CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Geotechnical Investigation

PROJECT NAME Enterprise-Whitesell Industrial Building

LOCATION Whitesell and Enterprise Drive
Hayward, California

CLIENT Dermody Properties

PROJECT NUMBER 916-3-1

DATE March 2, 2021


GEOTECHNICAL

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Enterprise-Whitesell Industrial Building**
Location	**Whitesell Street and Enterprise Drive** **Hayward, California**
Client	**Dermody Properties**
Client Address	**11900 NE 1st Street, Suite 300** **Bellevue, Washington**
Project Number	**916-3-1**
Date	**March 2, 2021**

REGISTERED PROFESSIONAL ENGINEER
MAURA F. RUFFATTO
No. 84145
Exp. 09-30-21
CIVIL
STATE OF CALIFORNIA

Maura Ruffatto

Prepared by **Maura F. Ruffatto, P.E.**
Project Engineer
Geotechnical Project Manager

Danh T. Tran, P.E.
Senior Principal Engineer
Quality Assurance Reviewer

TABLE OF CONTENTS

Type of Services | **Geotechnical Investigation**
Project Name | **Enterprise-Whitesell Industrial Building**
Location | **Whitesell Street and Enterprise Drive**
Hayward, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Dermody Properties for the Enterprise-Whitesell Industrial Building 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 2, Conceptual Site Plan, Whitesell St & Enterprise Ave, Hayward, CA 94545," prepared by Ware Malcomb, dated October 12, 2020.
- A preliminary (Progress Set) Grading and Drainage plan titled "3636 Enterprise Avenue," prepared by Kier and Wright, dated December 2020.
- A preliminary cut and fill plan sheet titled "Preliminary Earthwork Analysis for 3636 Enterprise Avenue," prepared by Kier and Wright, dated February 3, 2021.

1.1 PROJECT DESCRIPTION

The project will include redeveloping the approximately 11-acre site for a new industrial facility. The new facility will include an approximately 200,000 square foot building including about 10,000 square feet of office. The building will likely be high-bay, 36 to 38 feet clear and consist of tilt-up construction. Dock-high doors will be located along the west side of the building. Two grade-level doors will be located at the north and south ends of the dock doors. An approximately 1,600 square feet maintenance shed will be located in the northwest corner of the site. Additional trailer and auto parking will also be located to the north, east, and west sides of the building. A detention basin will run along the eastern site boundary. Appurtenant utilities, landscaping, and other improvements necessary for the overall site development will also be constructed.

Building loads are expected to be typical of this type of construction. We understand the building pad will be raised approximately 3 to 6 feet to bring the Building Finished Floor Elevation to 15 feet and raise the pad above the minimum flood hazard elevation. Minor cuts on the order of 1 to 3 feet along the perimeter of the site are anticipated as well.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated November 4, 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 five borings drilled on January 6 and 7, 2021 with truck-mounted, hollow-stem auger drilling equipment and five Cone Penetration Tests (CPTs) advanced on December 23, 2020. The borings were drilled to depths of 25 to 45 feet; the CPTs were advanced to depths of 50 to 135 feet. Seismic shear wave velocity measurements were collected from CPT-3. Boring EB-1 to EB-5 were advanced adjacent to CPT-1 to CPT-5, respectively, 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, Plasticity Index tests, a consolidation test, and a suite of corrosion tests. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Three samples from our borings at depths ranging from 1 to 6 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soils can be characterized as very severely corrosive to buried metal, and non-corrosive to buried concrete. Additional recommendations are provided in Section 3.4 below.

1.6 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 1 mile east of the San Francisco Bay. Our review of published regional geologic maps covering the general area of the site shows the site as underlain by two Holocene geologic units [alluvial fan fine grained facies (Qhff) and bay mud (Qhbm)]. The contact between the two geologic units occurs near the northeast property line (CGS, 2003). The Holocene geologic units along the margins of the bay are typically 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.

The alluvial fan fine-facies unit ("Qhff") is described by Wentworth et al. (1998) as:

> *Fine-grained alluvial fan and flood plain over bank deposits laid down in a very gently sloping portions of the alluvial fan or valley floor. Slopes in these distal fan areas are generally less than or equal to 0.5 degrees, soil are clay rich, and groundwater is within 3 meters of the ground surface. Deposits are dominated by clay and silt, with interbedded lobes of coarser alluvium (sand and occasional gravel). Deposits of coarse material within these fine-grained deposits are elongated in the down fan or down valley direction. These lobes are potential conduits for groundwater flow. The surface contact with relatively coarser facies, fan (Qhf) and levee (Qhl), is both gradational and interfingering, this us dashed.*

The Qhbm unit consists of estuarine deposits typically occurring in areas situated between the modern shoreline and the historical limits of tidal marshes and mudflats. These deposits interfinger with fine grained alluvium (Qaf). Based on a compilation map by McDonald et al. (1978), the younger (Holocene) portion of the bay mud (Qhbm) may be located closer to the west property line or just beyond it. These deposits are generally underlain by older Pleistocene-aged bay mud and alluvial soils consisting of interbedded clays, silts, sands, and gravels of varying thickness and composition (Brabb et al., 1998).

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, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults.

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)	3.8	6.1
Calaveras	11.4	18.4
San Andreas (1906)	14.7	23.6
Monte Vista-Shannon	15.2	24.5

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 SURFACE DESCRIPTION

The project site is located in an industrial area with warehouse and industrial buildings located north and west of the site. A wastewater treatment plant is located directly north across Enterprise Drive. A creek channel and small railroad berm with tracks are located directly south of the site. The creek appears to have been straightened and graded in a trapezoidal configuration with channel slopes appearing to be close to 1:1 slope. The site is currently occupied by four large radio towers that are located towards the center of the site with guy wires across the site for support. A small maintenance building is also located adjacent to the north/northeast tower. The remainder of the site is open field with low grass and some shrubs. The site is generally flat with site grades slightly higher on the northern section of the site compared to the southern section of the site. Site grades range from about 7½ to 10½ feet within the future building pad based on the preliminary grading and drainage plan provided. The site slopes from an elevation of about 7½ feet in the southwest corner to about 13½ feet in the northeast corner and along the northern side (along Enterprise Avenue). The adjacent buildings to the west appear to be a couple feet higher than the current project site elevation.

3.2 SUBSURFACE CONDITIONS

Below the existing ground surface, our exploratory borings generally encountered very stiff highly expansive clay with varying amounts of sand to depths of about 2 to 3¾ feet. Beneath the surficial clay, Boring EB-1 encountered very stiff lean clay with varying amounts of sand to a depth of about 12 feet underlain by interbedded layers of very dense clayey sand with varying amounts of gravel and dense well-graded sand with clay and gravel to a depth of about 20 feet. Beneath the sands, Boring EB-1 encountered stiff lean clay with sand to the terminal boring depth of 31 ½ feet. Beneath the fat clays, Borings EB-2 through EB-5 generally encountered stiff to hard lean clay with varying amounts of sand to the terminal boring depths of about 25 to 45 feet. Below the maximum boring depth of 45 feet, our CPTs generally encountered stiff to hard silts and clays with varying amounts of sand with interbedded layers of medium dense to dense sands with varying amounts of clay and silt to the maximum depth explored of 135 feet.

3.2.1 Plasticity/Expansion Potential

We performed three 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 result of the surficial PI test resulted in a PI of 43, indicating very high expansion potential to wetting and drying cycles. The result of the PI tests in a potentially liquefiable layers resulted in PIs of 9 and 25.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 3 percent below to about 14 percent above the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our borings at depths ranging from about 6½ to 8 feet below current grades. Pore pressure dissipation testing was conducted at CPT-1 through CPT-3. Groundwater was inferred based on pore pressure dissipation testing performed and estimated to be at depths ranging from about 3 to 10 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. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. A summary of groundwater measured or encountered during our field explorations is presented in Table 2 below.

Table 2: Depth to Groundwater

Boring/CPT Number	Date Drilled	Depth to Groundwater (feet)	Depth of Boring/CPT
CPT-1	12/23/2020	2.9	50
CPT-2	12/23/2020	5.7	50
CPT-3	12/23/2020	9.8	135
EB-1	01/06/2021	8	31½
EB-2	01/06/2021	8	25
EB-3	01/06/2021	7	45
EB-4	01/07/2021	6½	37½
EB-5	01/07/2021	8	30

*Elevation datum unknown (based on plans provided)

Historic high groundwater levels are mapped at a depth of approximately 5 feet or less below current grades (CGS, San Leandro 7.5-Minute Quadrangle, 2003). Based on our research of the California Geotracker website, the site directly east at 3590 Enterprise Avenue recorded monitoring well groundwater depths ranging from 6½ to 8½ feet elevation during the August of 1997. In general, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. A groundwater design depth of 4 feet below existing grades was selected based on the depth to groundwater maps, depth of groundwater encountered in our borings and our previous experience in the area.

3.4 PRELIMINARY CORROSION SCREENING

We tested three samples collected at depths between 1 to 6 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 3A.

Table 3A: Summary of Corrosion Test Results

Boring	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-2	1	7.6	545	373	378
EB-4	6	7.5	381	526	293
EB-5	3½	7.6	893	98	81

Notes: [1] ASTM G51
[2] ASTM G57 – 100% saturation
[3] ASTM D4327/Cal 422 Modified
[4] ASTM D4327/Cal 417 Modified
[5] 1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential. Based on the laboratory test results summarized in Table 3A and published correlations between resistivity and corrosion potential, the soils may be considered very severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 below in Table 3B.

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required. If pressurized metallic pipes or pipes with metallic fittings are used, we recommend a corrosion engineer be consulted to evaluate the need for cathodic protection and design the cathodic protection systems.

Table 3B: ACI Sulfate Soil Corrosion Design Values and Parameters

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Sulfate (S) Class	Cementitious Materials (2)
S, Sulfate	< 0.10	S0	no type restriction

Notes: (1) above values and parameters are from on ACI 318-14, Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1
(2) cementitious materials are in accordance with ASTM C150, ASTM C595, and ASTM C1157

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 $(PGA)_M$ was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2019 edition of the California Building Code. For our liquefaction analysis we used a PGA_M of 0.752g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, San Leandro Quadrangle, 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 ground water depth of 4 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), 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 ground water. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water 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. Selected soil samples collected from advancing the CPT equipment adjacent to CPT-1 to CPT-5 were tested to evaluate grain size, as well as visually observed for confirmation of CPT soil behavior types.

The results of our CPT analyses (CPT-1 through CPT-5) are presented on Figures 4A to 4E of this report.

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 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, differential settlements are anticipated to be on the order of ½-inch over a horizontal distance of 50 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation 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 minimum 16-foot thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable. In addition, the building pad will be raised about 3 to 5 feet increasing the non-liquefiable cap.

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 is an open channel located just south of the project site. However, based on our explorations, the layers susceptible to liquefaction at the site are below the bottom of the channel.; 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. As the soils encountered at the site above the design groundwater depth of 4 feet were predominantly stiff to very stiff clays, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

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 1 mile inland from the San Francisco Bay shoreline and is approximately 7½ to 13½ feet above mean sea level (based on topographic data provided). The southwestern section of the site is located within a tsunami inundation zone based on CGS Maps (2009). The potential for tsunami inundation to impact the site should be considered in design and further evaluated by the design team.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, most of the site is located within Zone AE, an area with a Base Flood Elevation of 11 feet. 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.

- Potential for liquefaction-induced settlements
- Potential for static settlements from new fill placement

- Shallow groundwater
- Presence of very highly expansive soils
- Soil corrosion potential

5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for 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 liquefaction-induced settlement on the order of ¼- to ¾-inch could occur, resulting in differential settlement up to about ½-inch. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.2 Potential for Static Settlement from New Fill Placement

As discussed, based on existing grades and preliminary grading plans provided to us, we understand up to about 5 feet of new fill will be placed within the planned building footprint to bring Finished Floor Elevations to 15 feet. We anticipate aerial settlements on the order of ½- to ¾-inch from the fill placement could occur in the underlying alluvial soils, mainly due to consolidation. In addition, we estimate static foundation settlements based on assumed building loads on the order of ¼- to ½-inch could occur. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.3 Shallow Groundwater

Shallow groundwater was measured at depths ranging from approximately 3 to 10 feet below the existing ground surface in our explorations across the site. Historic high groundwater is also mapped at about 5 feet or less below current grades. We used a depth to groundwater of 4 feet for our liquefaction analyses, which we recommend be used for planning purposes. 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 isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.4 Expansive Soils

Highly to very 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. Evaluation of potential import sources for the site should consider the acceptable range of plasticity, especially in the upper 2 to 3 feet of fill. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.5 Soil Corrosion Potential

A preliminary soil corrosion screening was performed based on the results of analytical tests on samples of the near-surface soil. In general, the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered very severely corrosive based on comparisons to published standards. We recommend that special requirements for corrosion control be made to protect metal pipes. We recommend that a corrosion engineer be retained to review this information and provide recommendations, as needed. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 6 feet for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

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 may be 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.

As an owner value-engineered option, existing slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to allow subsurface drainage. Future distress and/or higher maintenance may result from leaving these prior improvements in place. A discussion of recycling existing improvements is provided later in this report.

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. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 4 inches below existing grade in vegetated areas.

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

While fills were not encountered in our borings, any fills encountered during site grading should be removed from the building areas. 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 be 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 Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

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.

As discussed in the "Subsurface" section in this report, the in-situ moisture contents range up to about 14 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

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 6 to 12 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 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 warehouse pad. 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.

6.7.3 Non-Expansive Fill Using Lime 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 to further evaluate the feasibility of chemical treatment including material type and estimated percentage.

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 5 feet or less, 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 10 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: FOUNDATIONS

7.1 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.

7.2 SEISMIC DESIGN CRITERIA

We understand the project design will be based on the 2019 California Building Code (CBC). The 2019 CBC provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficient” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Shear wave velocity measurements performed at CPT-3 to a depth of approximately 100 feet resulted in average shear wave velocities of 758 feet per second (or 231 meters per second). Therefore, we have classified the site as Site Class D. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. We understand from our discussions with the structural engineer that they intend to take an exception per Chapter 20 of ASCE 7-16, and as such, a full Ground Motion Hazard Analysis will not be required. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 5: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.63234°
Site Longitude	-122.13121°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.625g
1-second Period Mapped Spectral Acceleration[1], S_1	0.614g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	null[2]
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.625g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	null[2]
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.083g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	null[2]

[1] For Site Class B, 5 percent damped.
[2] See Section 11.4.8 of ASCE 7-16 for values and calculations.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 12 inches wide, and extend at least 30 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 very highly 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.

7.3.2 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 6: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	100 to 140 kips
Exterior Isolated Column Footing	25 to 75 kips
Perimeter Strip Footing	2 to 3 kips per lineal foot

Based on the above loading, the allowable bearing pressures presented above, and a Finished Floor Elevation of 15 feet, we estimate that the total static footing settlement will be on the order of ¼- to ½-inch, with less than ½-inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential settlement due to new fill placement will be on the order of ½-inch or less and differential seismic movement will be on the order of ½-inch over a horizontal distance of 50 feet, resulting in a total estimated differential footing movement on the order of about 1 to 1¼ inches between foundation elements, assumed to be on the order of 30 feet. 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. In addition, if additional fill is planned we will need to review our analysis and provide updated settlement estimates.

Approximately ¾ to 1-inch of the total static settlement discussed above is due to primary consolidation of saturated clay layers. The time to the achieve about 90 to 95 percent of the primary consolidation is anticipated to take several months to a year after all the dead and live loads are in place based on the encountered alluvial conditions. The contractor should take this into consideration when scheduling the construction of sensitive finishes.

7.3.3 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 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 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.

7.3.4 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 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 5 inches thick should have a minimum compressive strength of 3,500 psi. The warehouse slab should also be supported on at least 12 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. We understand about 3 to 5 feet of fill is planned to bring building to a Finished Floor Elevation of 15 feet. If imported fill material meets our recommendations provided in this report, the imported fill will likely meet NEF requirements and additional NEF below the 12 inches of granular base will not be required. If less than about 18 inches of fill is placed below the granular base, we recommend an additional NEF section be included. Due to the very high plasticity of the surficial soils, an additional 18 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.

8.2 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.

8.3 EXTERIOR FLATWORK

Exterior slabs-on-grade, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 12 inches of non-expansive fill. In areas where no imported fill is placed during mass grading, the non-expansive fill thickness should be increased to 15 inches. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. 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.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner's option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

SECTION 9: VEHICULAR PAVEMENTS

9.1 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 7: 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	12.5	16.0
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 8: 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
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 use 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.

9.2 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 9: PCC Pavement Recommendations (Untreated Subgrade)

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

Table 10: PCC Pavement Recommendations (Chemical-Treated Subgrade)

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.0
130	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.

9.2.1 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.

9.3 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 6 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 10: RETAINING WALLS

10.1 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 11: 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.

10.2 SEISMIC LATERAL EARTH PRESSURES

10.2.1 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we understand two retaining walls, one along the eastern edge and one southern edge of the site are planned. However, we understand the walls will be 6 feet or less in height. We are not aware of any retaining walls greater than 6 feet in heigh planned for the site. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

10.3 WALL DRAINAGE

10.3.1 At-Grade Site Walls

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.

10.4 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.

10.5 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 11: 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 Enterprise-Whitesell Industrial Building 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 ground water 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 12: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and Dileo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

Boulanger, R.W. and Idriss, I.M., 2004, Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis.

Boulanger, R.W. and Idriss, I.M., 2014, CPT and SPT Based Liquefaction Triggering Procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/GCM-14/01, April 2014

Brabb, E. E., Graymer, R. W. and Jones, D. 1., 1998, Geologic Map of the Palo Alto 30' x 60' Quadrangle, California: Derived From The Digital Database Open File 98-348: U. S. Geological Survey, Miscellaneous Field Studies, Map MF-2332.

California Building Code, 2010, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (2008), “Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

California Geological Survey, 2003, State of California Seismic Hazard Zones, San Leandro 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 078.

Federal Emergency Management Administration (FEMA), 2018, FIRM City of Hayward, California, Community Panel #06001C0269H.

Helley, E.J. and Graymer, R.W., 1997, Quaternary geology of Alameda County, and parts of Contra Costa, Santa Clara San Mateo, San Francisco, Stanislaus and San Joaquin counties, California: A digital database: U.S. Geological Survey Open File Report 97-97.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 237 p.

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes: Proceedings Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco.

Ishihara, K. and Yoshimine, M., 1992, Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, 32 (1): 173-188.

McDonald, S.D., Nichols, D. R., Wright, N.A., and Atwater B., 1978, Map showing thickness of young bay mud, southern San Francisco Bay, California, 1:125,000 scale.

Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.

Ritter, J.R., and Dupre, W.R., 1972, Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region, California: San Francisco Bay Region Environment and Resources Planning Study, USGS Basic Data Contribution 52, Misc. Field Studies Map MF-480.

Robertson, P.K., Shao, Lisheng, 2010, Estimation of Seismic Compression in Dry Soils Using the CPT, 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 4.05a, May 24-29, 2010.

Schwartz, D.P. 1994, New Knowledge of Northern California Earthquake Potential: in Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice, Applied Technology Council 35-1.

Seed, H.B. and I.M. Idriss, 1971, A Simplified Procedure for Evaluation soil Liquefaction Potential: JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 – 1274.

Seed, H.B. and I.M. Idriss, 1982, Ground Motions and Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute.

Seed, Raymond B., Cetin, K.O., Moss, R.E.S., Kammerer, Ann Marie, Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, Jonathan D., Kayen, Robert E., and Faris, A., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework., University of California, Earthquake Engineering Research Center Report 2003-06.

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, March.

State of California Department of Transportation, 2018, Highway Design Manual, Sixth Edition, December 14, 2018.

Tokimatsu, K., and Seed, H. Bolton, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, August 1987, pp. 861-878.

Wentworth, C.M., Blake, M.C., Jr., McLaughlin, R.J., and Graymer, R.W., 1998, Preliminary geologic description of the San Jose 30 X 60 Minute quadrangle, California: U.S. Geological Survey Open-File Report 98-795, 47 pp.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S, K. and Gans, K.D., 2006, Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region, California: U.S. Geological Survey, Open-File Report OF-2006-1037, scale 1:200000.

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at:

www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

Yoshimine, M., Nishizaki, H., Amano, KI, and Hosono, Y., 2006, Flow Deformation of Liquefied Sand Under Constant Shear Load and Its Application to Analysis of Flow Slide in Infinite Slope, Soil Dynamics and Earthquake Eng. 26, 253-264.

