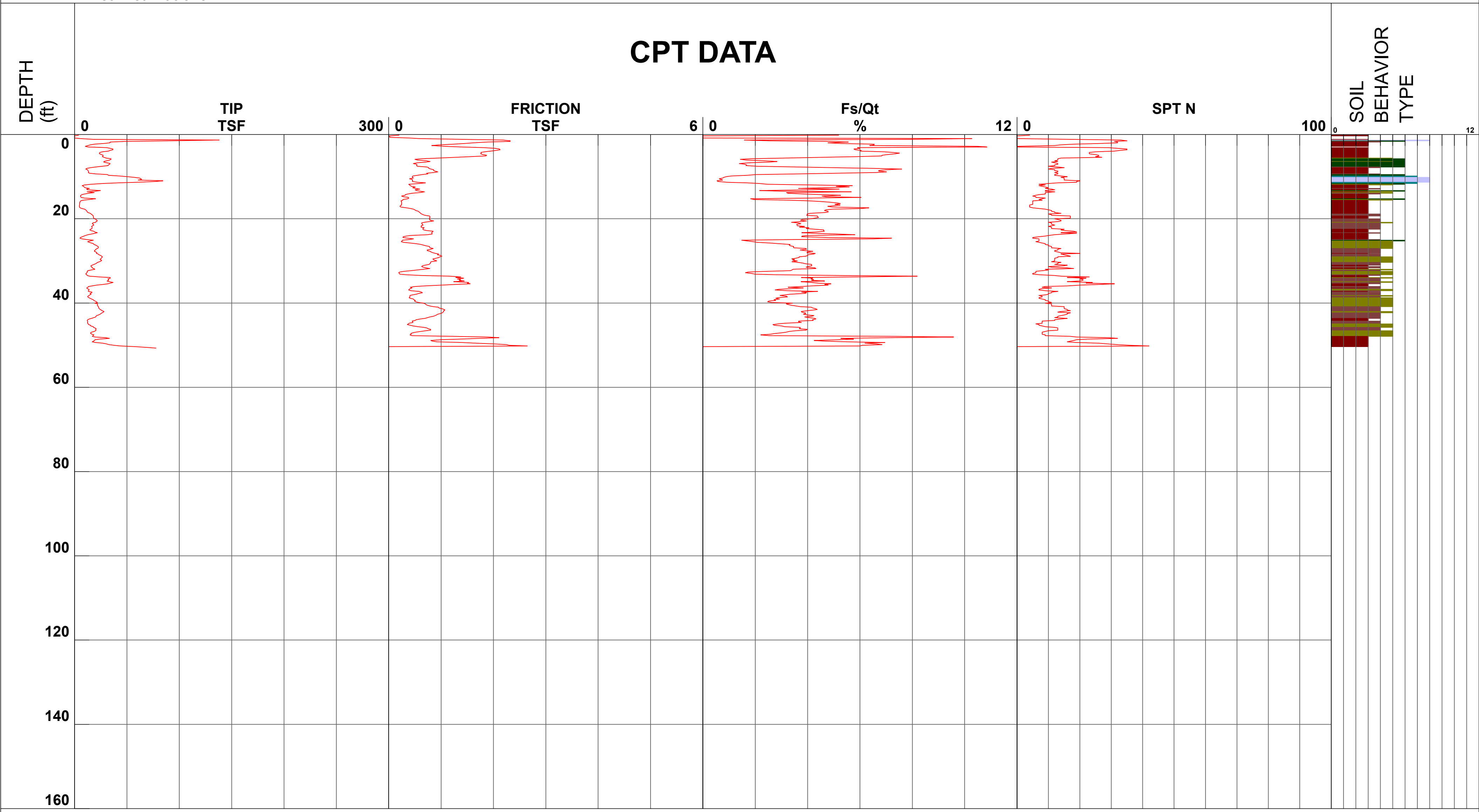


Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

Project	25550 Clawiter Rd	**Operator**	JM-ZG	**Filename**	SDF(944).cpt
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-04	**Date and Time**	7/30/2020 12:44:37 PM	**Maximum Depth**	50.69 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8



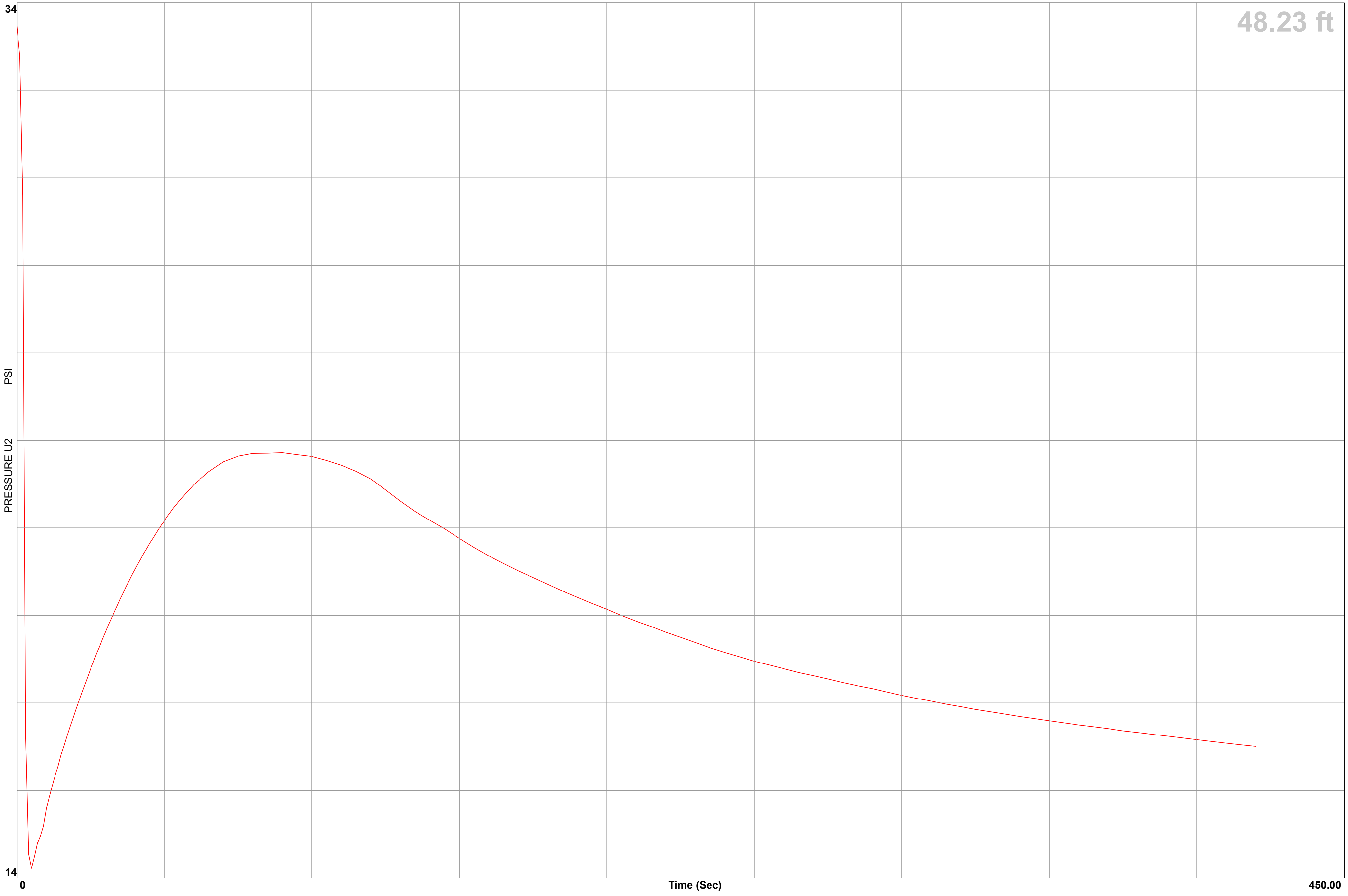
- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	25550 Clawiter Rd	**Operator**	JM-ZG		
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-04	**Date and Time**	7/30/2020 12:44:37 PM		
Equilized Pressure	17.0	**EST GW Depth During Test**	8.9		