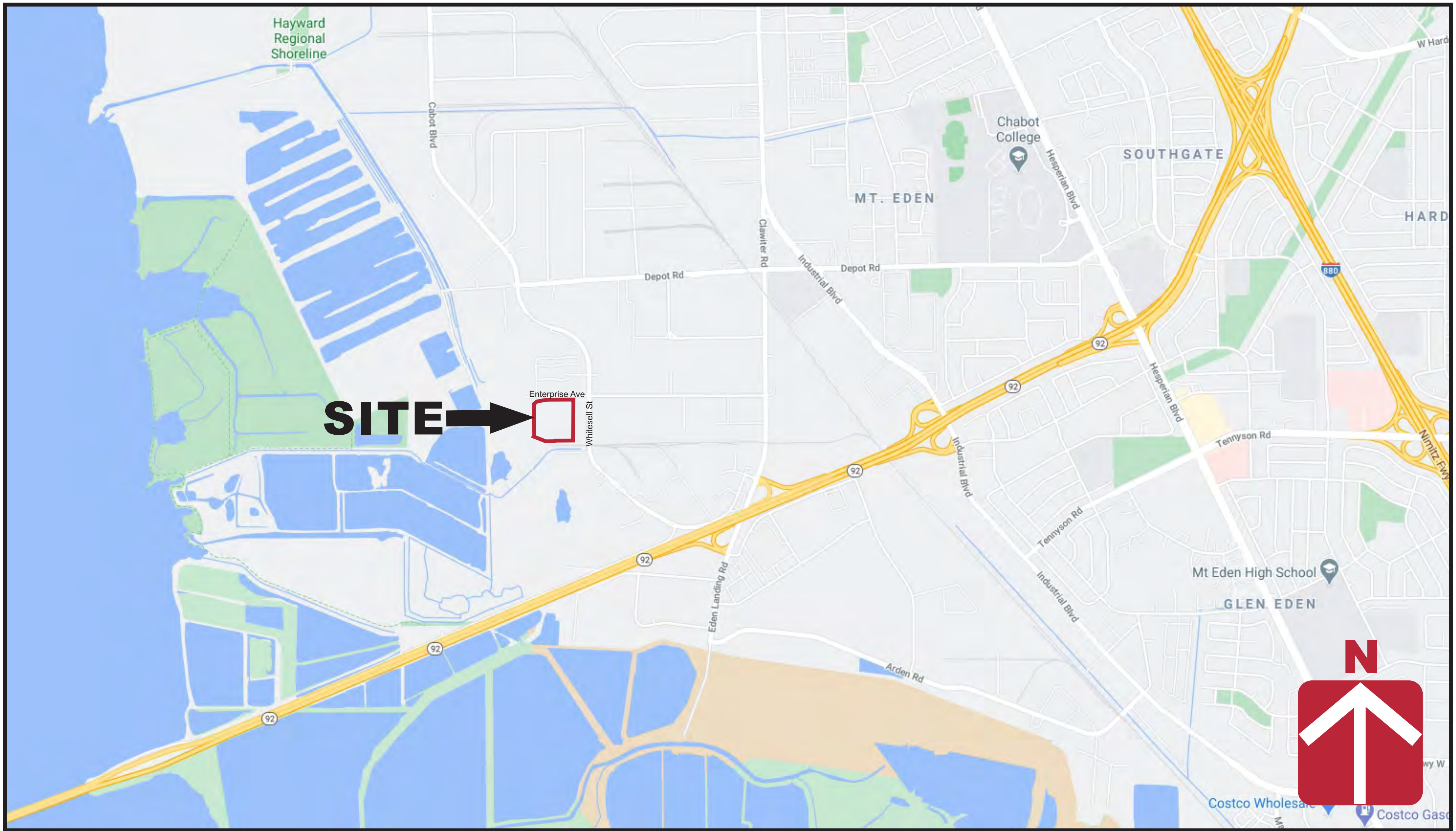
Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

Youd et al., 2001, “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

Youd, T. Leslie, Hansen, Corbett M., and Bartlett, Steven F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement: ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, December 2002, p 1007-1017.

Youd, T.L. and Hoose, S.N., 1978, Historic Ground Failures in Northern California Triggered by Earthquakes, United States Geologic Survey Professional Paper 993.



CORNERSTONE EARTH GROUP

Vicinity Map

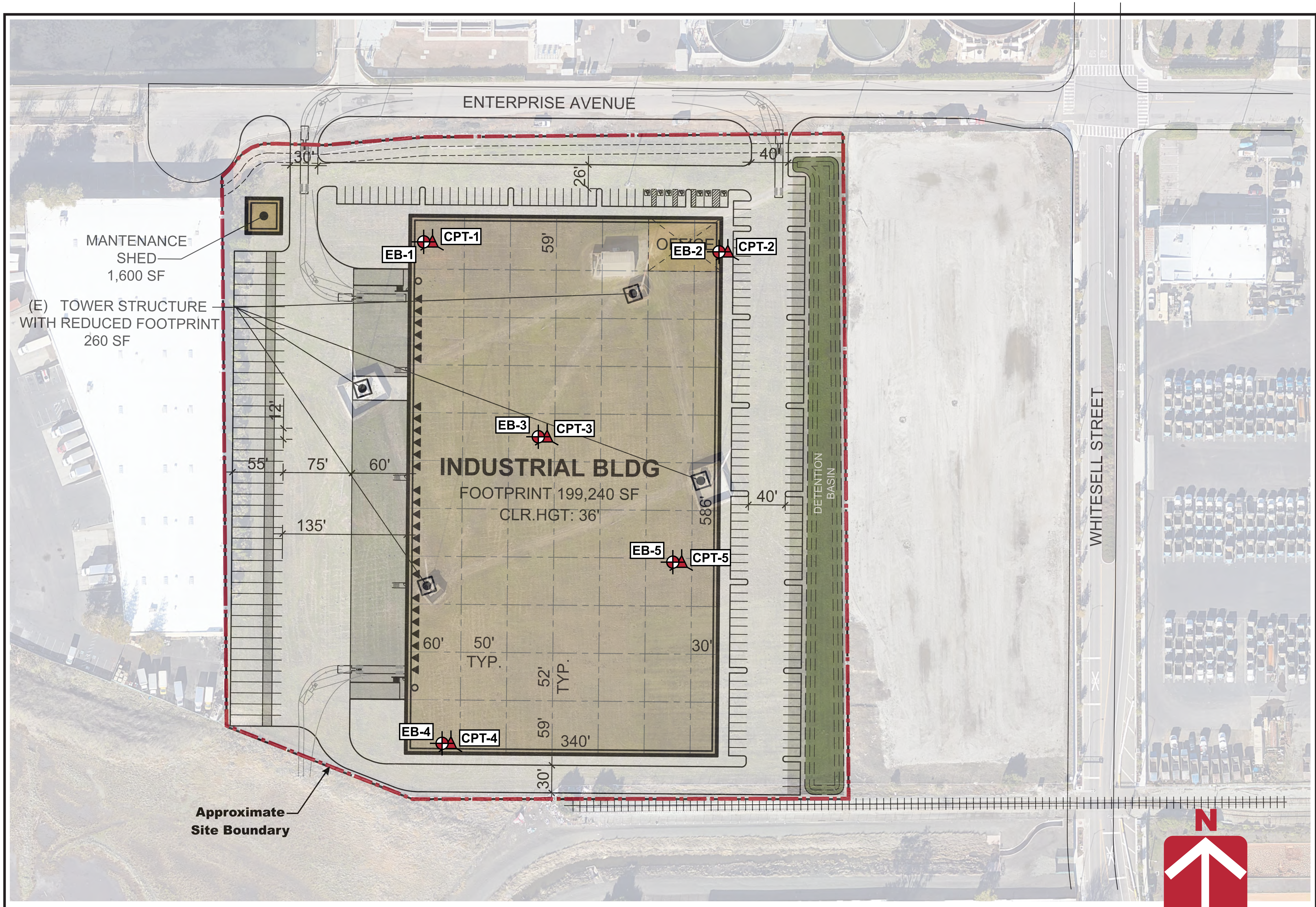
Enterprise/Whitesell Industrial Building
Hayward, CA

Project Number 916-3-1

Figure Number Figure 1

Date January 2021

Drawn By RRN



Legend

- ⊕ Approximate location of exploratory boring (EB)
- ▲ Approximate location of cone penetration test (CPT)

Base by Google Earth, dated 03/28/2018
Overlay by Ware Malcomb, Conceptual Site Plan, Scheme 2 - Sheet 1, dated 10/12/2020

Project Number 916-3-1

Figure Number Figure 2

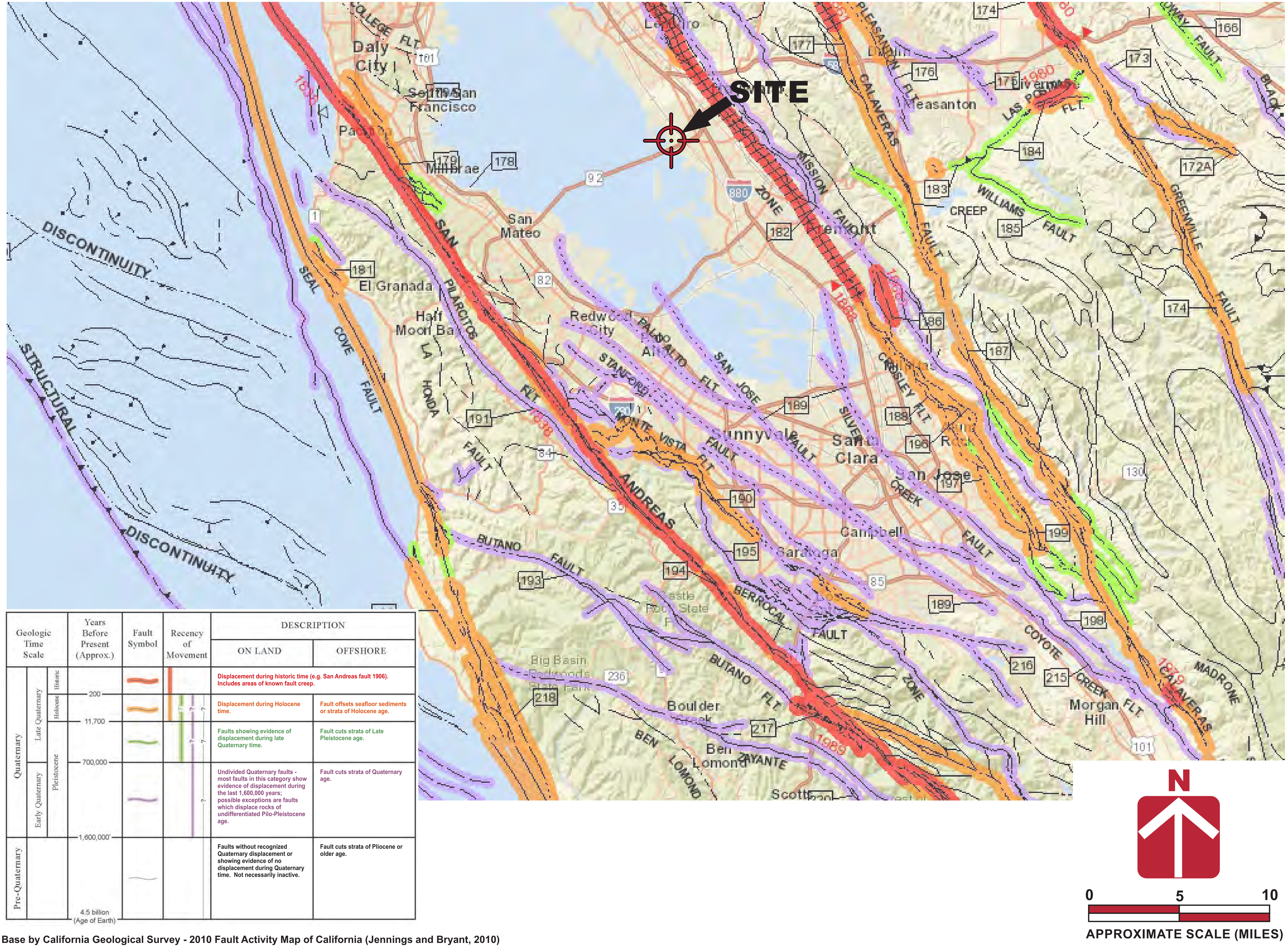
Drawn By RRN

Date January 2021

Site Plan

Enterprise/Whitesell Industrial Building
Hayward, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: 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)

CORNERSTONE EARTH GROUP

Regional Fault Map

Enterprise/Whitesell Industrial Building
Hayward, CA

Project Number 916-3-1

Figure Number Figure 3

Date January 2021

Drawn By RRN

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title: **Enterprise-Whitesell Industrial Bldg**

Project No.: **916-3-1**

Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**

Earthquake Magnitude (Mw): **7.1**

PGA (Amax): **0.752** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **3**

Design Water Depth (feet): **4**

Ave. Unit Weight Above GW (pcf): **122**

Ave. Unit Weight Below GW (pcf): **130**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.83 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.8 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 170.0

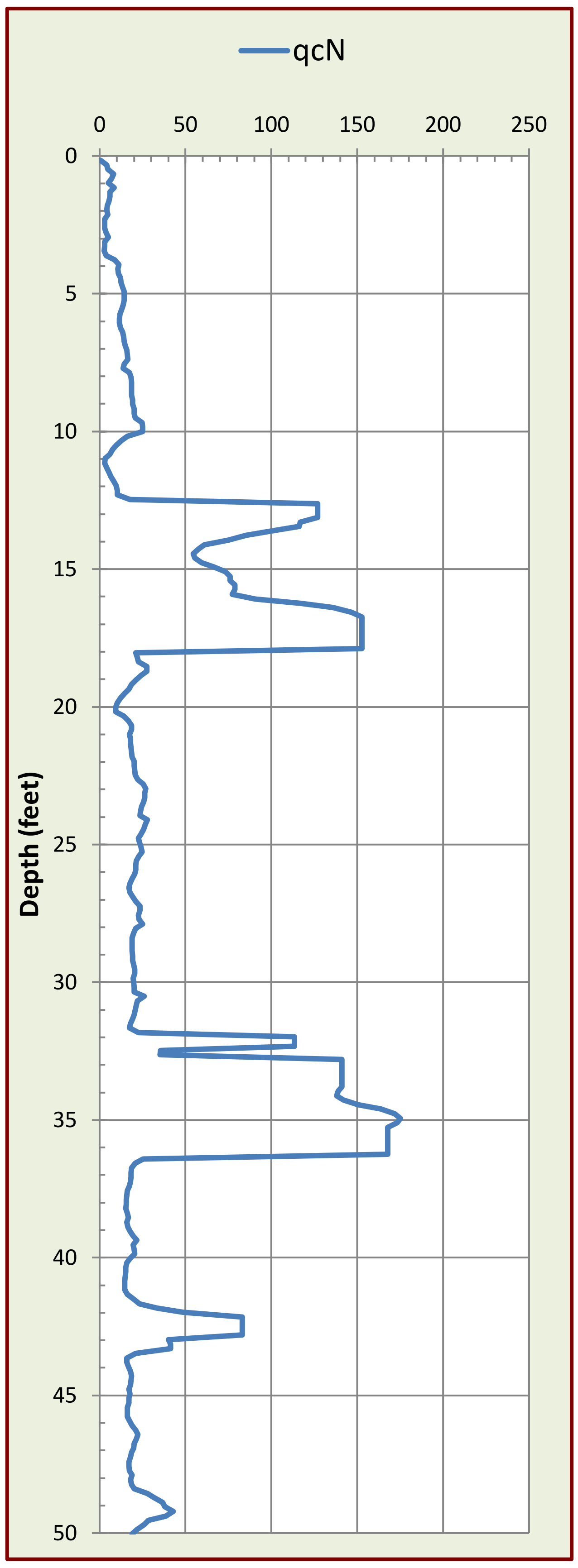
LDI^1$_{\text{Corrected for Distance}}$ 0.00 (4 < L/H < 40)

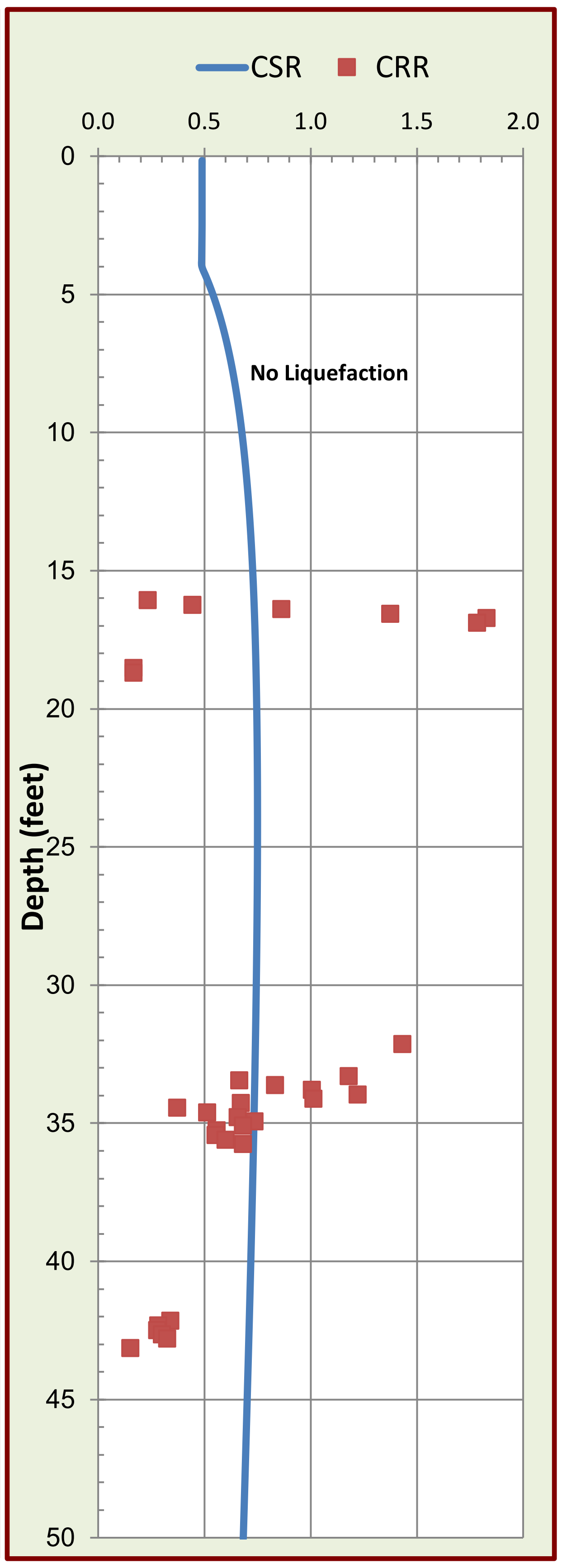
EXPECTED RANGE OF DISPLACEMENT

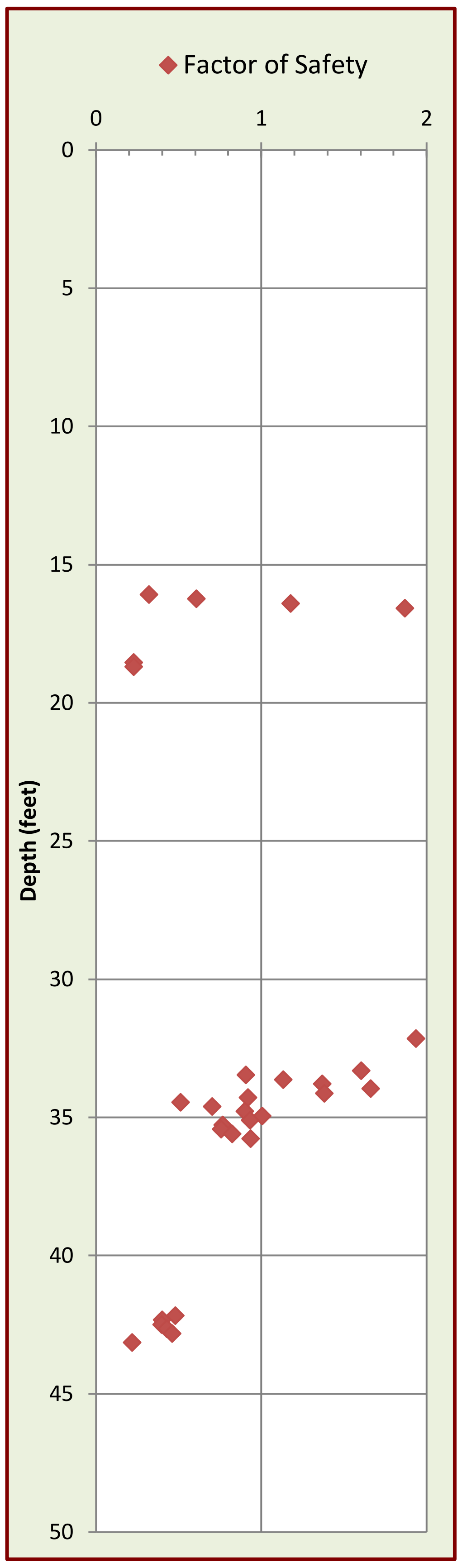
0.0 **to** 0.0 **feet**

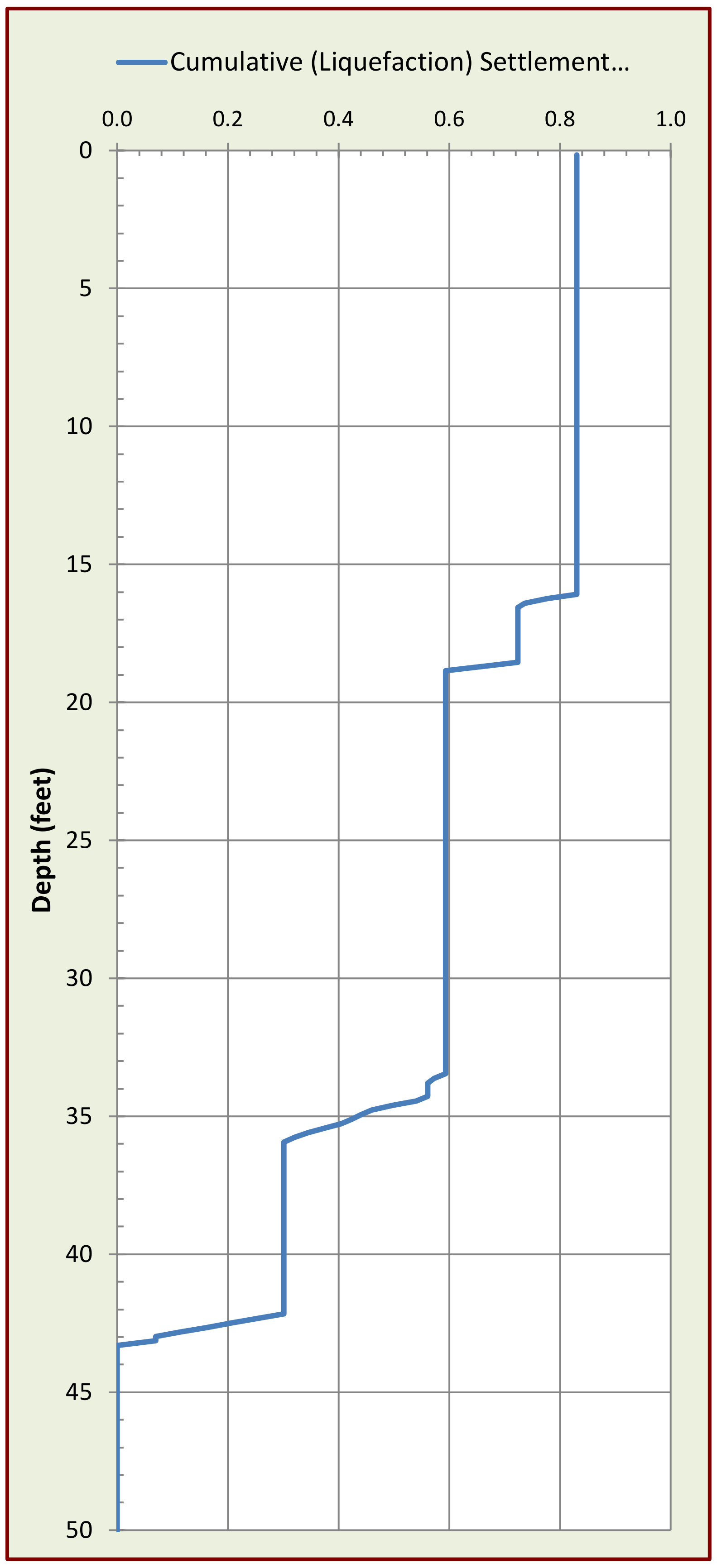
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title **Enterprise-Whitesell Industrial Bldg**