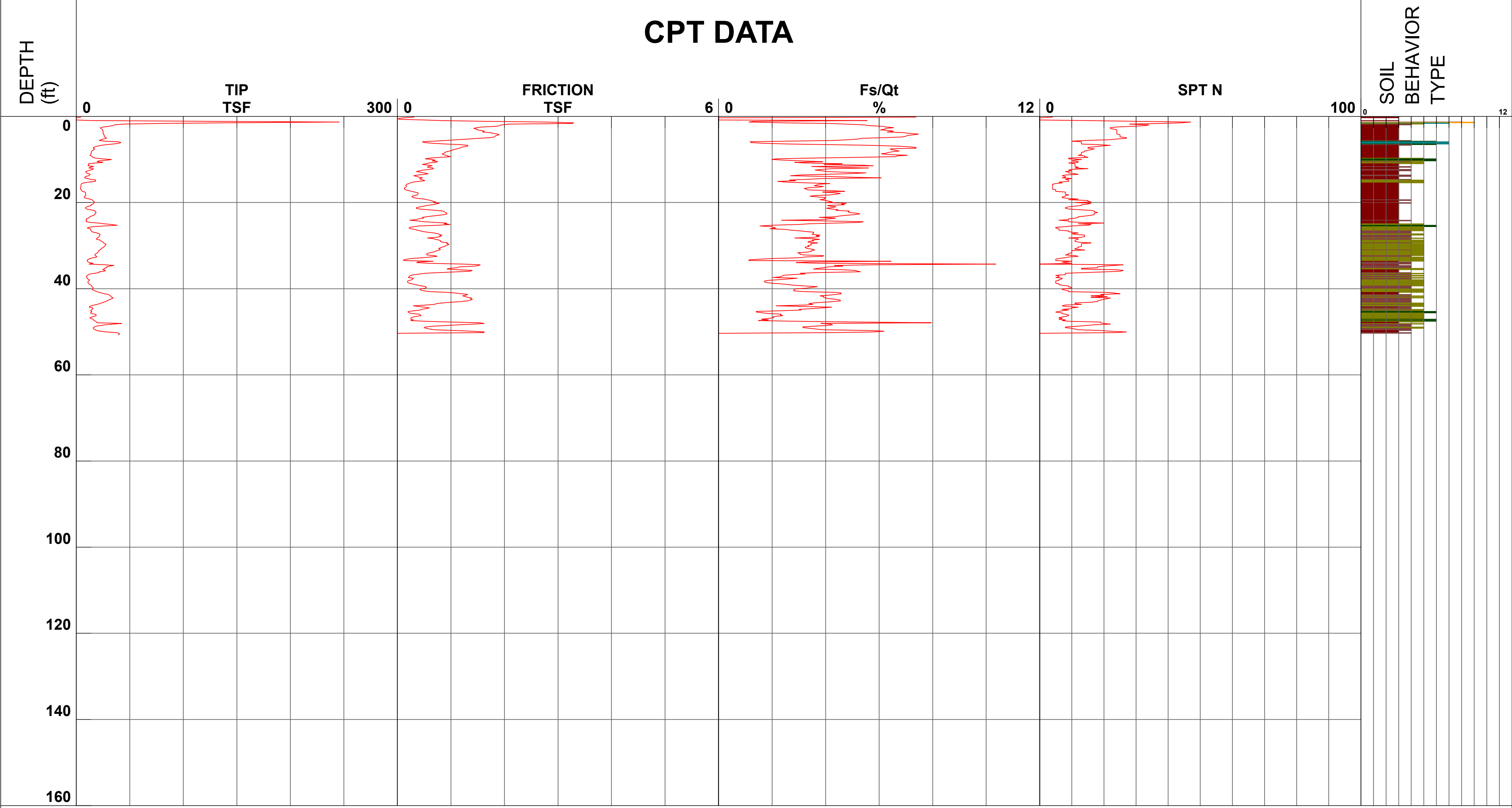


Cornerstone Earth Group

Project 25550 Clawiter Rd
Job Number 916-2-1
Hole Number CPT-05
EST GW Depth During Test 13.00 ft
Operator JM-ZG
Cone Number DDG1530
Date and Time 7/30/2020 1:39:40 PM
Filename SDF(945).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8

CPT DATA



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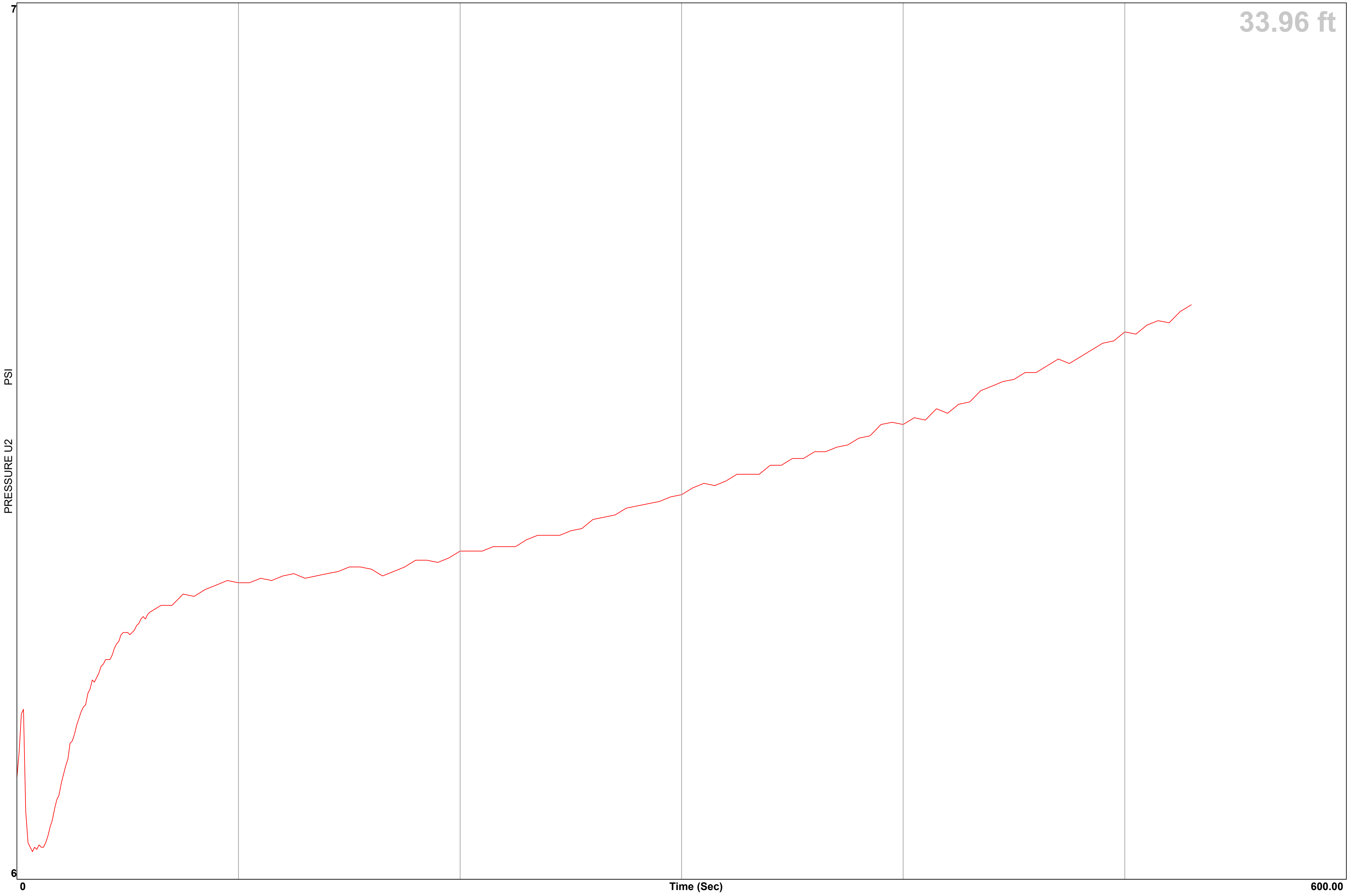
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	25550 Clawiter Rd	**Operator**	JM-ZG		
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-05	**Date and Time**	7/30/2020 1:39:40 PM		
Equilized Pressure	6.6	**EST GW Depth During Test**	18.5		

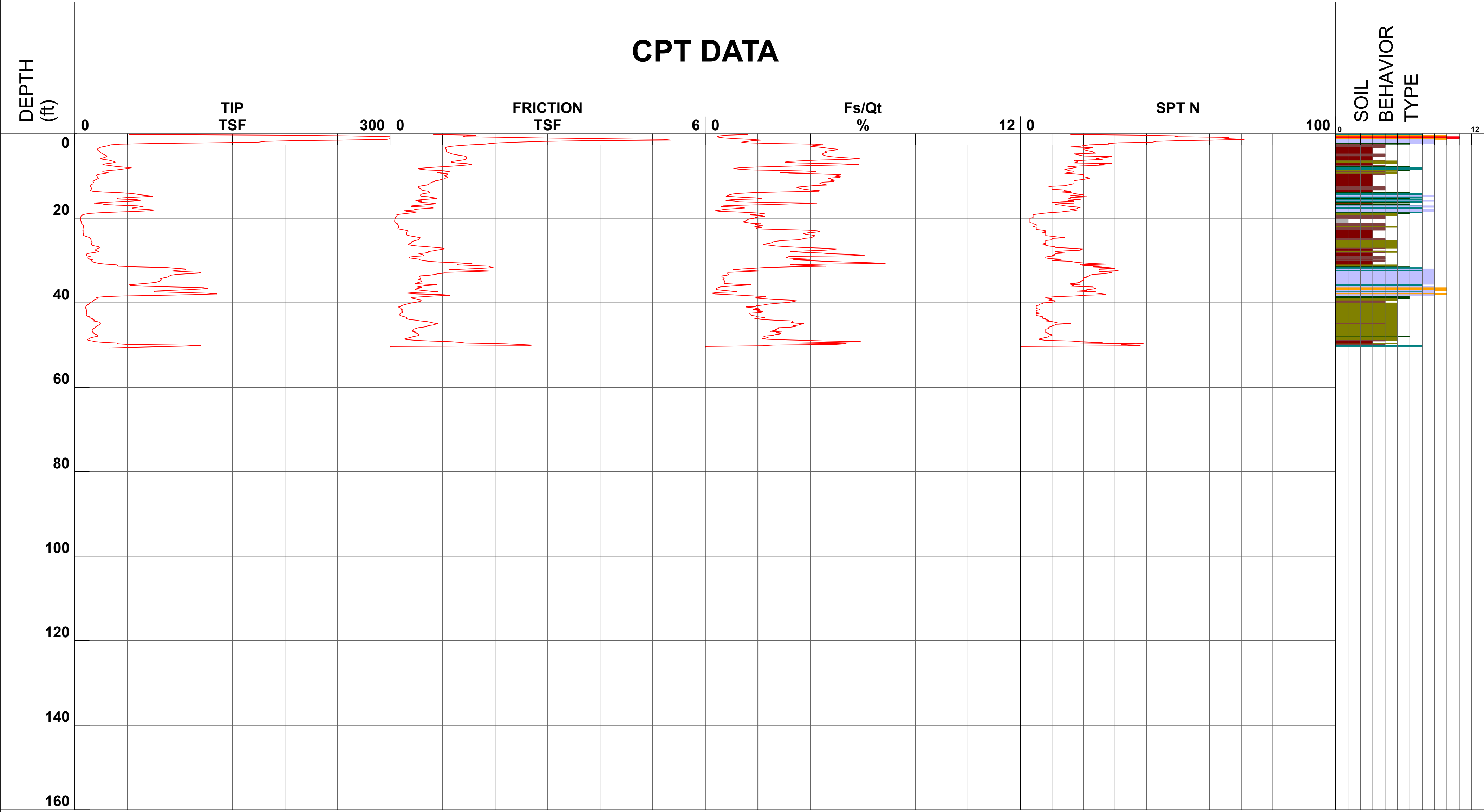


Cornerstone Earth Group

Project	25550 Clawiter Rd	**Operator**	JM-ZG	**Filename**	SDF(946).cpt
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-06	**Date and Time**	7/30/2020 2:36:56 PM	**Maximum Depth**	50.69 ft
EST GW Depth During Test		16.70 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
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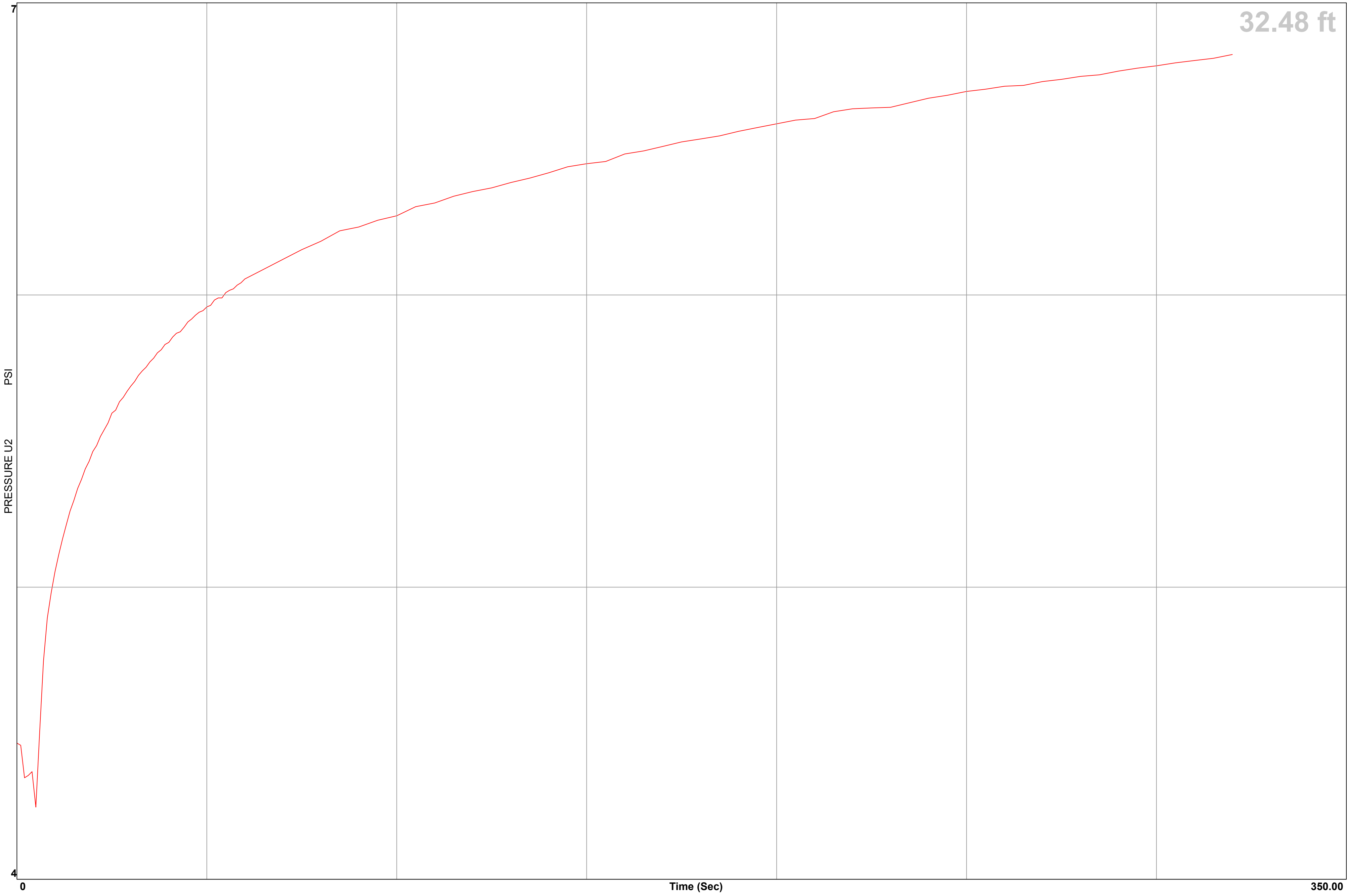
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