Project No. **916-3-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.1**

PGA (Amax) **0.752** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5.7**

Design Water Depth (feet) **4**

Ave. Unit Weight Above GW (pcf) **125**

Ave. Unit Weight Below GW (pcf) **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.27 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 160.0

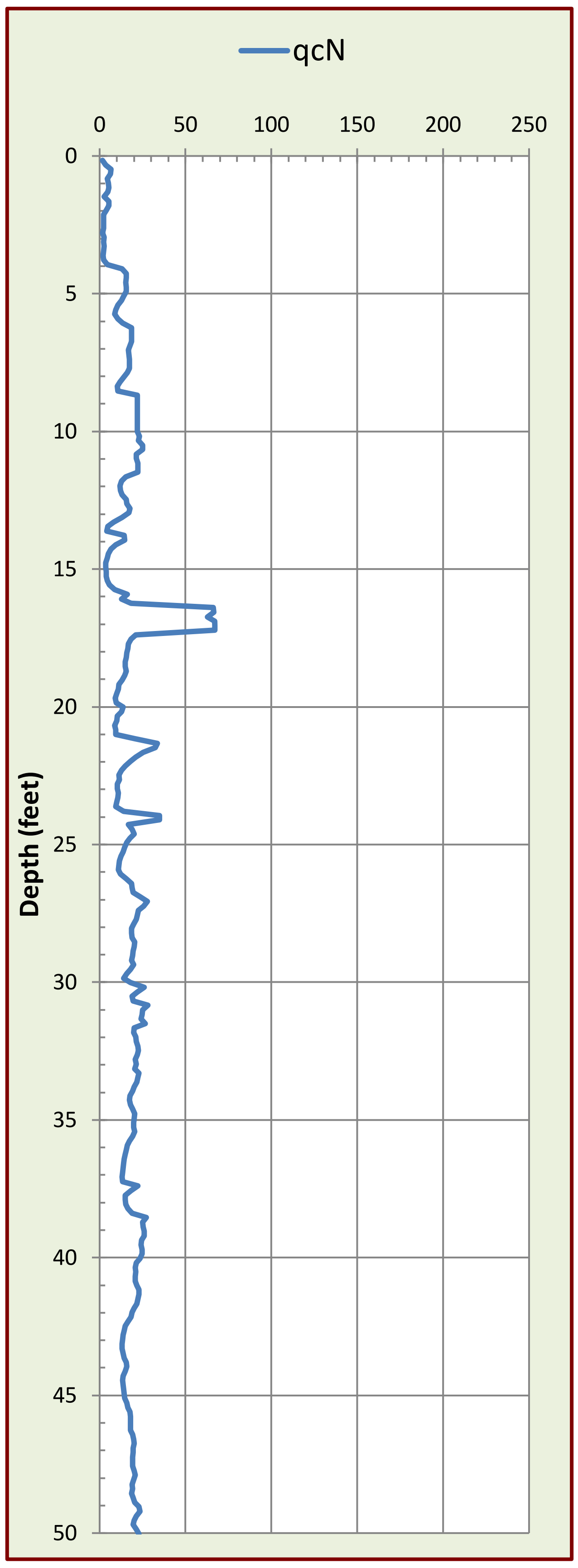
LDI^1$_{\text{Corrected for Distance}}$ 0.00 (4 < L/H < 40)

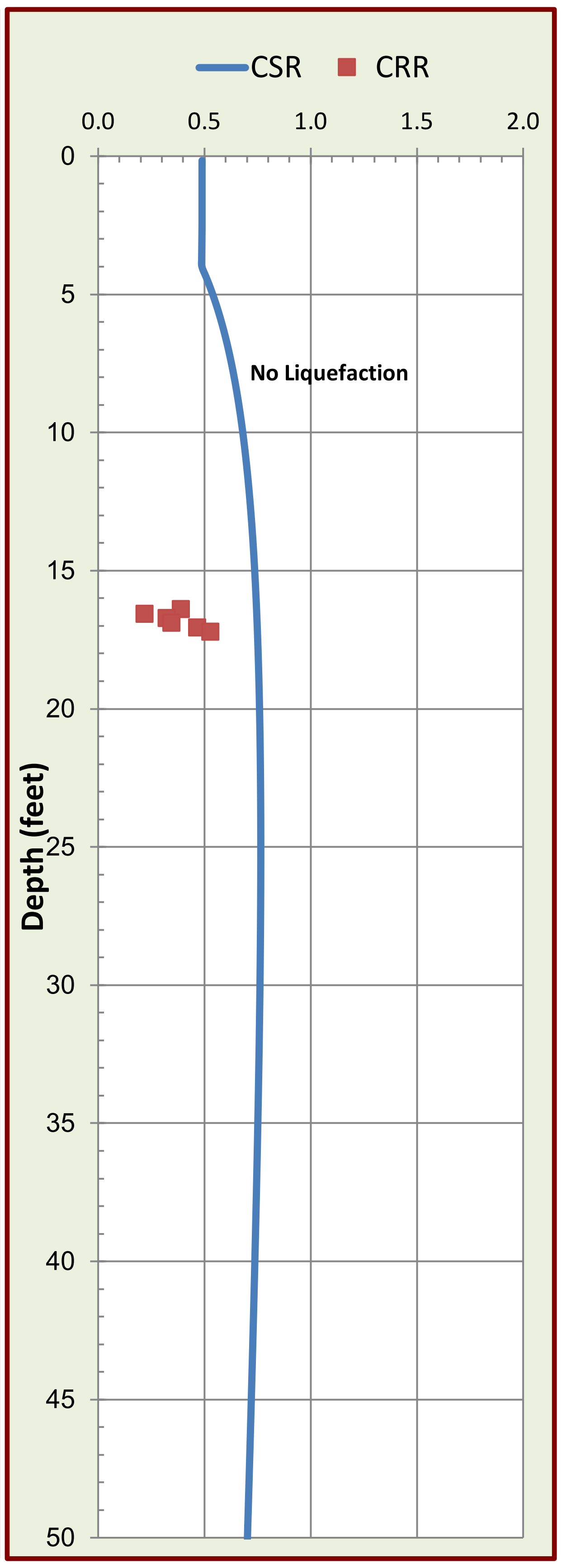
EXPECTED RANGE OF DISPLACEMENT

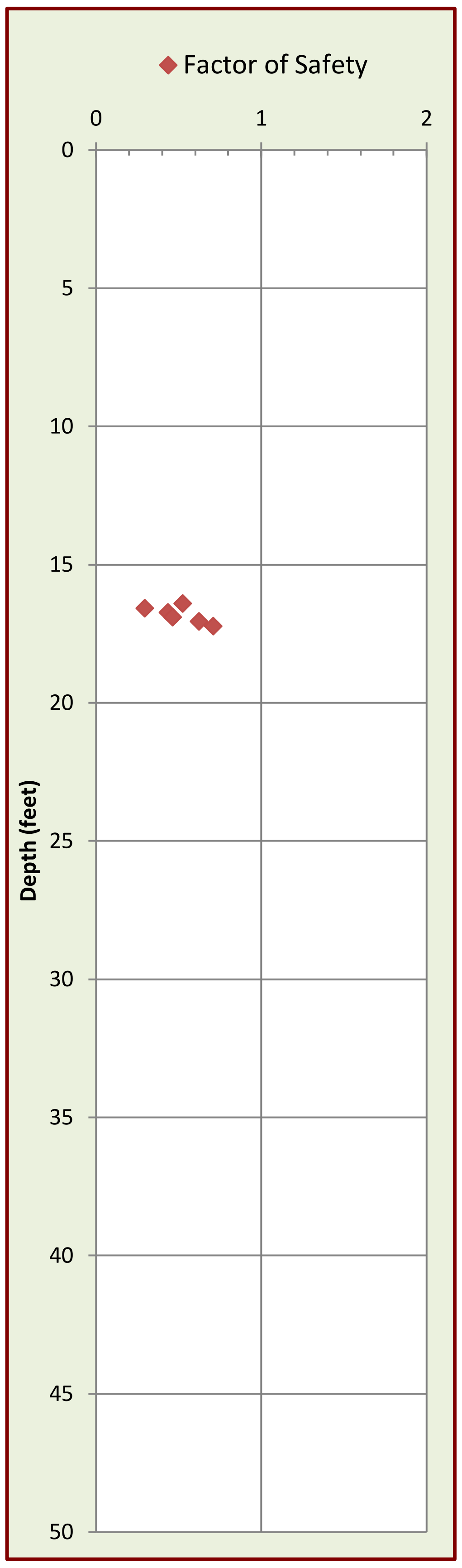
0.0 **to** **0.0** **feet**

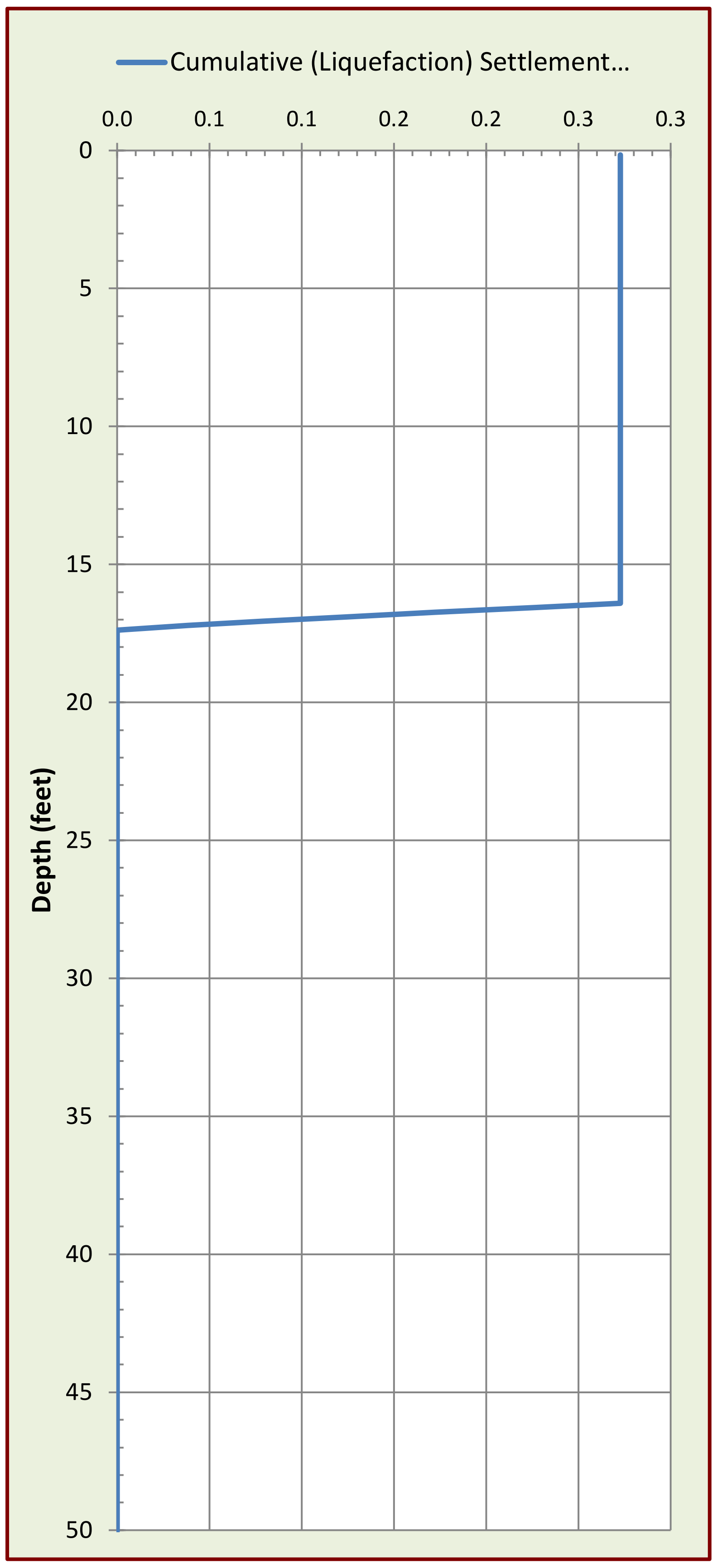
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title: **Enterprise-Whitesell Industrial Bldg**
Project No.: **916-3-1**
Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**
Earthquake Magnitude (Mw): **7.1**
PGA (Amax): **0.752** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **9.8**
Design Water Depth (feet): **4**
Ave. Unit Weight Above GW (pcf): **127**
Ave. Unit Weight Below GW (pcf): **127**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**
0.00 **(Inches)**
LIQUEFACTION SETTLEMENT FROM 50 **FEET**
0.00 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.0 **INCHES**

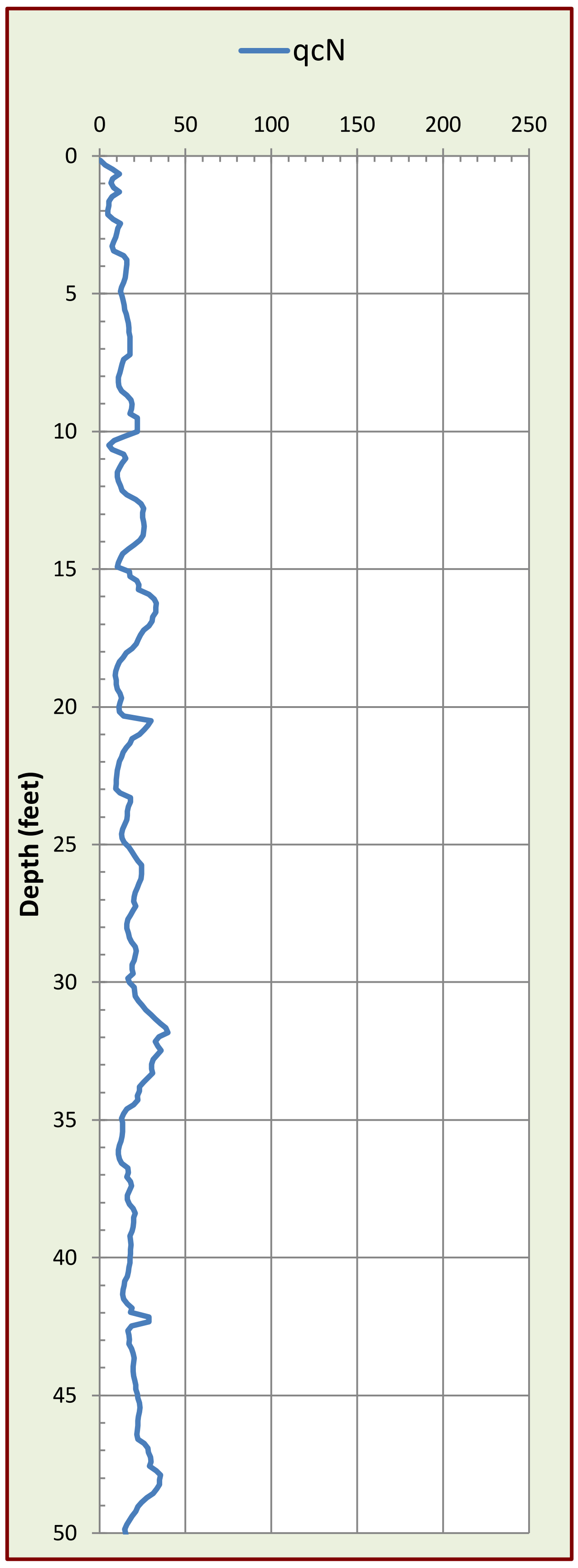
POTENTIAL LATERAL DISPLACEMENT

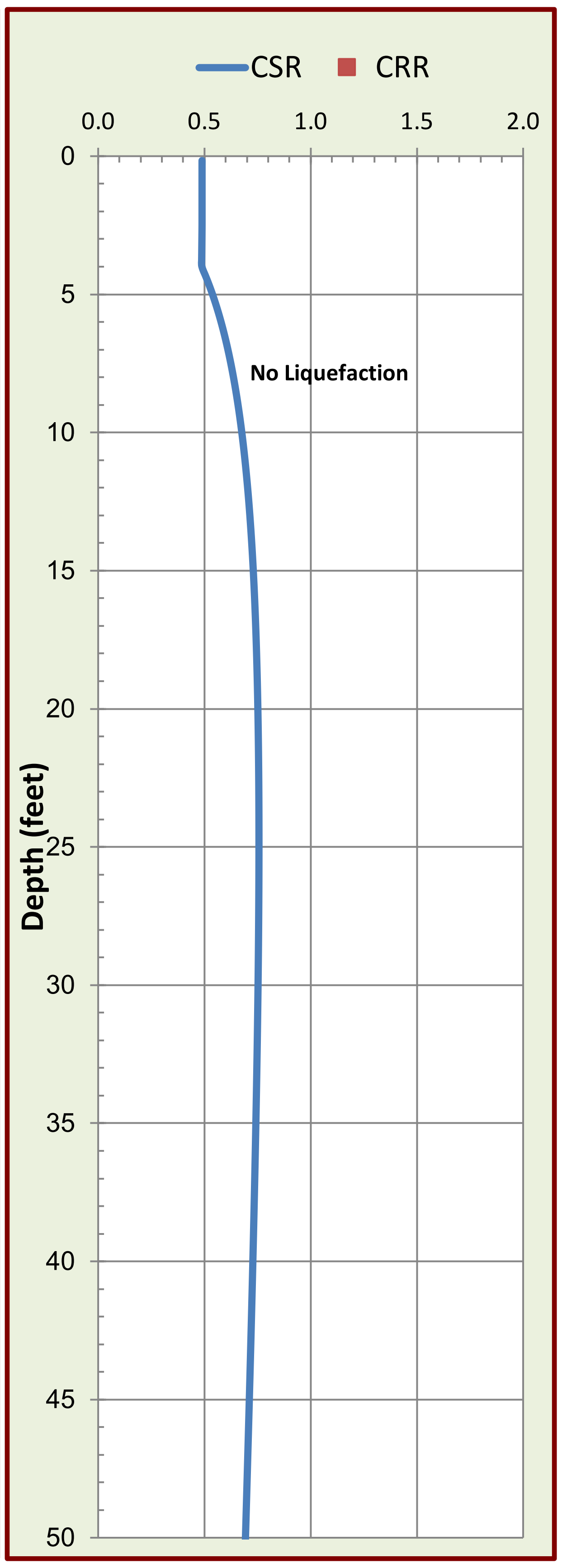
LDI[2] 0.00 **L/H** 153.3
LDI[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

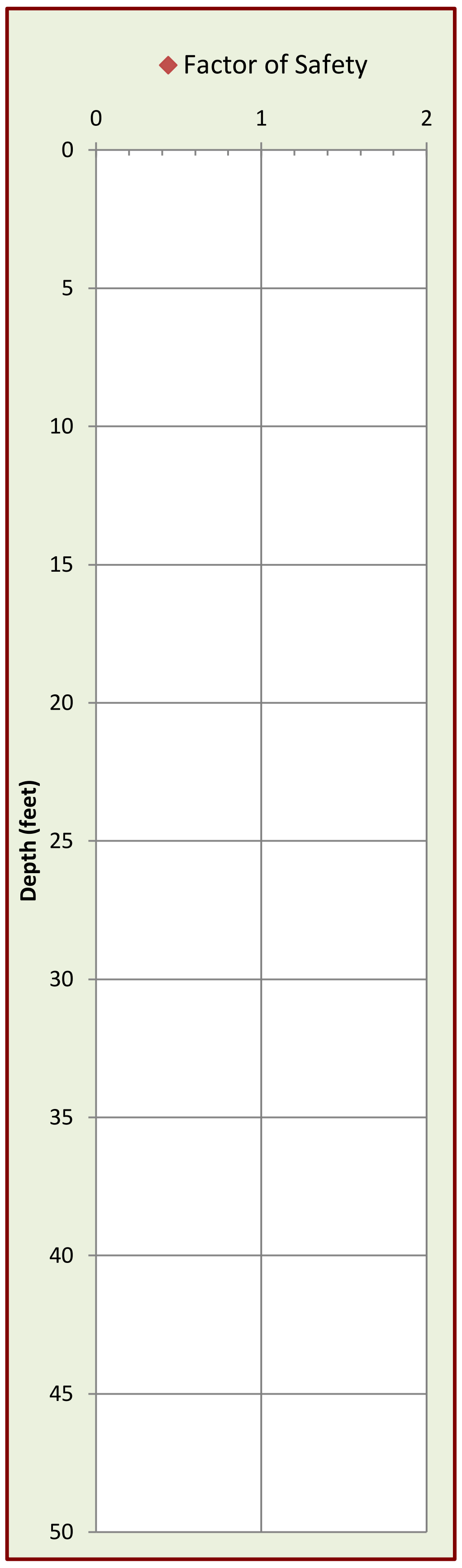
EXPECTED RANGE OF DISPLACEMENT

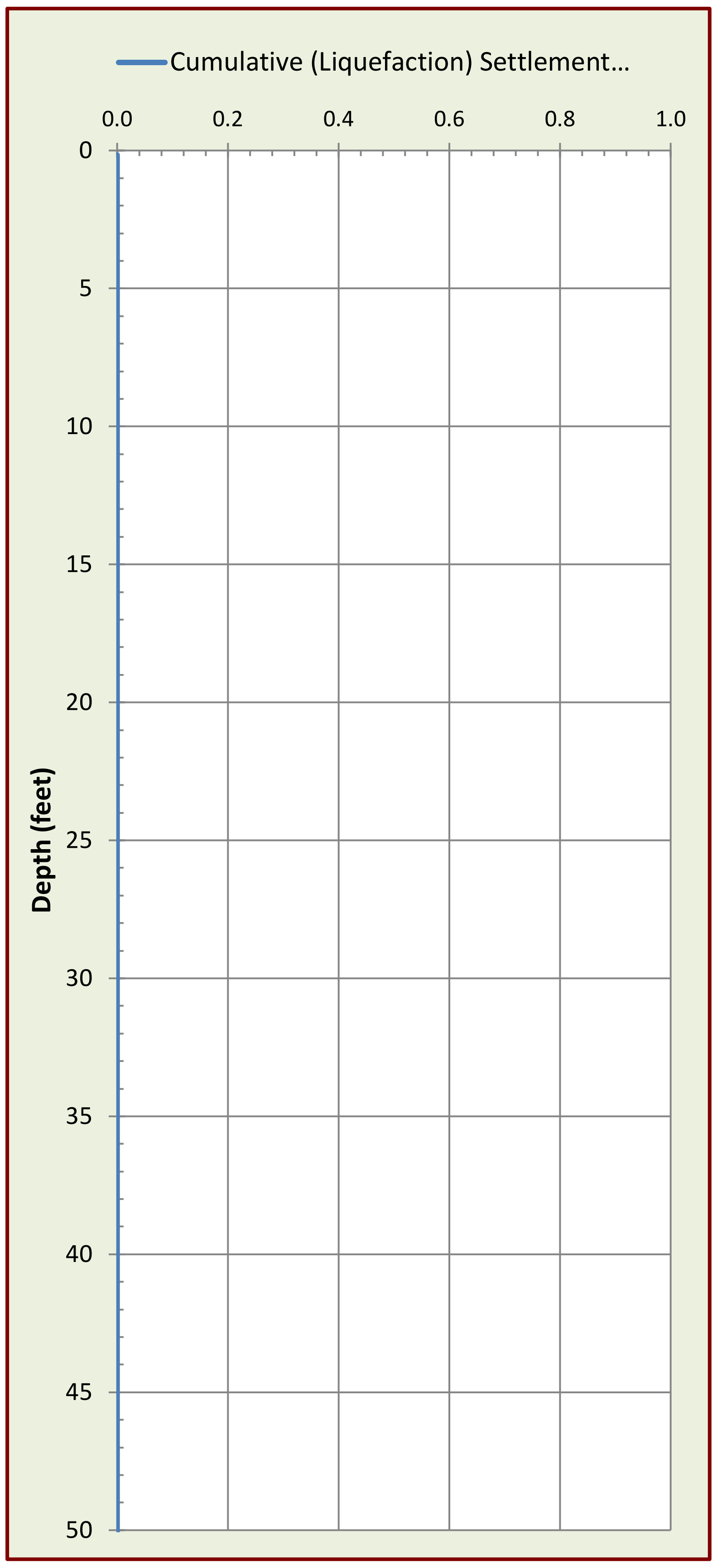
0.0 **to** **0.0** **feet**

[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: **Enterprise-Whitesell Industrial Bldg**

Project No.: **916-3-1**

Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**

Earthquake Magnitude (Mw): **7.1**

PGA (Amax): **0.752** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **5**

Design Water Depth (feet): **4**

Ave. Unit Weight Above GW (pcf): **125**

Ave. Unit Weight Below GW (pcf): **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.33 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 58.3

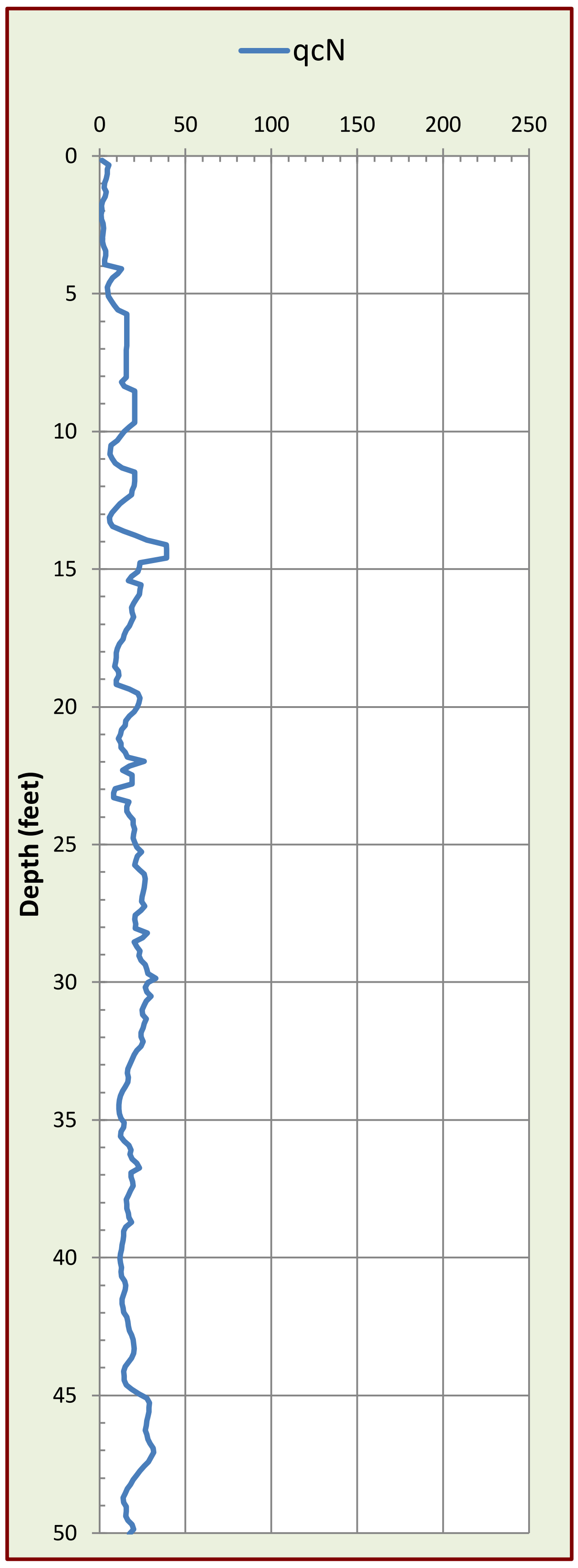
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

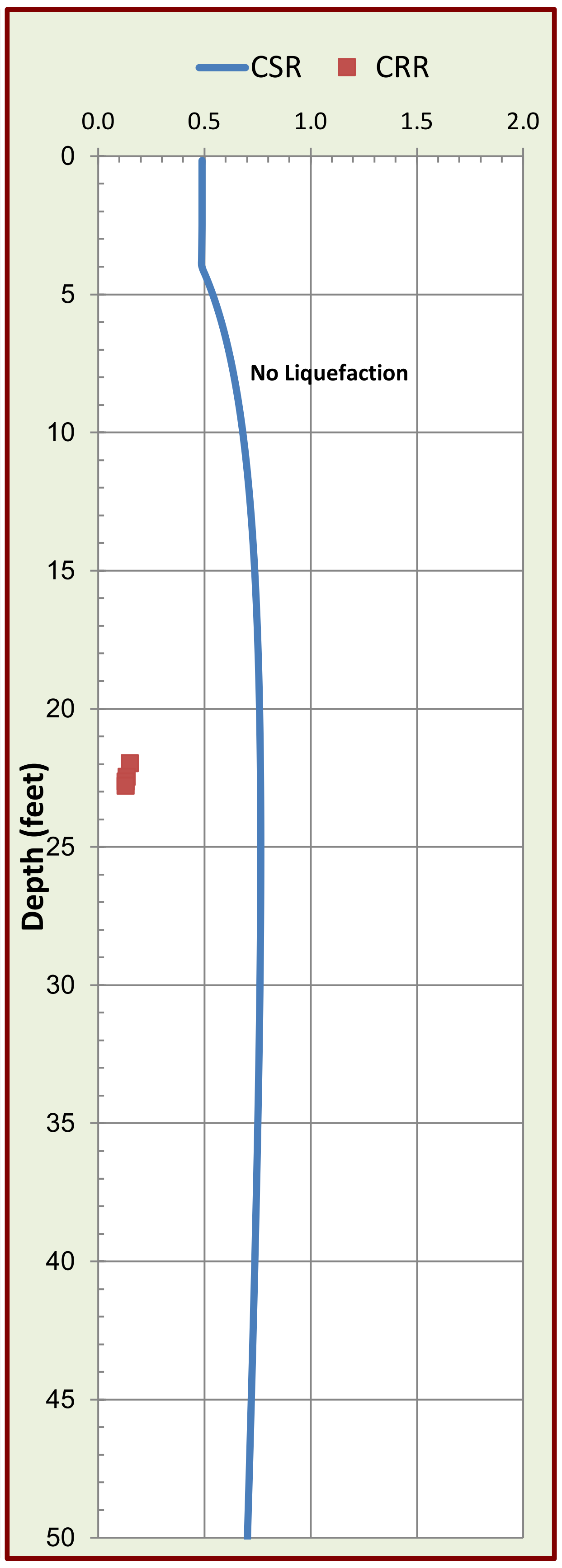
EXPECTED RANGE OF DISPLACEMENT

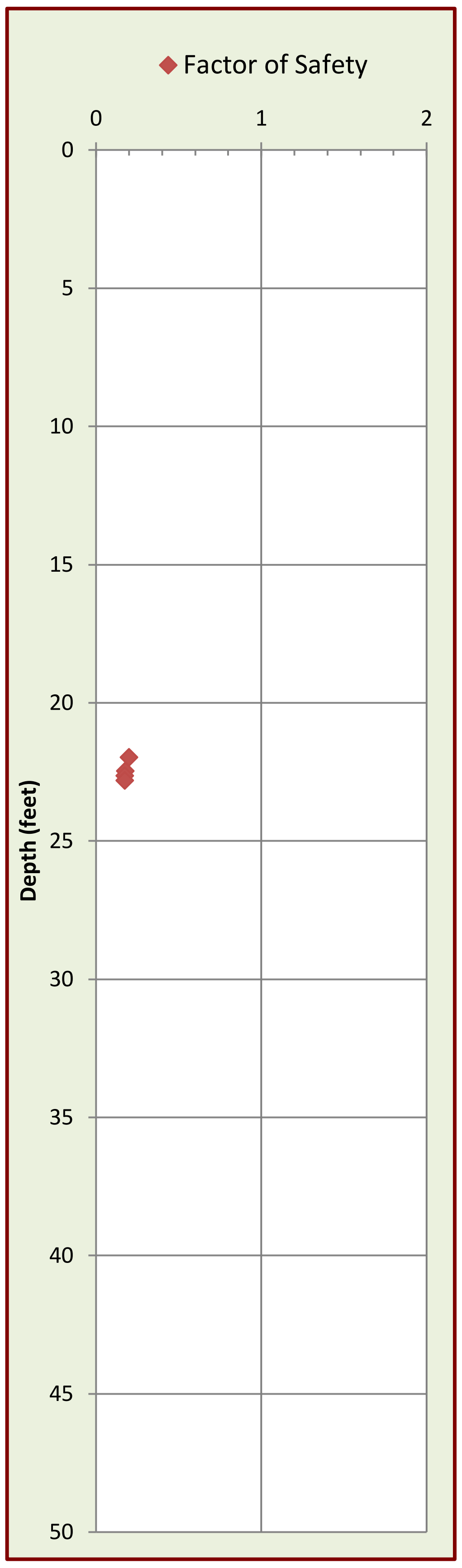
0.0 **to** 0.0 **feet**

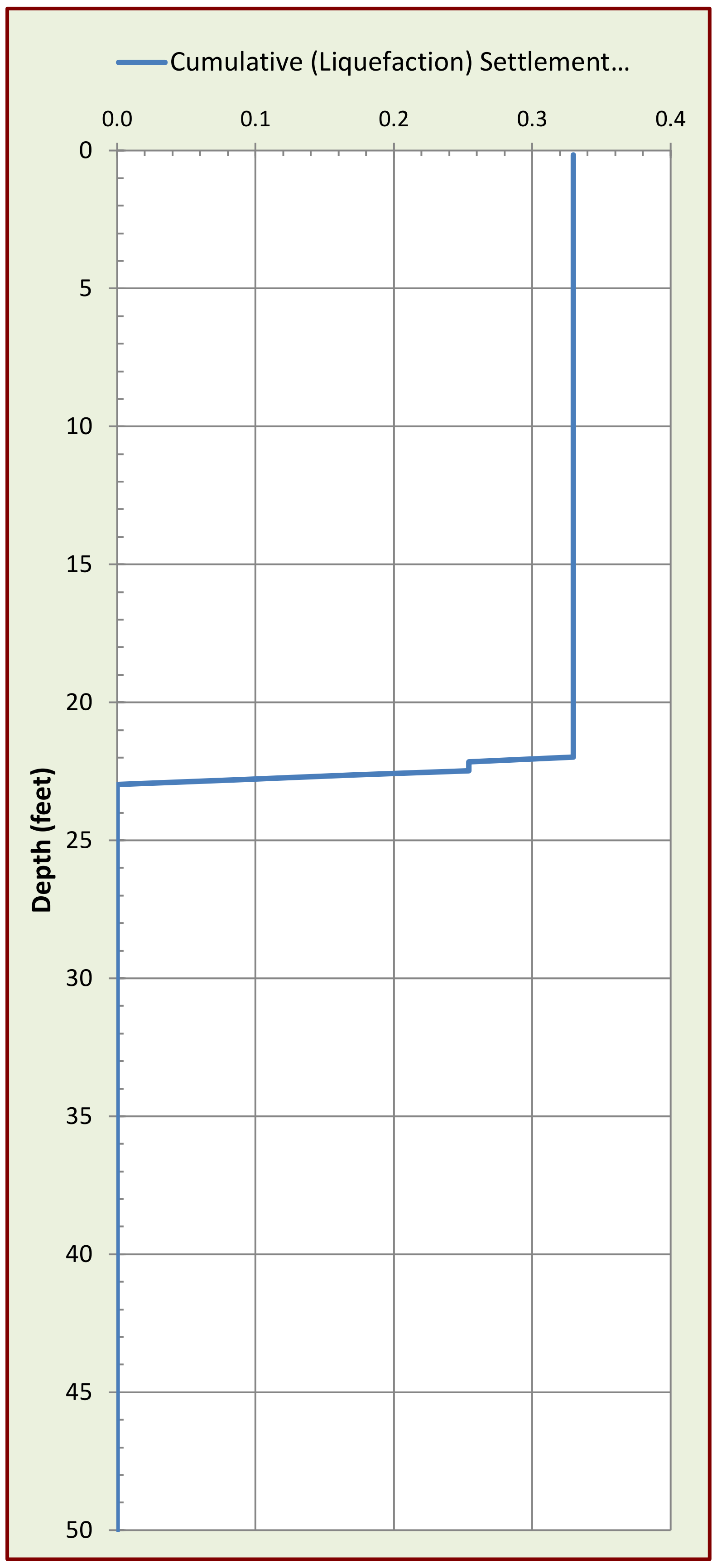
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

FIGURE **4E**

CPT NO. **5**

PROJECT/CPT DATA

Project Title **Enterprise-Whitesell Industrial Bldg**

Project No. **916-3-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **Hayward**

Earthquake Magnitude (Mw) **7.1**

PGA (Amax) **0.752** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5**

Design Water Depth (feet) **4**

Ave. Unit Weight Above GW (pcf) **122**

Ave. Unit Weight Below GW (pcf) **128**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.00 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.0 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.00 **L/H** 105.0