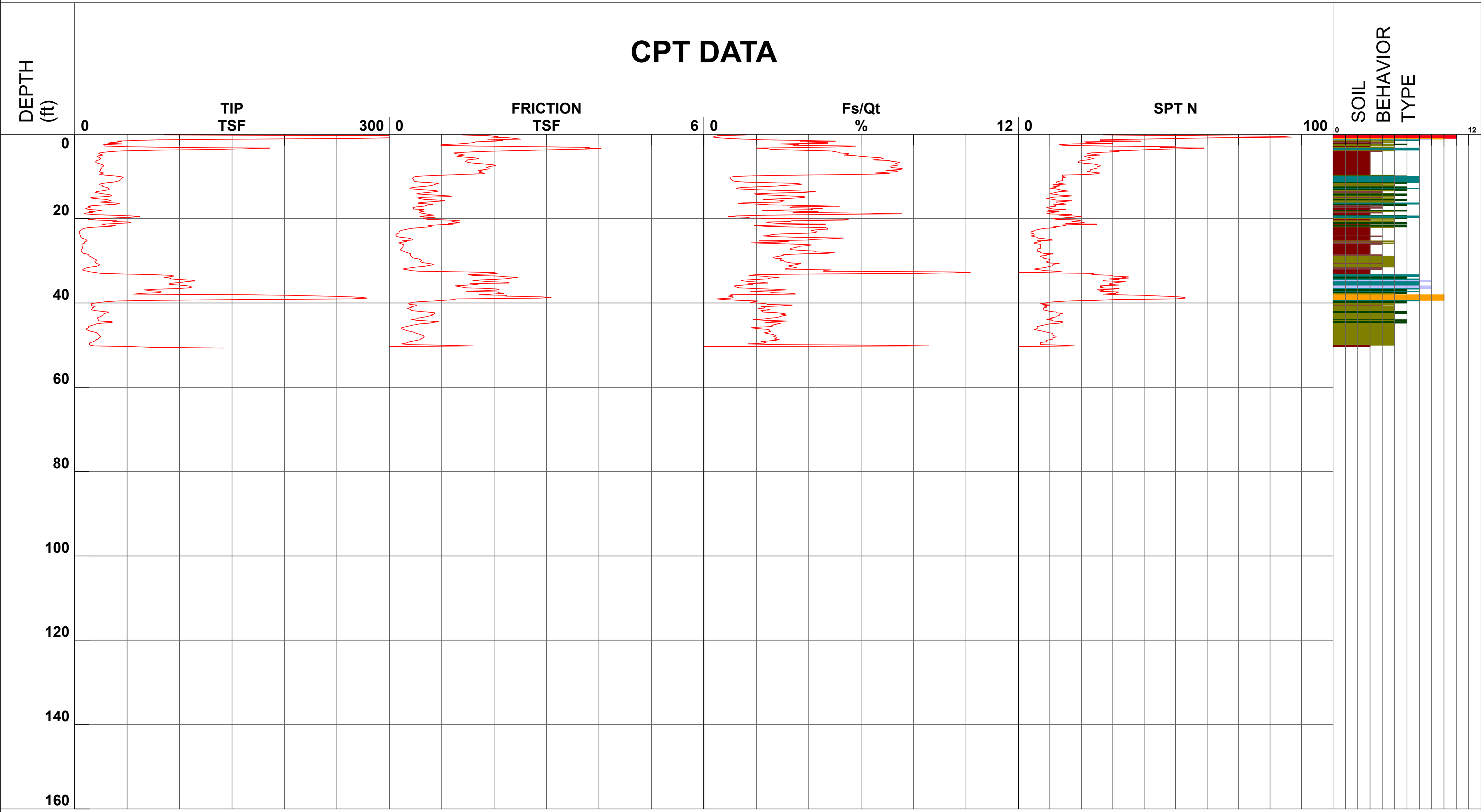
Location	25550 Clawiter Rd	Operator	JM-ZG		
Job Number	916-2-1	Cone Number	DDG1530	GPS	
Hole Number	CPT-06	Date and Time	7/30/2020 2:36:56 PM		
Equilized Pressure	6.8	EST GW Depth During Test	16.7		