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

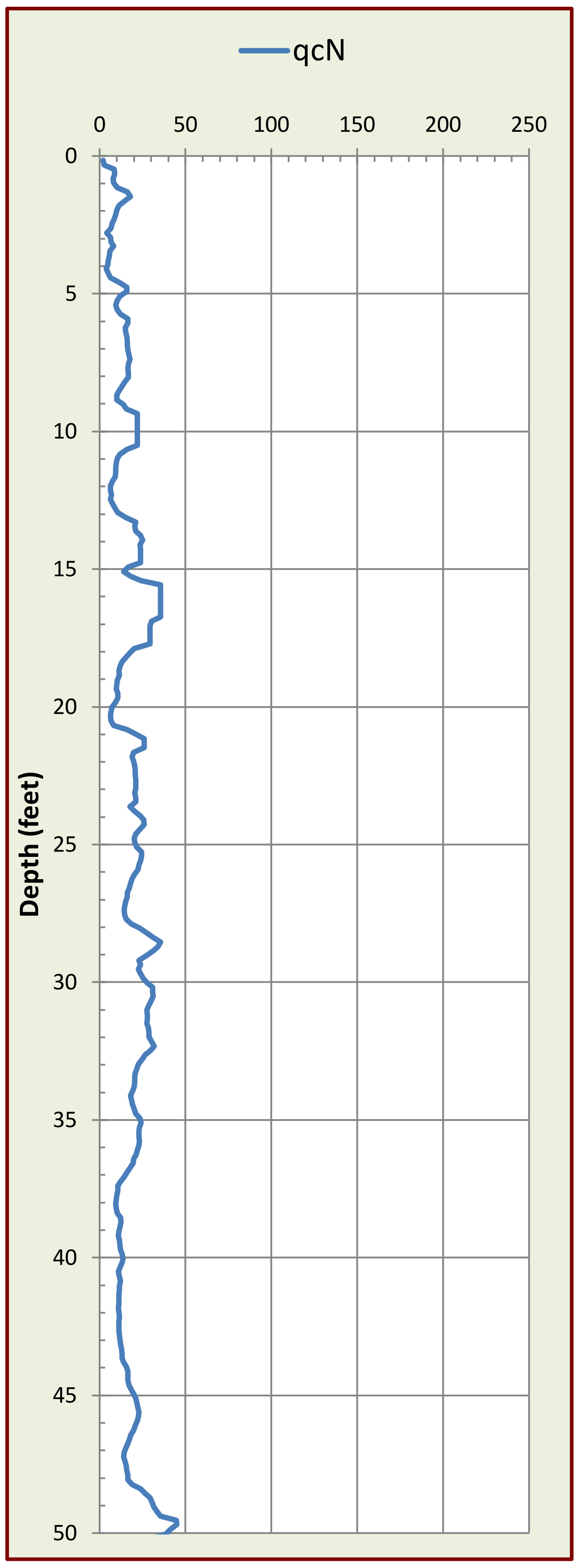
EXPECTED RANGE OF DISPLACEMENT

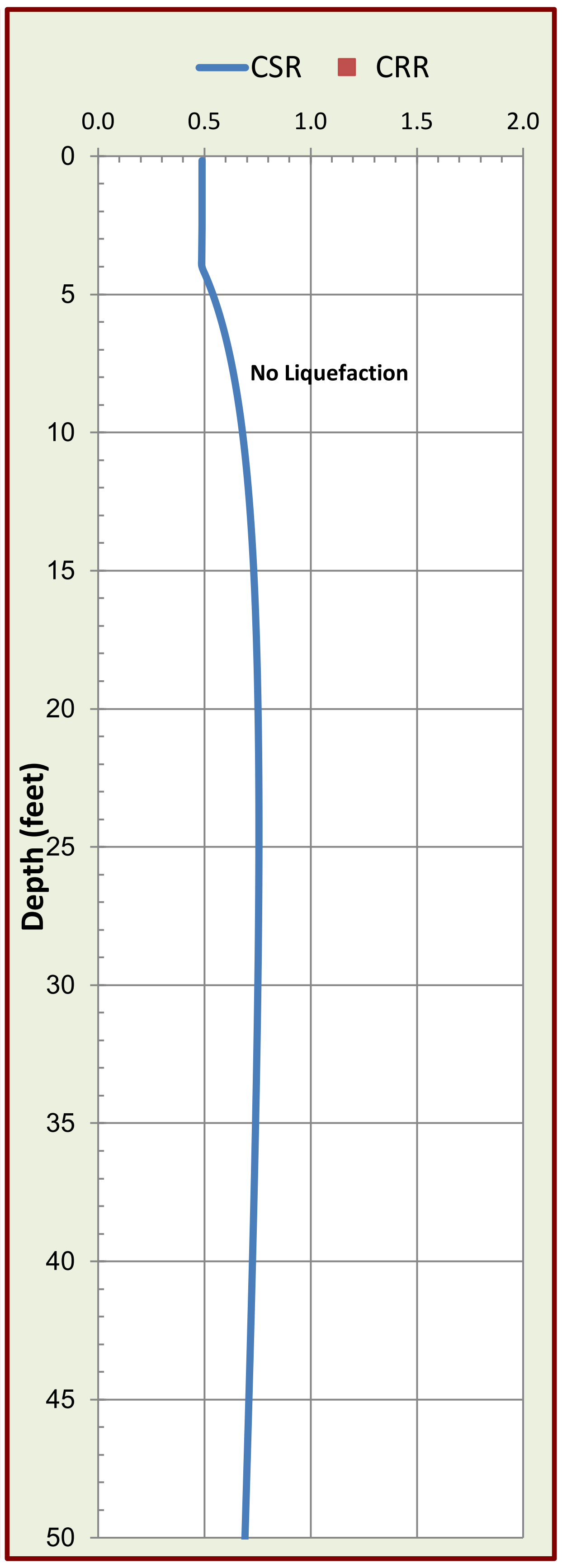
0.0 **to** **0.0** **feet**

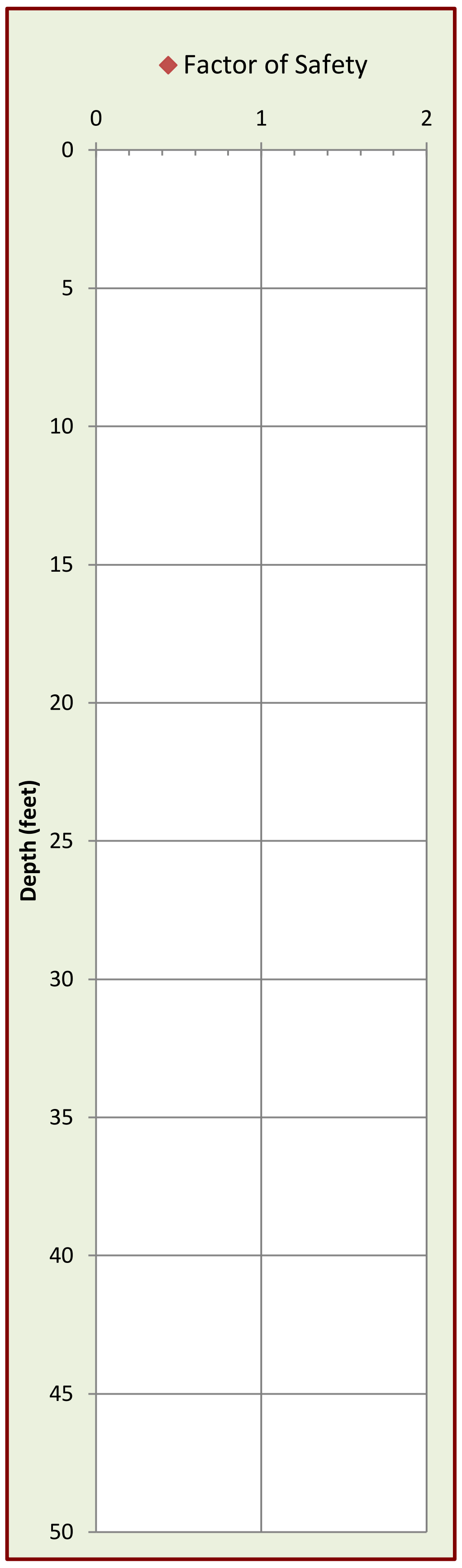
---

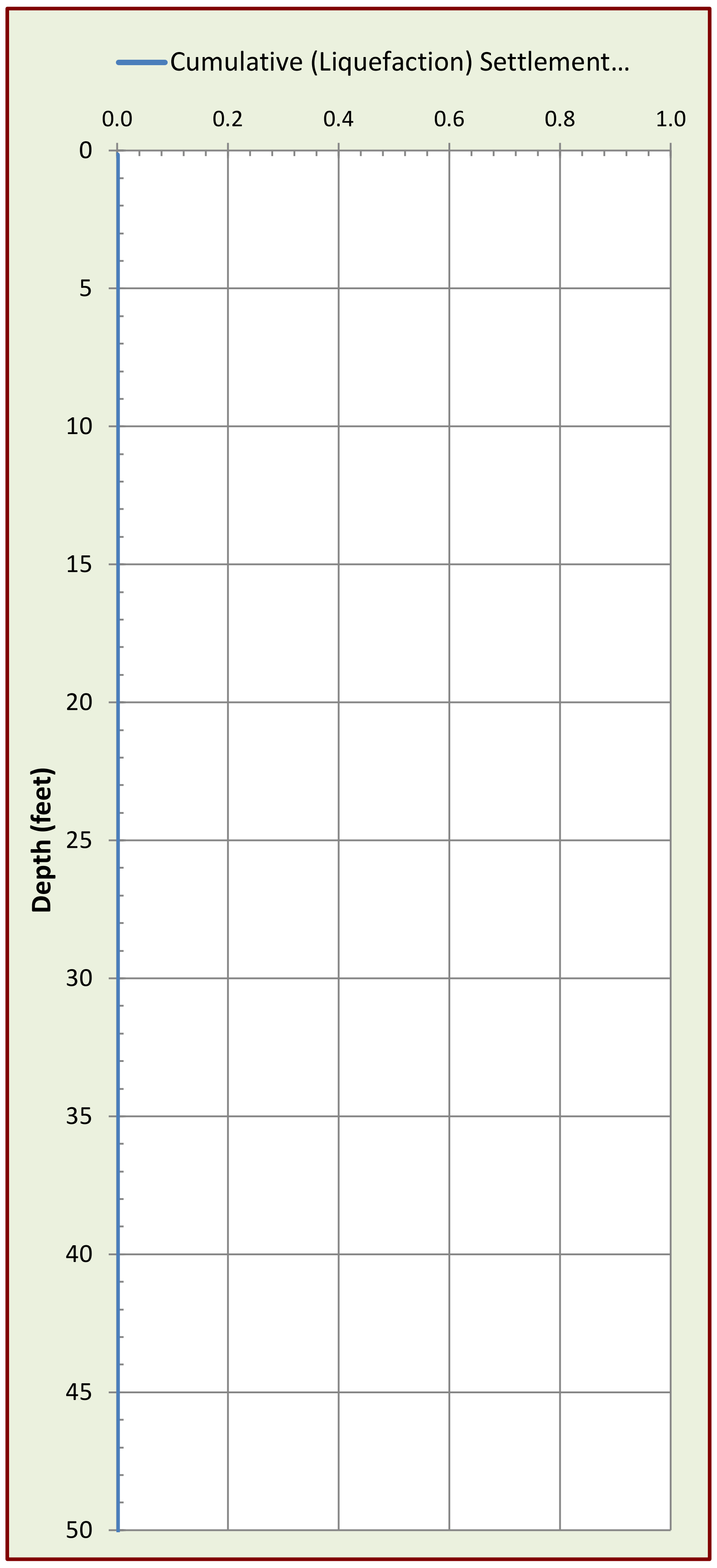
[1] Not Valid for L/H Values < 4 and > 40.

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









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. Five 8-inch-diameter exploratory borings were drilled on January 6 and 7, 2021 to depths of 25 to 45 feet. Five CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on December 23, 2020, to depths ranging from 50 to 135 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 and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries and other site features as references. 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. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	⁚	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		

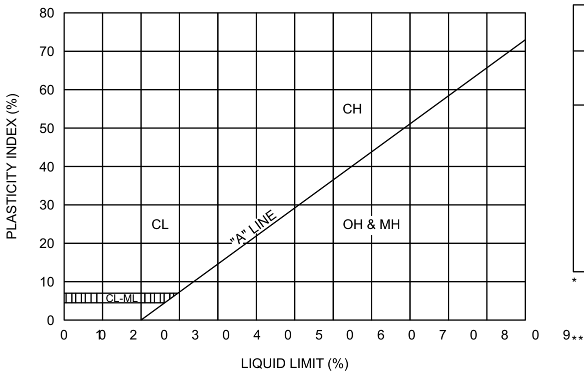
SAMPLER TYPES

☒	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 3636 Enterprise Road

PROJECT NUMBER 916-3-1

PROJECT LOCATION Hayward, CA

DATE STARTED 1/6/21 **DATE COMPLETED** 1/6/21

GROUND ELEVATION **BORING DEPTH** 31.5 ft.

DRILLING CONTRACTOR Exploration Geoservices Inc.

LATITUDE 37.632815° **LONGITUDE** -122.131656°

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

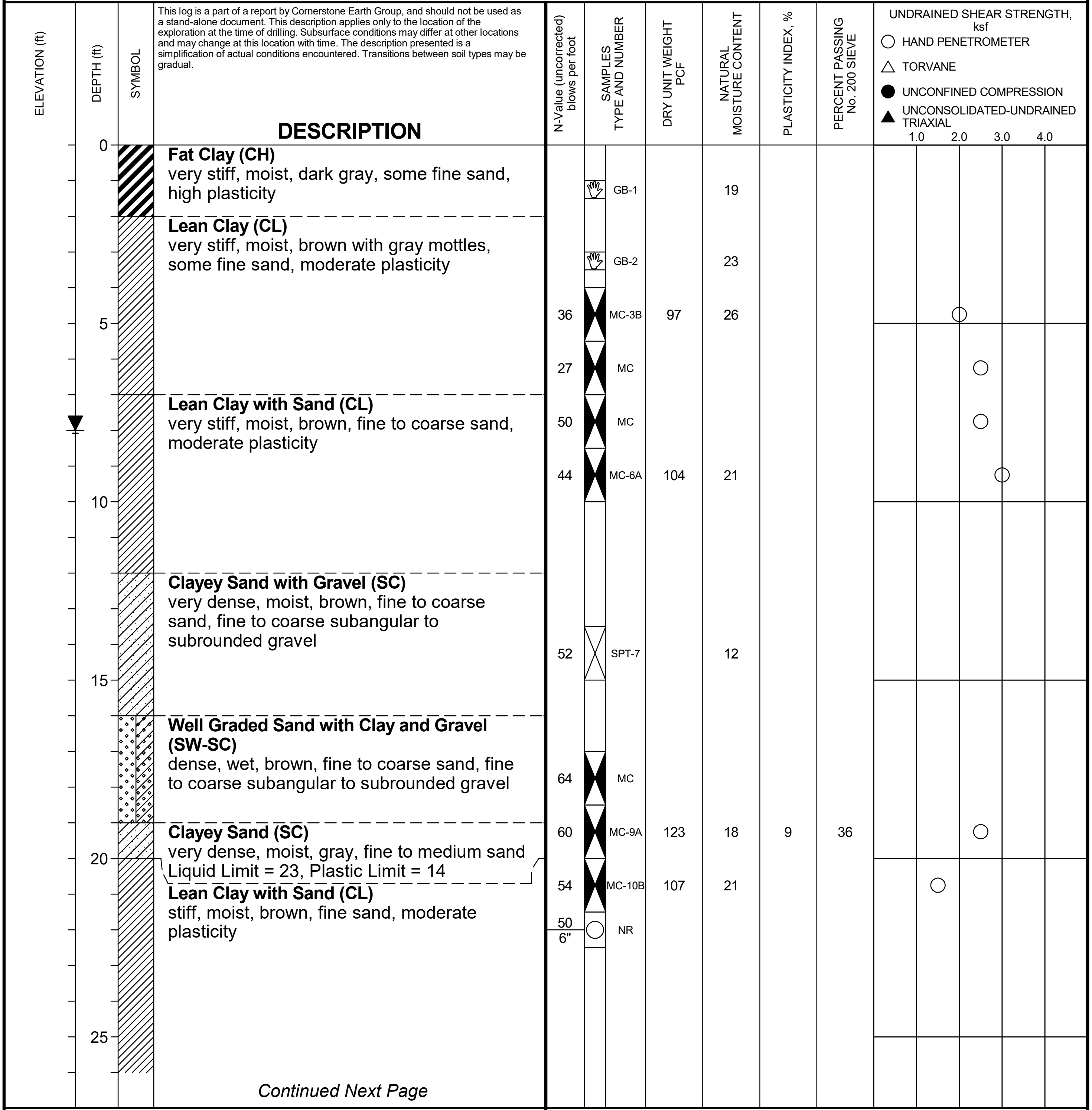
GROUND WATER LEVELS:

LOGGED BY RAH

▽ **AT TIME OF DRILLING** 8 ft.

NOTES

▼ **AT END OF DRILLING** 8 ft.



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

DESCRIPTION

Fat Clay (CH)
very stiff, moist, dark gray, some fine sand, high plasticity

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

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

Clayey Sand with Gravel (SC)
very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel

Well Graded Sand with Clay and Gravel (SW-SC)
dense, wet, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel

Clayey Sand (SC)
very dense, moist, gray, fine to medium sand
Liquid Limit = 23, Plastic Limit = 14

Lean Clay with Sand (CL)
stiff, moist, brown, 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
	GB-1		19		
	GB-2		23		
36	MC-3B	97	26		
27	MC				
50	MC				
44	MC-6A	104	21		
52	SPT-7		12		
64	MC				
60	MC-9A	123	18	9	36
54	MC-10B	107	21		
50 6"	NR				

Continued Next Page

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME 3636 Enterprise Road

PROJECT NUMBER 916-3-1

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.

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)** stiff, moist, brown, fine sand, moderate plasticity							
		▨		50 5"	○ NR					
	30	▨			⧗ MC					○
			Bottom of Boring at 31.5 feet.							
	35									
	40									
	45									
	50									
	55									

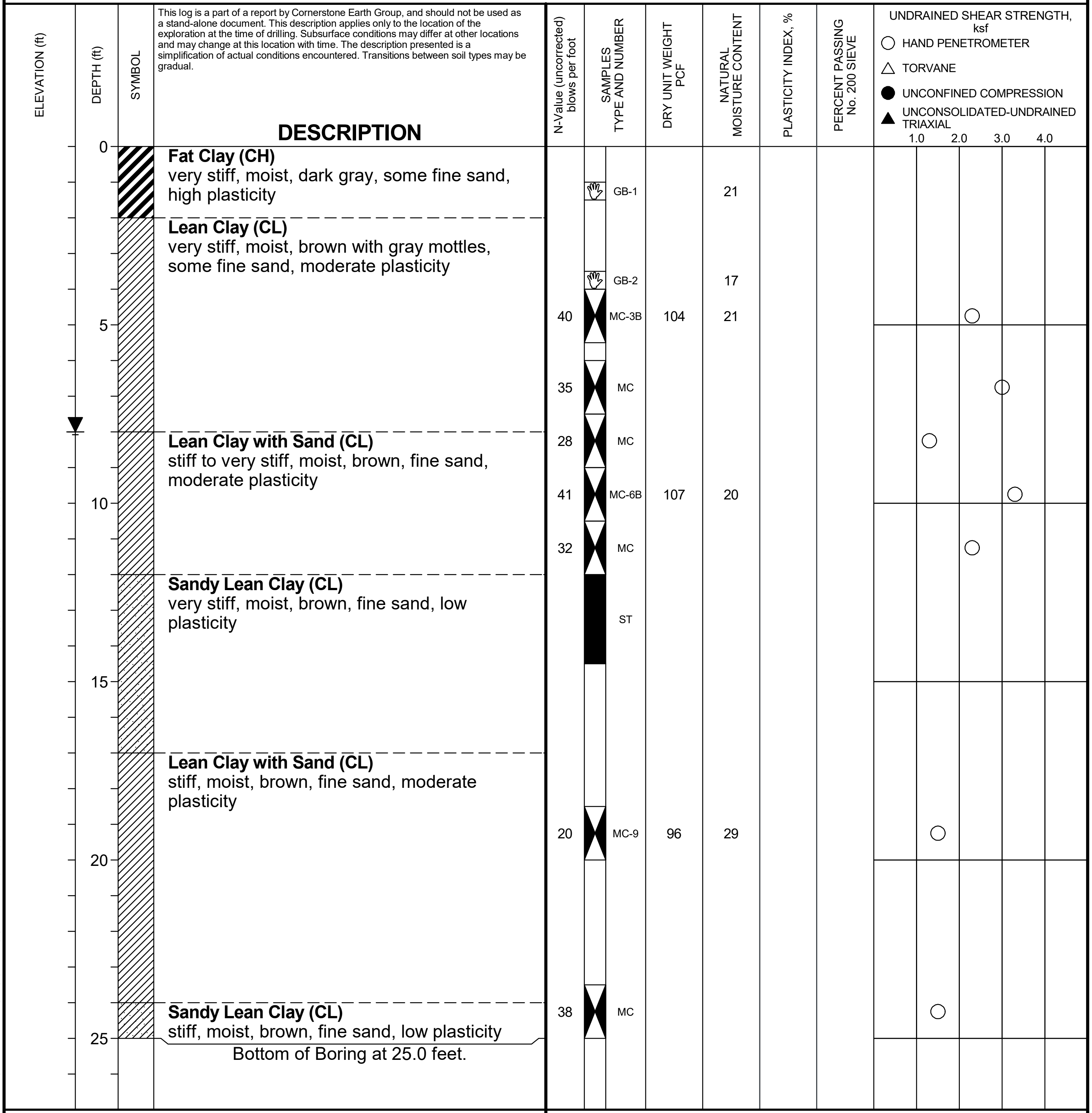
CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 1/21/21 07:57 - P:\DRAFTING\GINT FILES\916-3-1 3636 ENTERPRISE ROAD.GPJ

BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 3636 Enterprise Road
PROJECT NUMBER 916-3-1
PROJECT LOCATION Hayward, CA
DATE STARTED 1/6/21 **DATE COMPLETED** 1/6/21
GROUND ELEVATION ____ **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE ____ **LONGITUDE** ____
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 8 ft.
NOTES ____
▼ **AT END OF DRILLING** 8 ft.



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

ELEVATION (ft)	DEPTH (ft)	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		**Fat Clay (CH)** very stiff, moist, dark gray, some fine sand, high plasticity		GB-1		21		
			Lean Clay (CL) very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity		GB-2		17		
	5			40	MC-3B	104	21		
				35	MC				
			Lean Clay with Sand (CL) stiff to very stiff, moist, brown, fine sand, moderate plasticity	28	MC				
	10			41	MC-6B	107	20		
				32	MC				
			Sandy Lean Clay (CL) very stiff, moist, brown, fine sand, low plasticity		ST				
	15								
			Lean Clay with Sand (CL) stiff, moist, brown, fine sand, moderate plasticity	20	MC-9	96	29		
	20								
			Sandy Lean Clay (CL) stiff, moist, brown, fine sand, low plasticity	38	MC				
	25		Bottom of Boring at 25.0 feet.						

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

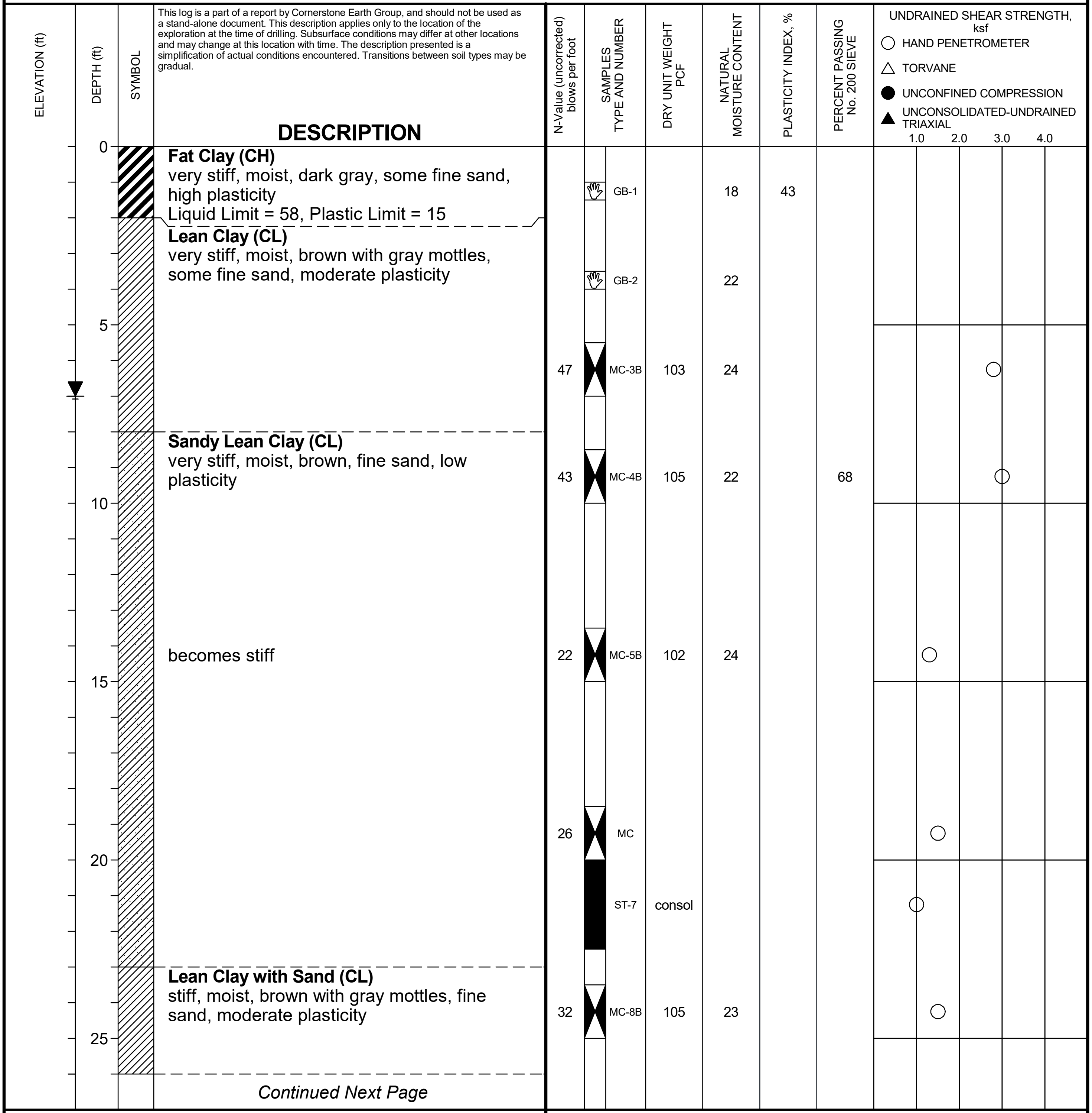
BORING NUMBER EB-3

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 3636 Enterprise Road
PROJECT NUMBER 916-3-1
PROJECT LOCATION Hayward, CA
DATE STARTED 1/6/21 **DATE COMPLETED** 1/6/21
GROUND ELEVATION **BORING DEPTH** 45 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.632320° **LONGITUDE** -122.131209°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 7 ft.
NOTES
▼ **AT END OF DRILLING** 7 ft.

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



DESCRIPTION

Fat Clay (CH)
very stiff, moist, dark gray, some fine sand, high plasticity
Liquid Limit = 58, Plastic Limit = 15

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

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

becomes stiff

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

Continued Next Page

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	GB-1		18	43	
	GB-2		22		
47	MC-3B	103	24		
43	MC-4B	105	22		68
22	MC-5B	102	24		
26	MC				
	ST-7	consol			
32	MC-8B	105	23		

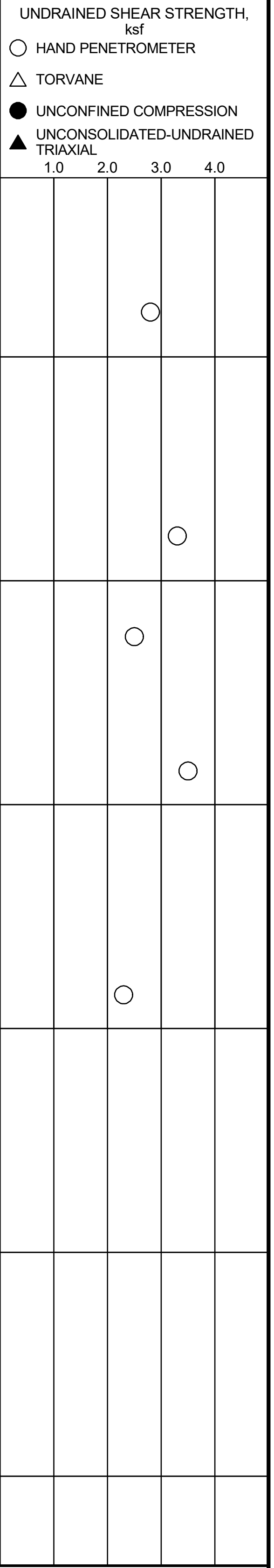
BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME 3636 Enterprise Road
PROJECT NUMBER 916-3-1
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
		▨	**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, moderate plasticity						
				50 6"	MC				
	30								
				50 6"	MC-10B	105	22		
	35				ST				
		▨	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, low plasticity	33	MC				
	40								
		▨	**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, moderate plasticity	50 6"	MC-13B	103	25		
	45		Bottom of Boring at 45.0 feet.						
	50								
	55								



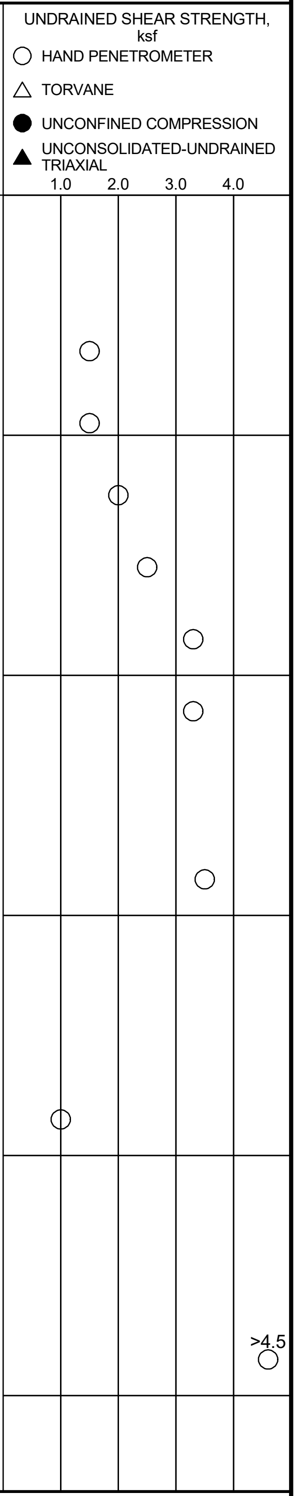
BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME 3636 Enterprise Road
PROJECT NUMBER 916-3-1
PROJECT LOCATION Hayward, CA
DATE STARTED 1/7/21 **DATE COMPLETED** 1/7/21
GROUND ELEVATION **BORING DEPTH** 37.5 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 6.5 ft.
NOTES
▼ **AT END OF DRILLING** 6.5 ft.

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	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	**Fat Clay (CH)** very stiff, moist, dark gray, some fine sand, high plasticity						
				GB-1		19		
		Lean Clay (CL) stiff, moist, brown with gray mottles, some fine sand, moderate plasticity						
				GB-2		26		
			22	MC-3B	100	25		
	5							
▼		becomes very stiff	43	MC-4B	97	27		
			42	MC				
		Lean Clay with Sand (CL) very stiff, moist, brown, fine sand, moderate plasticity						
			42	MC-6B	104	22		
	10							
			39	MC				
		Sandy Lean Clay (CL) stiff, moist, brown, fine sand, low plasticity						
			66	MC				
	15							
		Lean Clay with Sand (CL) very stiff, moist, brown, fine sand, moderate plasticity						
			32	MC-9B	102	26		
	20							
		Lean Clay (CL) hard, moist, brown, some fine sand, moderate plasticity						
			50 6"	MC				
	25							



Continued Next Page

BORING NUMBER EB-4

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 3636 Enterprise Road

PROJECT NUMBER 916-3-1

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.

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
		▨	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, some cemented clay nodules, low plasticity							
		▨		50 6"	MC-11B		13			○
	30	▨								
		▨	becomes stiff	44	MC-12B	112	17			○
	35	▨								
		▨			ST					○
			Bottom of Boring at 37.5 feet.							
	40									
	45									
	50									
	55									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PAGE 1 OF 2

PROJECT NAME 3636 Enterprise Road
PROJECT NUMBER 916-3-1
PROJECT LOCATION Hayward, CA
DATE STARTED 1/7/21 **DATE COMPLETED** 1/7/21
GROUND ELEVATION **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RAH
▽ **AT TIME OF DRILLING** 10 ft.
NOTES
▼ **AT END OF DRILLING** 8 ft.

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

DESCRIPTION

Fat Clay (CH)
very stiff, moist, dark gray, some fine sand, high plasticity

Lean Clay (CL)
very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity
Liquid Limit = 41, Plastic Limit = 16

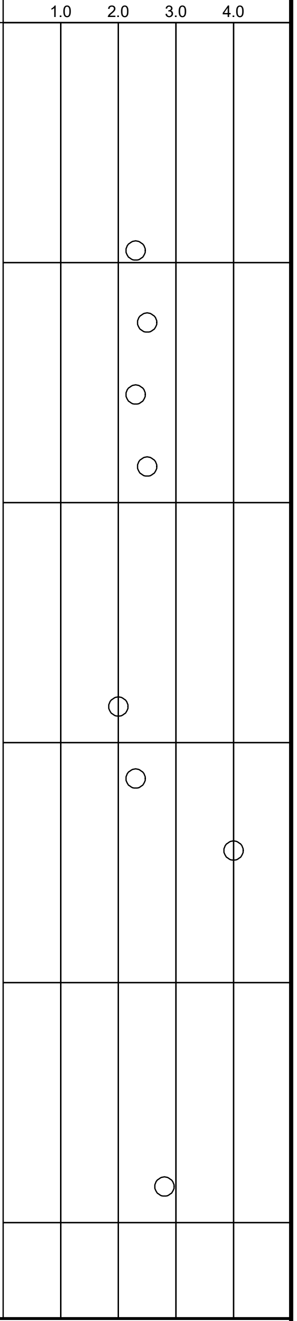
Lean Clay with Sand (CL)
very stiff, moist, brown, fine sand, moderate plasticity

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

Continued Next Page

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	GB-1		16		
	GB-2		18		
38	MC-3B	105	22	25	
38	MC				
42	MC				
42	MC-6B	104	21		
42	MC				
50 6"	MC-8B	109	21		
42	MC				
24	MC-10		22		

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



CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PAGE 2 OF 2

PROJECT NAME 3636 Enterprise Road

PROJECT NUMBER 916-3-1

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.

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 (CL)** very stiff, moist, brown, some fine sand, moderate plasticity							
		▨		76	⧗ MC					○
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

Middle Earth
GEO TESTING INC.

Cornerstone Earth Group

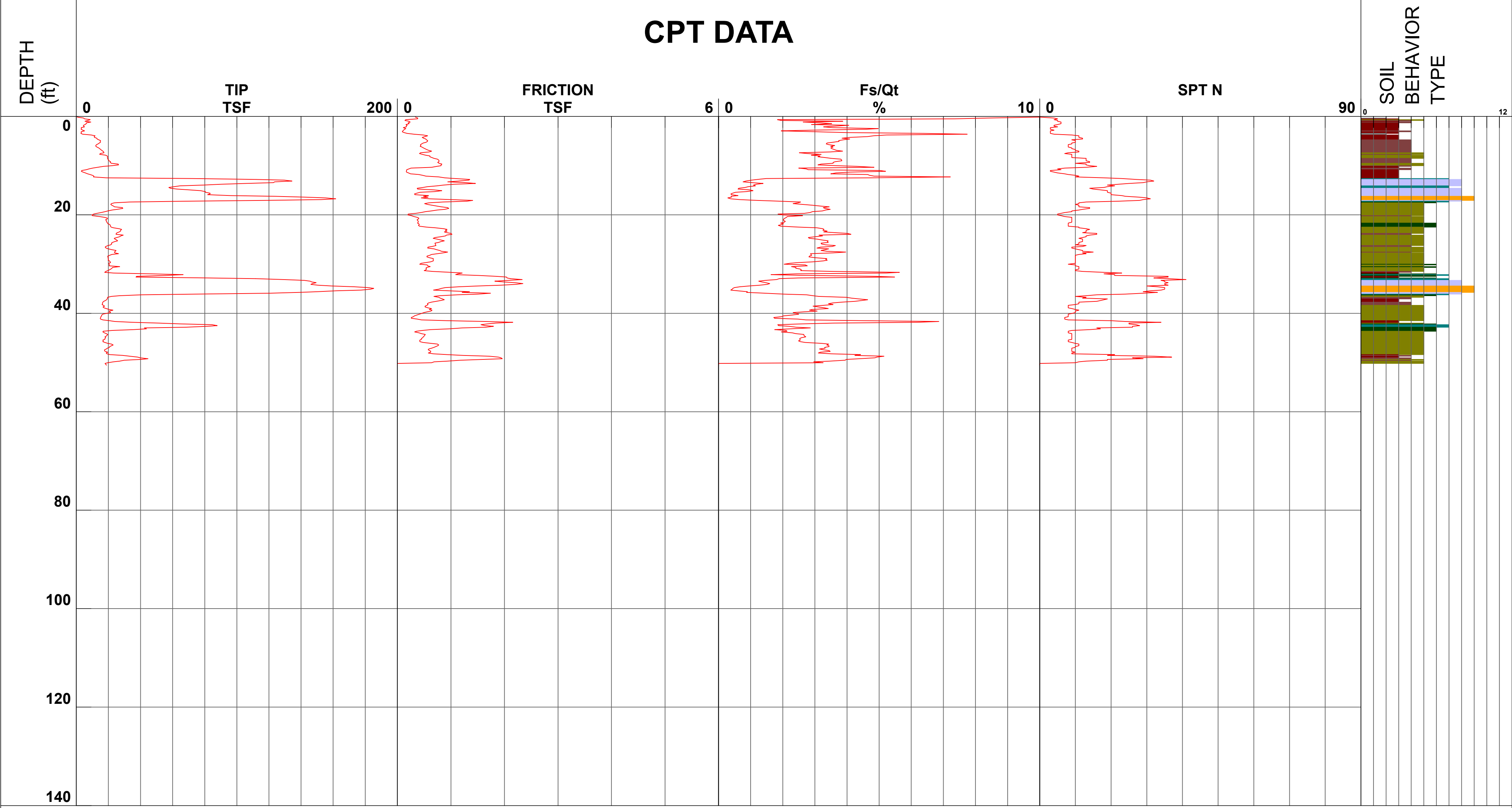
Project Enterprise-Whitesell Industrial BLDG
Job Number 916-3-1
Hole Number CPT-01
EST GW Depth During Test 5.00 ft

Operator JM-AJ
Cone Number DDG1530
Date and Time 12/23/2020 1:26:56 PM

Filename SDF(409).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8

CPT DATA



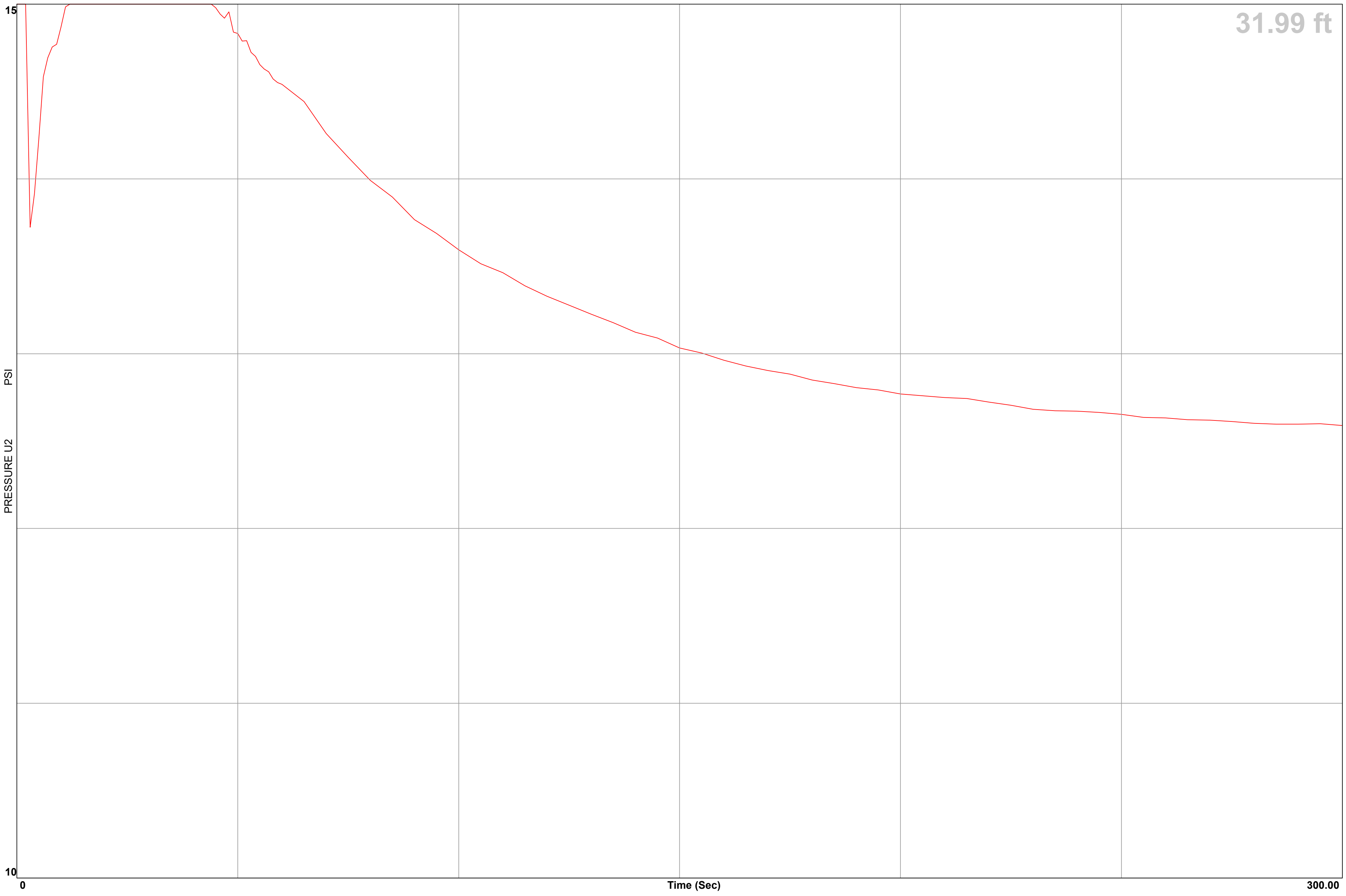
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	Enterprise-Whitesell Industrial BLDG	**Operator**	JM-AJ		
Job Number	916-3-1	**Cone Number**	DDG1530	**GPS**	
Hole Number	CPT-01	**Date and Time**	12/23/2020 1:26:56 PM		
Equilized Pressure	12.5	**EST GW Depth During Test**	2.9		