Cornerstone Earth Group

Project 25550 Clawiter Rd
Job Number 916-2-1
Hole Number CPT-07
EST GW Depth During Test 21.30 ft
Operator JM-ZG
Cone Number DDG1530
Date and Time 7/30/2020 3:29:03 PM
Filename SDF(948).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8



CPT DATA

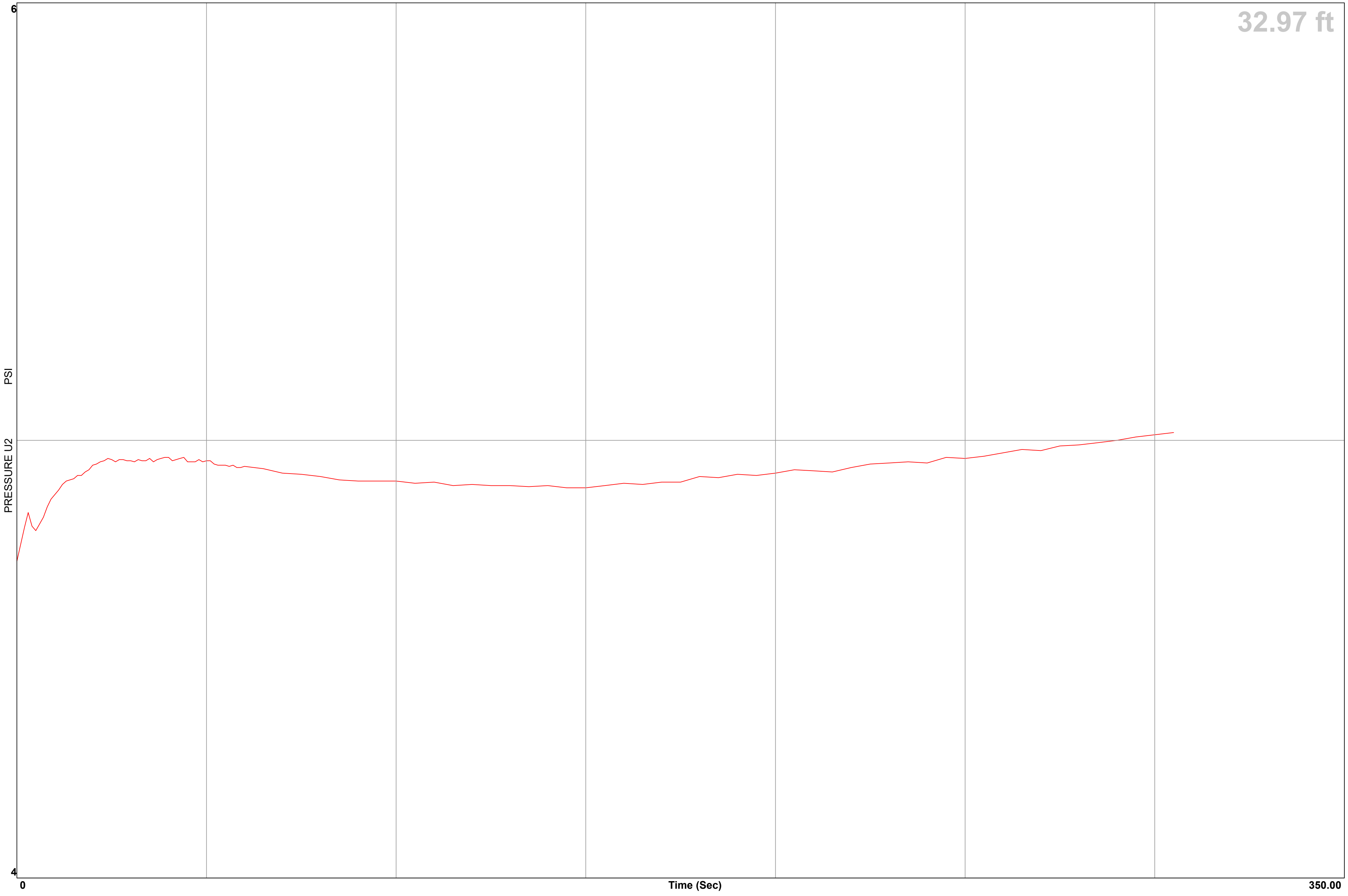
⬜ 1 - sensitive fine grained
⬛ 2 - organic material
🟥 3 - clay
🟫 4 - silty clay to clay
🟨 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt
🟦 7 - silty sand to sandy silt
⬜ 8 - sand to silty sand
🟧 9 - sand
🟥 10 - gravelly sand to sand
🟪 11 - very stiff fine grained (*)
🟦 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	25550 Clawiter Rd	**Operator**	JM-ZG		
Job Number	916-2-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-07	**Date and Time**	7/30/2020 3:29:03 PM		
Equilized Pressure	5.0	**EST GW Depth During Test**	21.3		



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 73 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