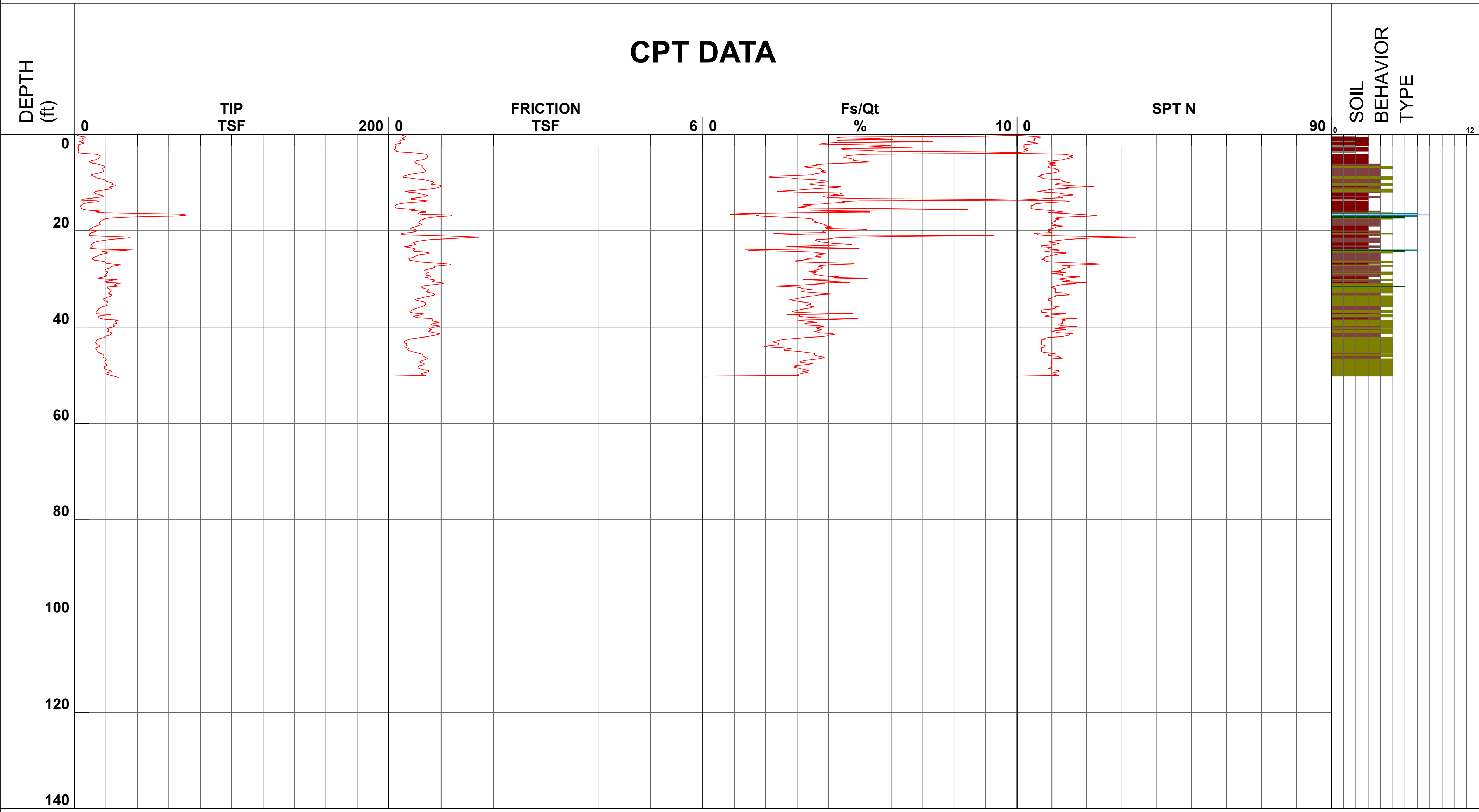


Page 1 of 1

Cornerstone Earth Group

Project Enterprise-Whitesell Industrial BLDG
Job Number 916-3-1
Hole Number CPT-02
EST GW Depth During Test 7.00 ft
Operator JM-AJ
Cone Number DDG1530
Date and Time 12/23/2020 3:15:22 PM
Filename SDF(410).cpt
GPS
Maximum Depth 50.52 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

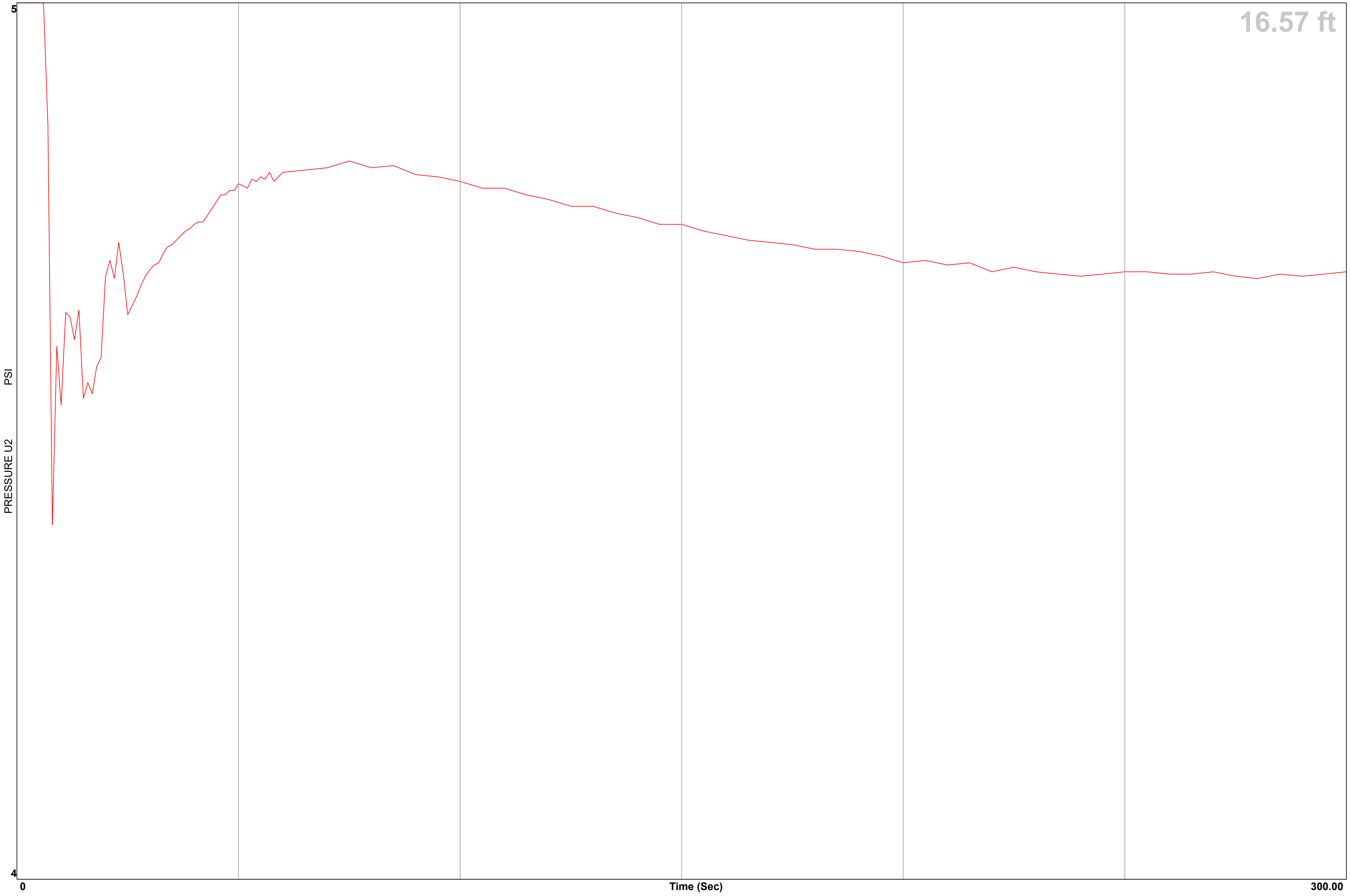
🟥 10 - gravelly sand to sand
🟦 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

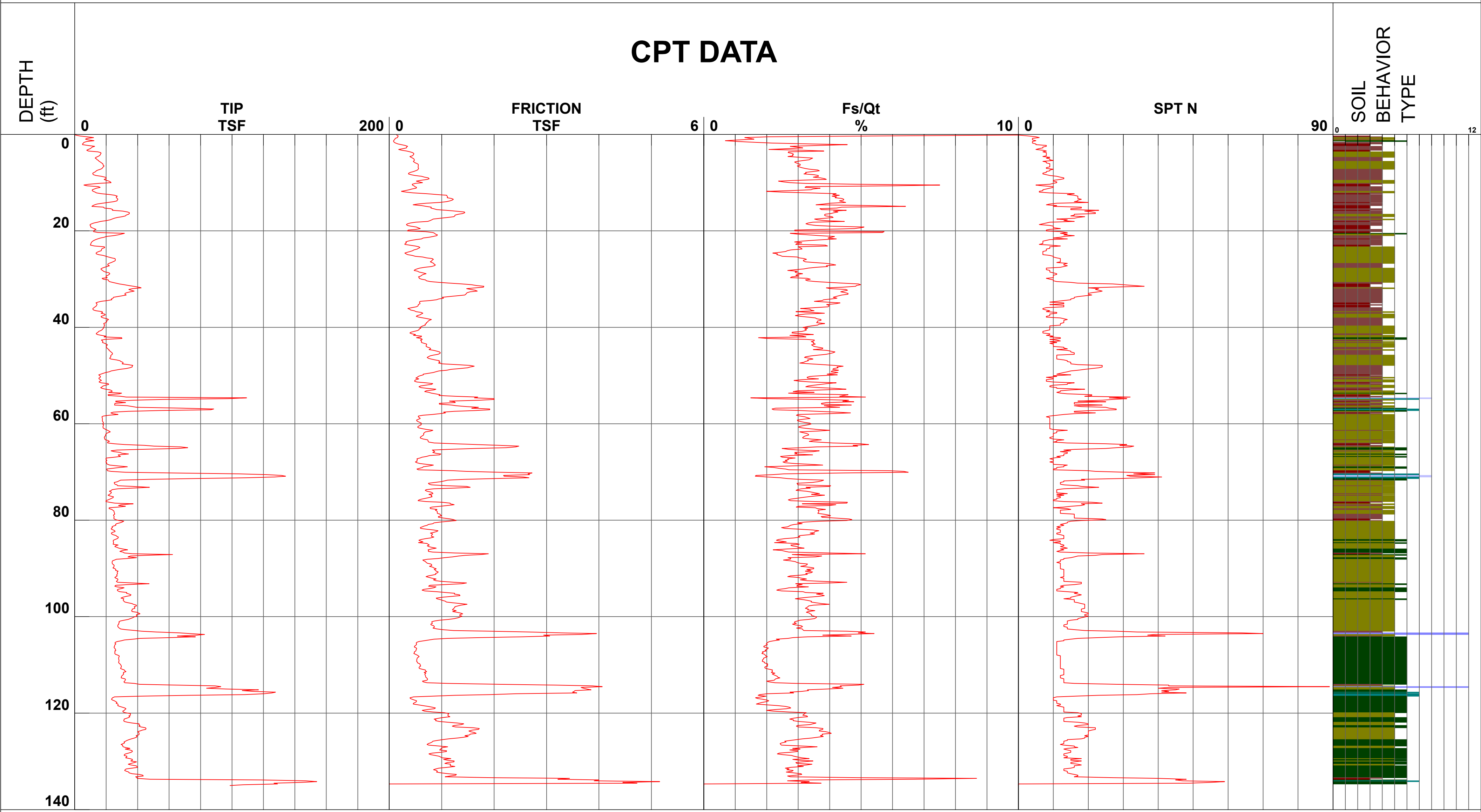
Location	Enterprise-Whitesell Industrial BLDG	Operator	JM-AJ		
Job Number	916-3-1	Cone Number	DDG1530	GPS	
Hole Number	CPT-02	Date and Time	12/23/2020 3:15:22 PM		
Equilized Pressure	4.6	EST GW Depth During Test	5.7		



Cornerstone Earth Group

Project	Enterprise-Whitesell Industrial BLDG	Operator	JM-AJ	Filename	SDF(407).cpt
Job Number	916-3-1	Cone Number	DDG1530	GPS	
Hole Number	CPT-03	Date and Time	12/23/2020 10:01:48 AM	Maximum Depth	135.00 ft
EST GW Depth During Test			5.00 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 (*)

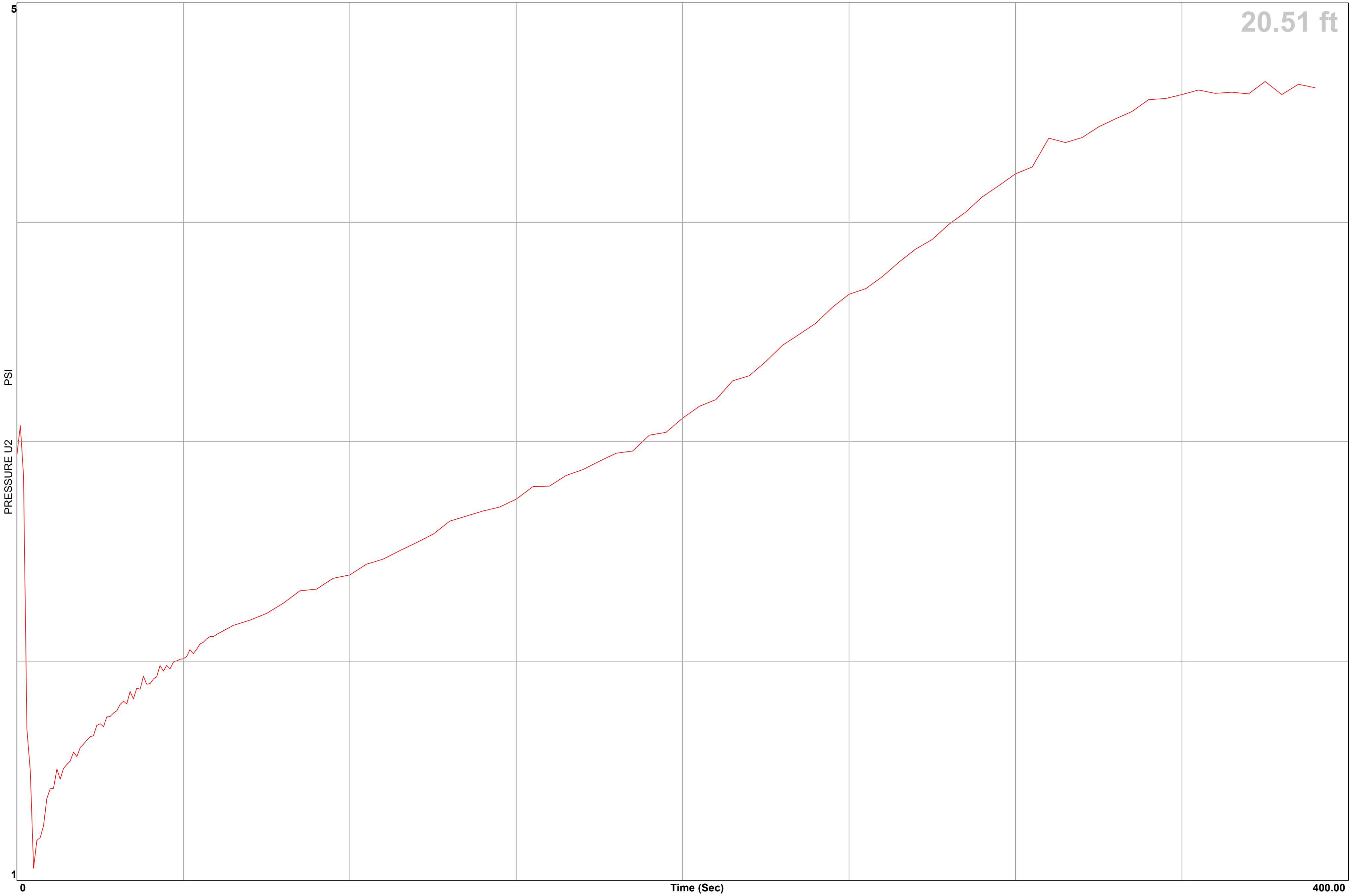
Cone Size 15cm squared

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

Cornerstone Earth Group

Location	Enterprise-Whitesell Industrial BLDG	Operator	JM-AJ
Job Number	916-3-1	Cone Number	DDG1530
Hole Number	CPT-03	Date and Time	12/23/2020 10:01:48 AM
Equilized Pressure	4.6	EST GW Depth During Test	9.8

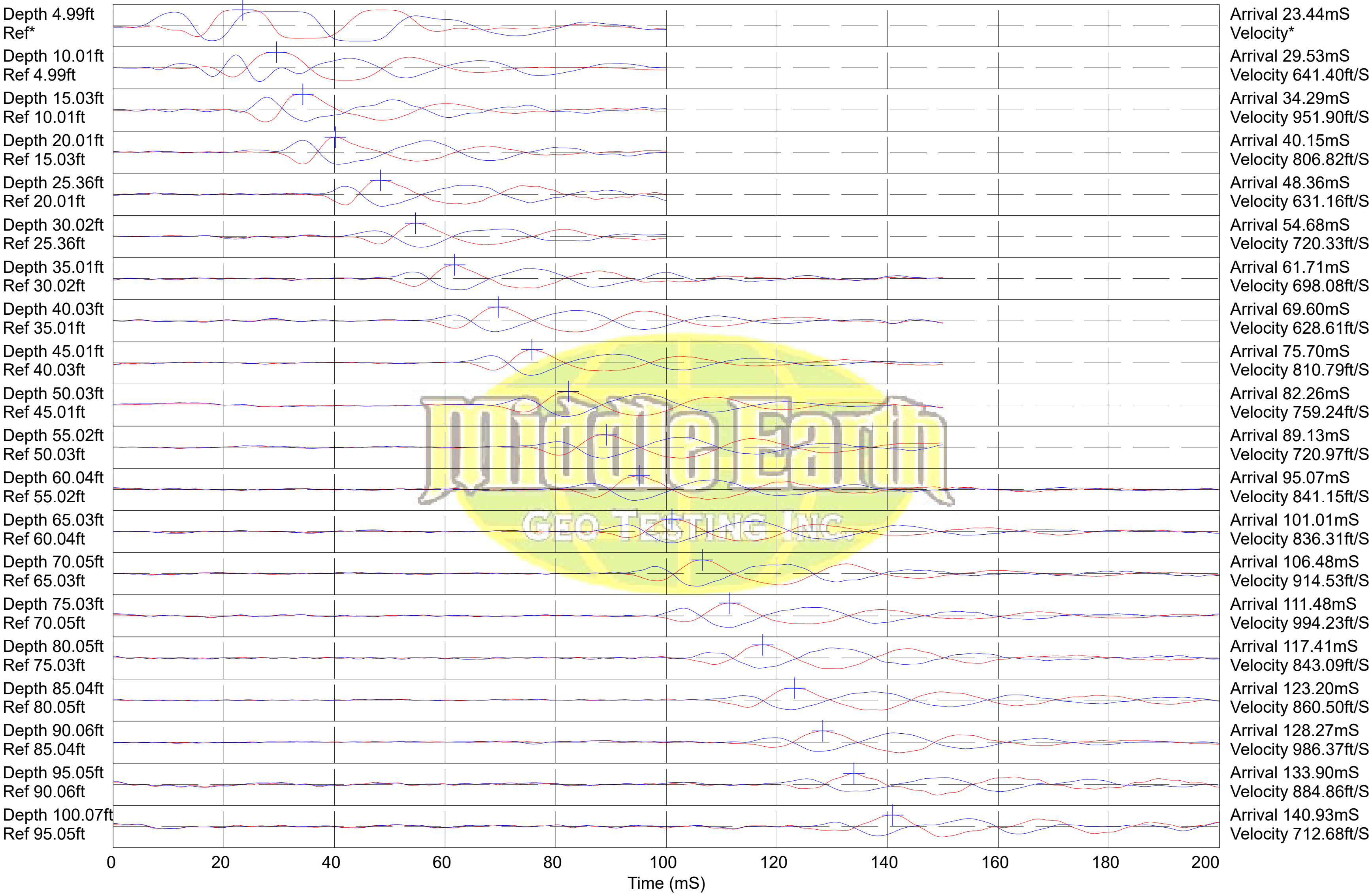
GPS



CPT-03

Cornerstone Earth Group

Enterprise-Whitesell Industrial BLDG



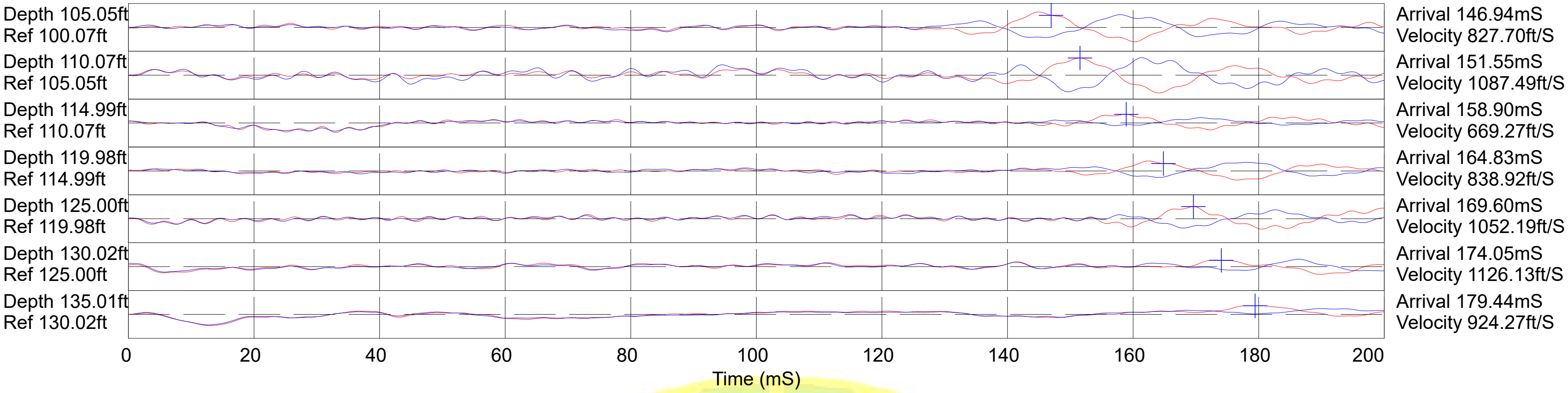
Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

CPT-03

Cornerstone Earth Group

Enterprise-Whitesell Industrial BLDG



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

COMMENT:

Cornerstone Earth Group

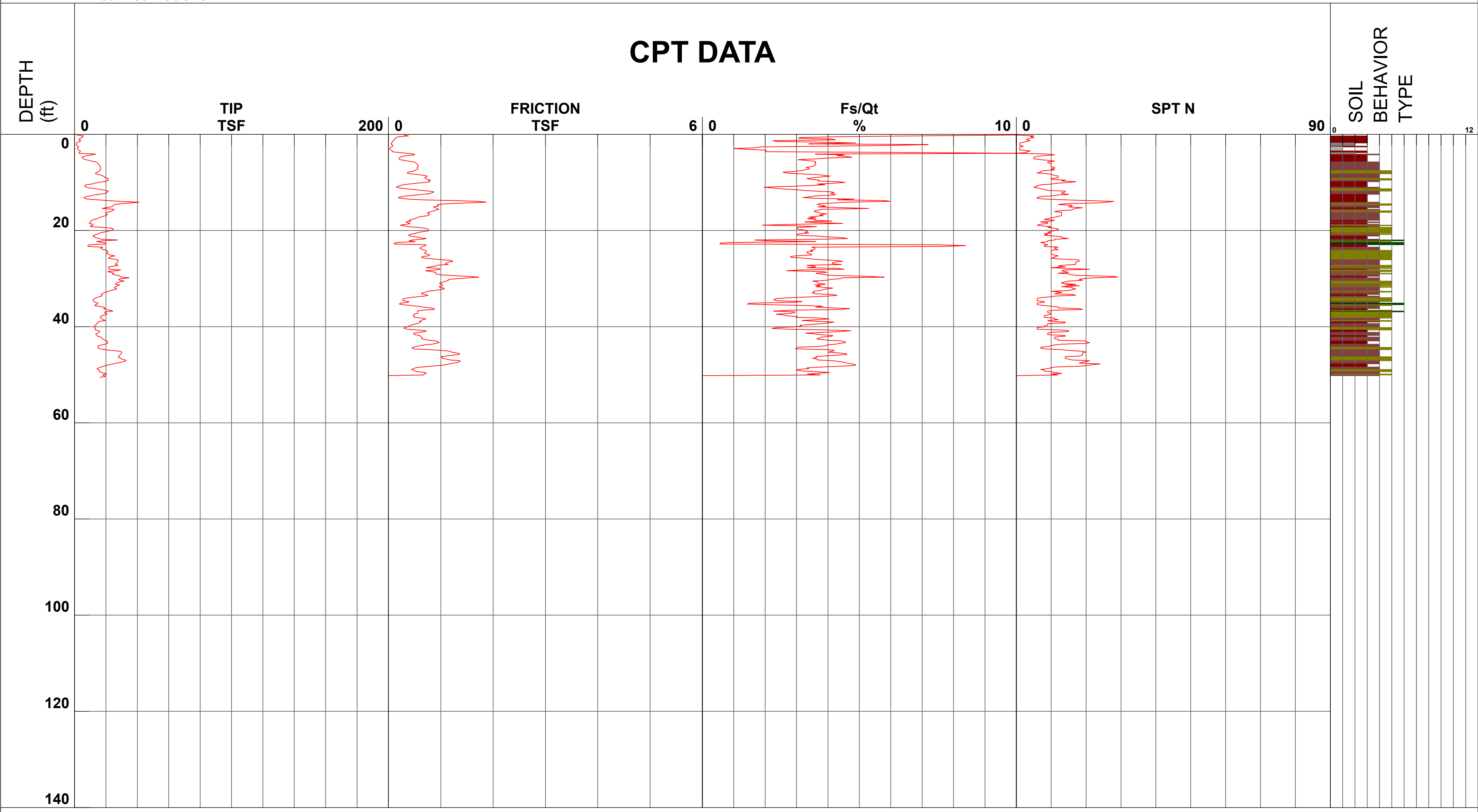
Middle Earth
GEO TESTING INC.

Project Enterprise-Whitesell Industrial BLDG
Job Number 916-3-1
Hole Number CPT-04
EST GW Depth During Test 4.00 ft

Operator JM-AJ
Cone Number DDG1530
Date and Time 12/23/2020 12:14:12 PM

Filename SDF(408).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8



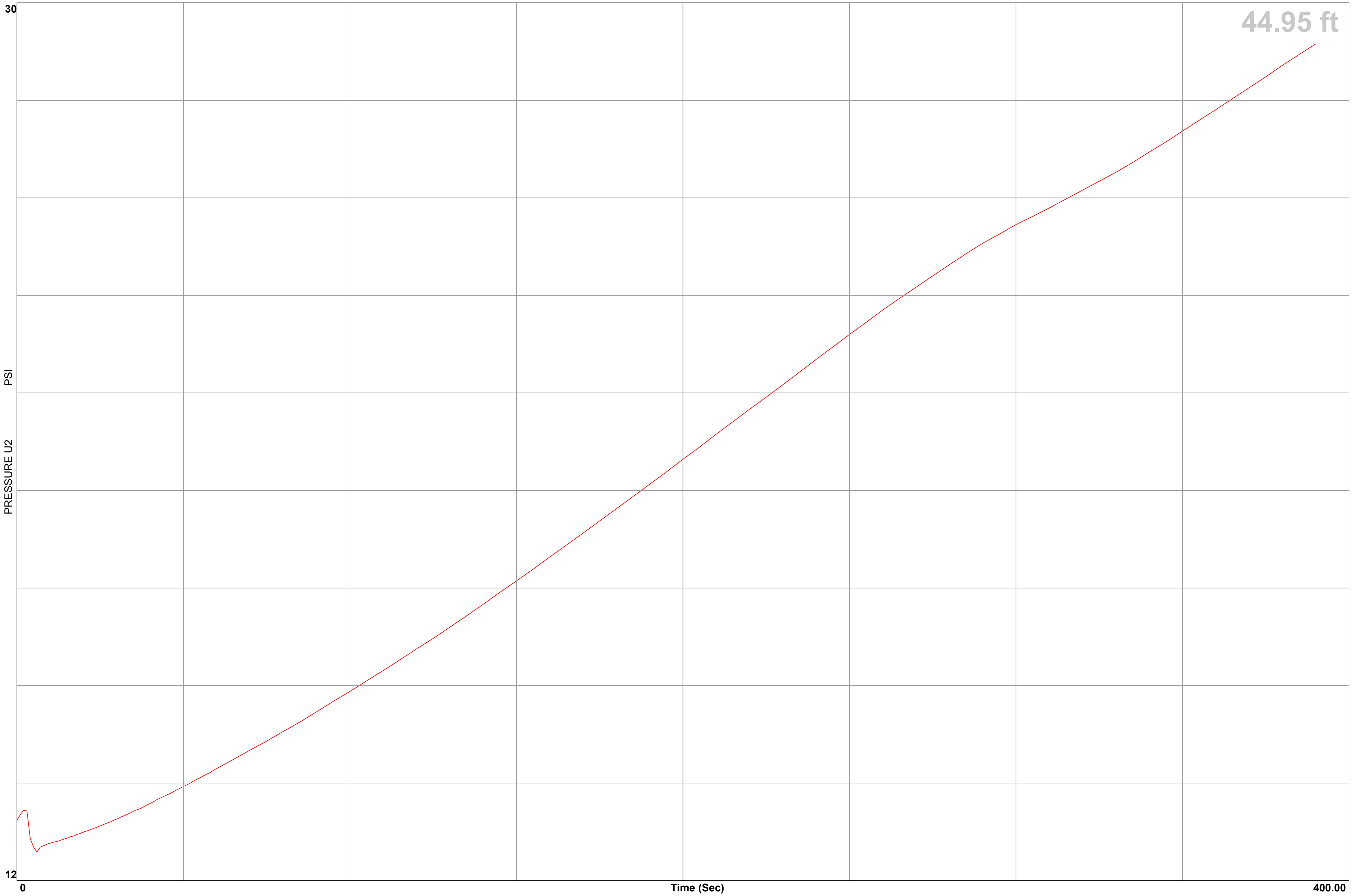
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location Enterprise-Whitesell Industrial BLDG
Job Number 916-3-1
Hole Number CPT-04
Equilized Pressure 29.0
Operator JM-AJ
Cone Number DDG1530
Date and Time 12/23/2020 12:14:12 PM
EST GW Depth During Test +22.1
GPS



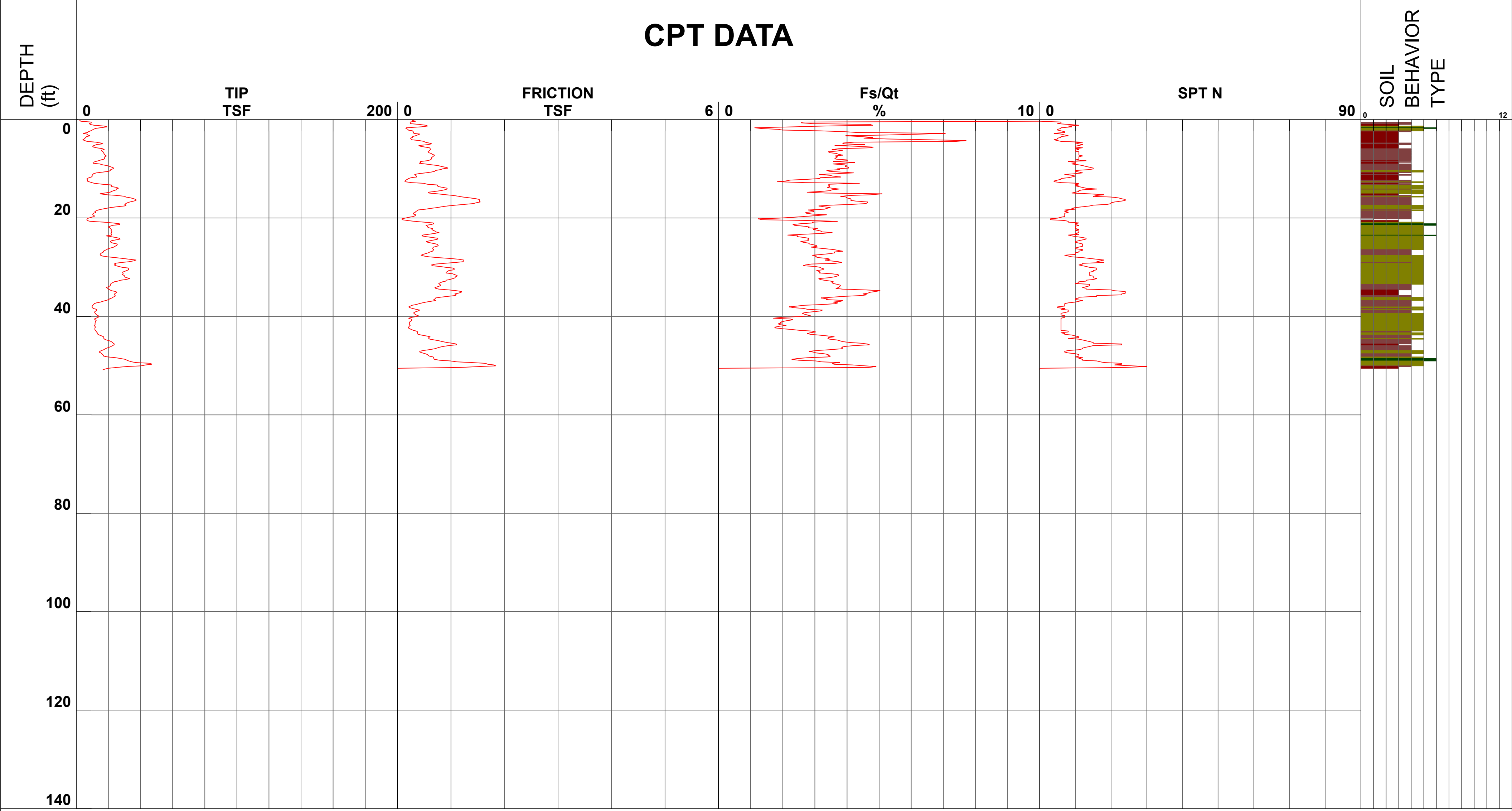
Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

Project Enterprise-Whitesell Industrial BLDG
Job Number 916-3-1
Hole Number CPT-05
EST GW Depth During Test 6.00 ft
Operator JM-AJ
Cone Number DDG1530
Date and Time 12/23/2020 4:13:05 PM
Filename SDF(411).cpt
GPS
Maximum Depth 50.85 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 34 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 21 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.

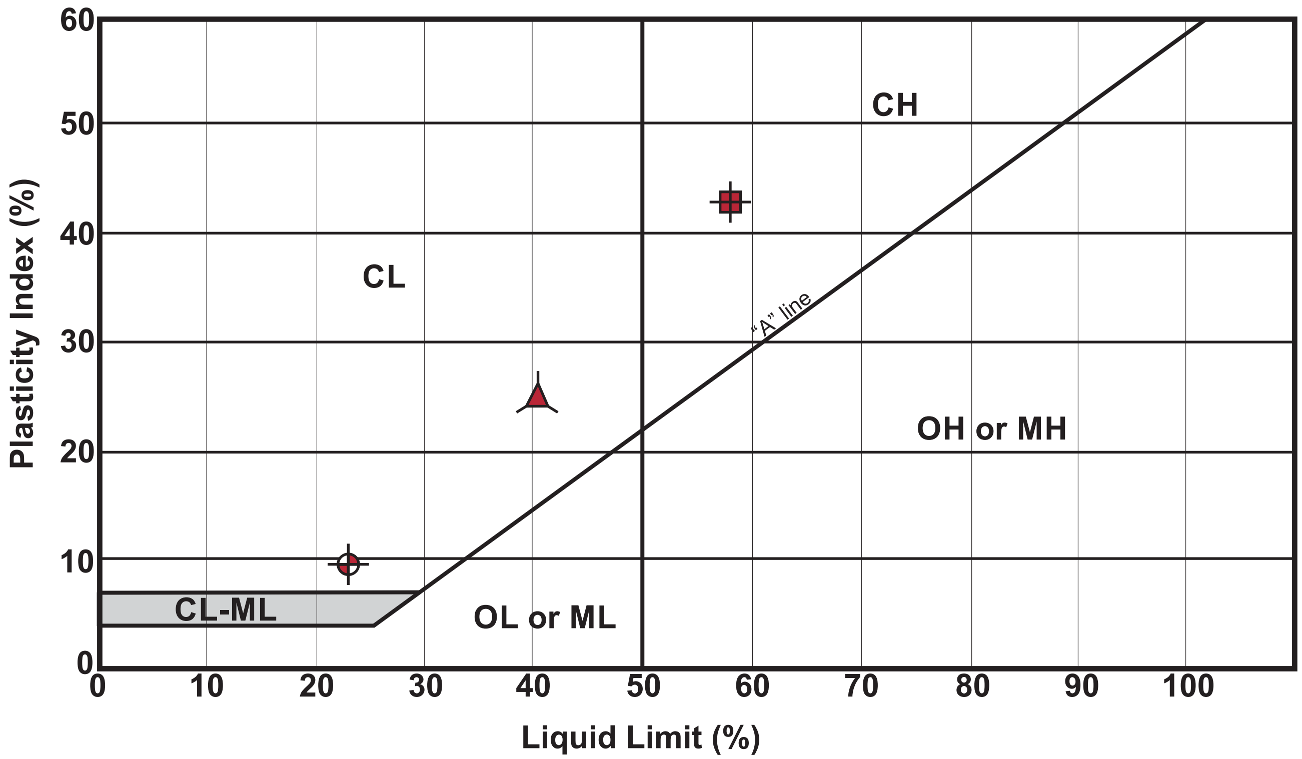
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on two samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

Preliminary Corrosion Screening: Three soluble sulfate determinations (ASTM D4327), three resistivity test (ASTM G57), three chloride determination (ASTM D4327), and three pH determination (ASTM G51) were performed on samples of the subsurface soil. Results of these tests are attached 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-1	19.0	18	23	14	9	36	Clayey Sand (SC) (CL fines)
■	EB-3	1.0	18	58	15	43	—	Fat Clay (CH)
▲	EB-5	5.0	22	41	16	25	—	Lean Clay (CL)

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

Enterprise/Whitesell Industrial Building
Hayward, CA

Project Number 916-3-1

Figure Number Figure B1

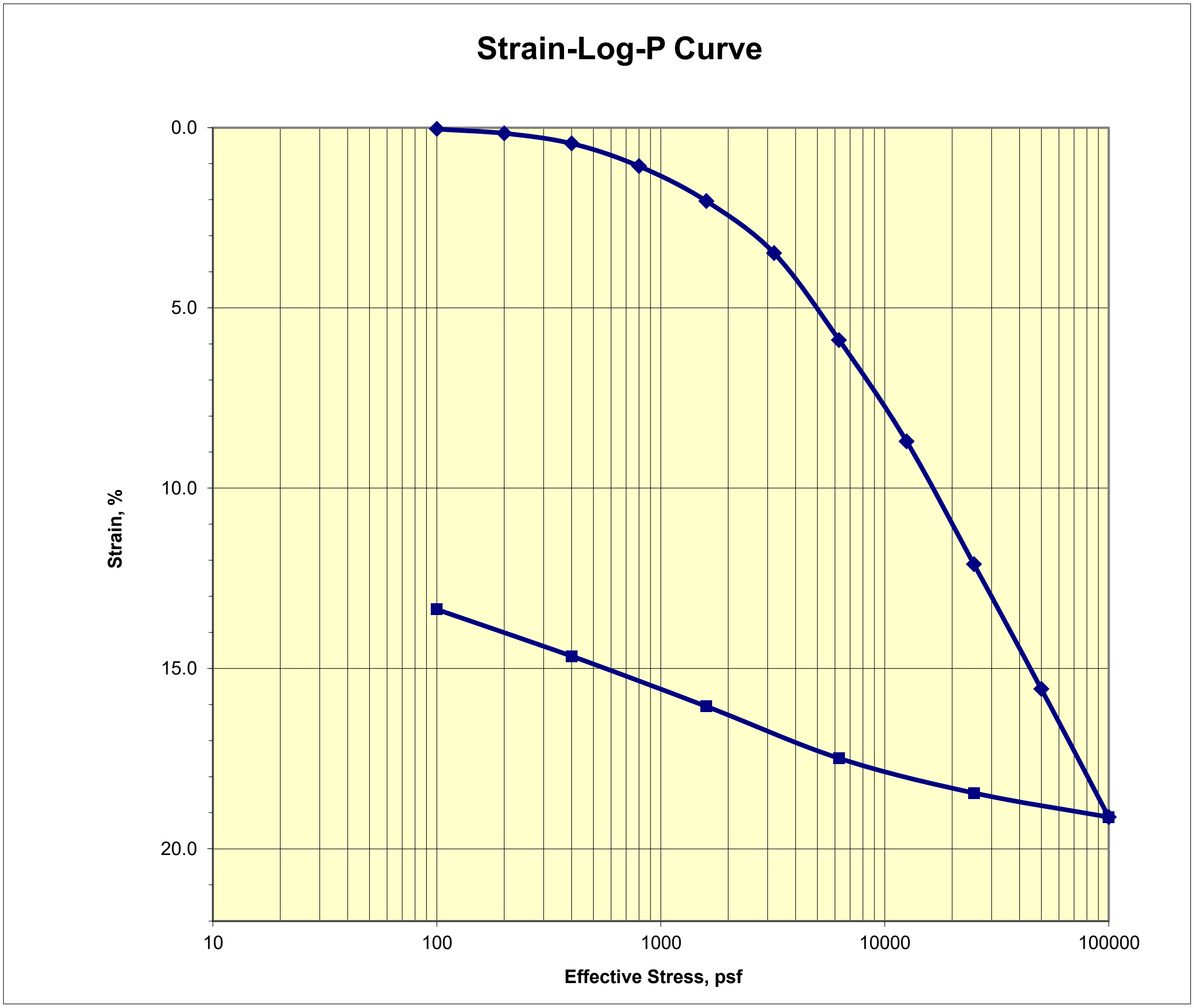
Date January 2021

Drawn By FLL

COOPER
TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	640-1440	**Boring:**	EB-3	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	7	**Reduced:**	PJ
Project:	916-3-1	**Depth, ft.:**	20(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Olive Brown Sandy CLAY			**Date:**	1/29/2021



Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	22.9	17.5
Dry Density, pcf:	100.4	115.8
Void Ratio:	0.710	0.482
% Saturation:	88.7	100.0

Remarks:

Corrosivity Tests Summary

CORNERSTONE EARTH GROUP

Job Number	916-3-1	Date Tested	1/19/2021
Job Name	Enterprise/Whitesell Industrial Building	Tested By	FLL
Location	3636 Enterprise Road, Hayward, CA		

Sample I.D.			Soil Visual Description	Moisture Content %	pH	Temp. at Testing C°	Resistivity (Ohm-cm) Corrected to 15.5 C°		Chloride Dry Wt. mg/kg	Sulfate Dry Wt. mg/kg
Boring	Sample No.	Depth, ft.					As Received	Saturated		
				ASTM D2216	ASTM G51		G57	ASTM G57	ASTM D4327	ASTM D4327
EB-2	1	1.0	Dark Gray Fat Clay (CH)	20.6	7.6	22.0	-	545	373	378
EB-4	4A	6.0	Brown Lean Clay (CL)	26.8	7.5	21.8	381	381	526	293
EB-5	2	3.5	Dark Gray Fat Clay (CH)	17.8	7.6	21.9	-	893	98	81