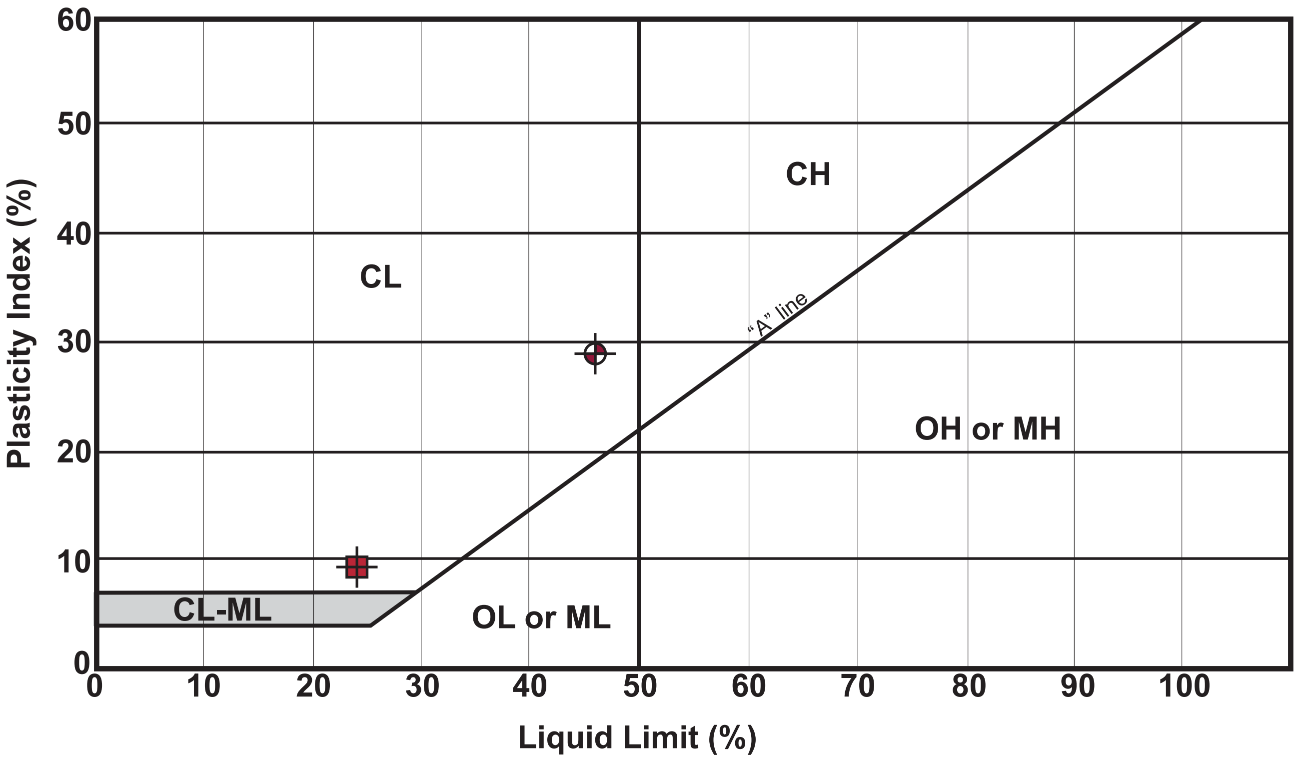
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 61 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on eight 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.

Consolidation: Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in 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-2	4.0	19	46	17	29	---	Lean Clay (CL)
■	EB-7	12.0	15	29	14	15	---	Clayey Sand (SC) (CL fines)

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

25500 Clawiter Road Industrial
Hayward, CA

Project Number
916-2-1

Figure Number
Figure B1

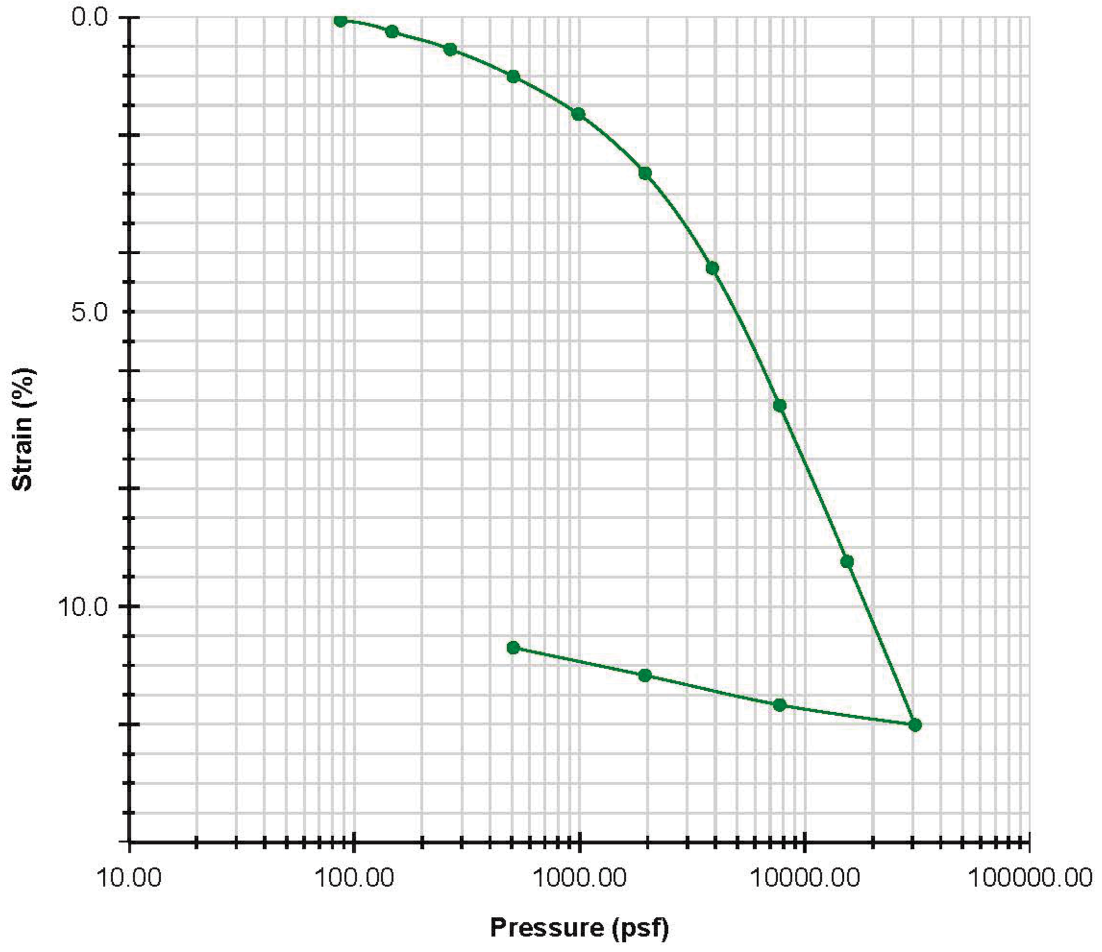
Date
August 2020

Drawn By
FLL

Consolidation Test ASTM D2435

Boring: EB-1 **Sample:** 7 **Depth:** 18.5’

Description: Sandy Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	17.2	14.8
Dry Density (pcf)	109.0	121.0
Saturation (%)	83.8	100.0
Void Ratio	0.56	0.40

—●— (A) Stress Strain Curve

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-1 @ 18.5’

25500 Clawiter Road Industrial
Hayward, CA

Project Number 916-2-1

Figure Number Figure B2

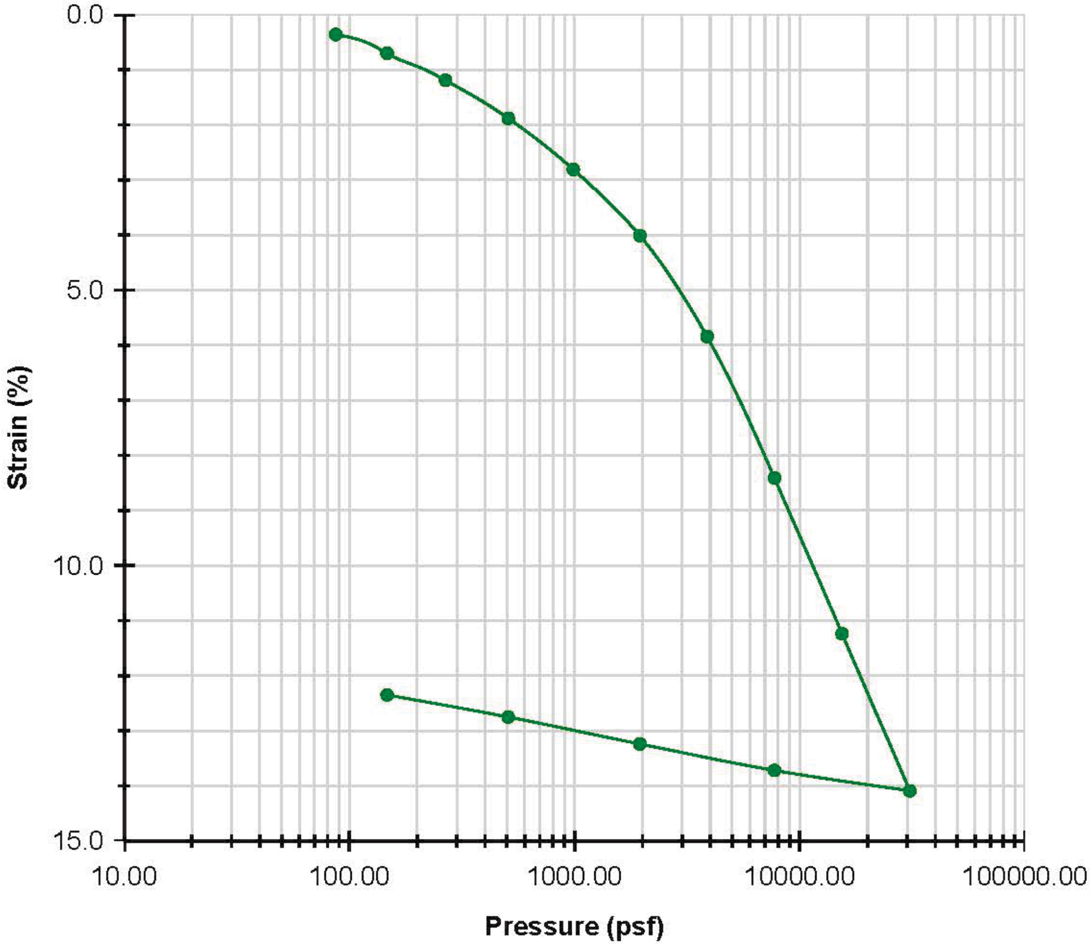
Date August 2020

Drawn By FLL

Consolidation Test ASTM D2435

Boring: EB-3 **Sample:** 7 **Depth:** 13.0’

Description: Sandy Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	22.3	17.4
Dry Density (pcf)	102.3	115.3
Saturation (%)	92.0	100.0
Void Ratio	0.66	0.47

(A) Stress Strain Curve

CORNERSTONE
EARTH GROUP

Strain-Log Curve - EB-3 @ 13.0’

25500 Clawiter Road Industrial
Hayward, CA

Project Number	916-2-1
Figure Number	Figure B3
Date	August 2020
Drawn By	FLL