CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Front Street Santa Cruz Mixed-Use
LOCATION	Front Street and Soquel Avenue Santa Cruz, California
CLIENT	Swenson
PROJECT NUMBER	100-49-1
DATE	November 6, 2017

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Location	**Front Street and Soquel Avenue** **Santa Cruz, California**
Client	**Swenson**
Client Address	**740 Front Street, Suite 315** **Santa Cruz, CA**
Project Number	**100-49-1**
Date	**November 6, 2017**

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Swenson for the Front Street Santa Cruz Mixed-Use project in Santa Cruz, 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 set of plans titled “Front Street/Riverfront Corridor Development Standards and Design Guidelines,” prepared by City of Santa Cruz, undated.

1.1 PROJECT DESCRIPTION

We understand the project will consist of a mixed-use development consisting of one level of retail along Front Street with two levels of podium parking behind (adjacent to the levee). The podium parking will extend a partial level below-grade. Four levels of residential is planned above both the retail and podium parking. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned.

Cuts on the order of 8 to 10 feet may be required for the below-grade podium parking excavation and fills on the order of 10 to 15 feet may be required between the podium parking and levee to bring the east side of the site up to the existing levee grade. Cuts and fills on the remainder of the site are anticipated to be minor and on the order of 2 to 3 feet. structural loads are not currently known for the proposed structure; however, structural loads are expected to be typical of similar type structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated July 28, 2017 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 three borings drilled on August 13 and 14, 2017 with track-mounted, rotary-wash drilling equipment and four Cone Penetration Tests (CPTs) advanced on July 25, 2017. The borings were drilled to depths of 30 to 99½ feet; the CPTs were advanced to depths of approximately 41 to 82 feet, or where practical refusal was encountered for the CPT Borings. Seismic shear wave velocity measurements were collected from CPT-4. Three of the borings (Boring EB-1, EB-2 and EB-3) were advanced adjacent to CPT-1, CPT-4, and CPT-3, 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.

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, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

The greater 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 2015 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)
Monterey Bay-Tularcitos	6.5	10.5
Zayante-Vergeles	7.9	12.7
San Gregorio	9.9	16.0
San Andreas (1906)	11.2	18.0
Sargent	13.7	22.0
San Gregorio (Sur region)	14.1	22.7

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 at Front Street and Soquel Avenue in Santa Cruz, California. The site is currently occupied by two at-grade commercial buildings and at-grade parking lots. The site is bounded by a levee and the San Lorenzo River to the east, commercial development to the south, Front Street to the west, and Soquel Avenue to the north. The site is relatively level with elevations of approximately 16 to 21 feet above sea level (Google Earth, 2017).

Surface pavements generally consisted of 2 to 3½ inches of asphalt concrete over 2 to 4 inches of aggregate base. Based on visual observations, the existing pavements are in fair to poor conditions with areas of significant alligator cracking.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our exploratory borings generally encountered approximately 6½ to 9 feet of undocumented fills consisting of loose poorly graded sands, medium dense clayey sands with gravel, and medium stiff to hard sandy lean clay with varying amounts of sands. Beneath the undocumented fills, our borings generally encountered loose to medium dense sands with varying amounts of silts and clays to depths of approximately 62 to 86 feet in Borings EB-1 and EB-2 and 30 feet, or bottom of boring in EB-3. Below the loose to medium dense sands in Borings EB-1 and EB-2, our borings generally encountered dense to very dense sands with varying amounts of silts to the maximum depth explored of 99½ feet. Our CPTs were consistent with subsurface conditions encountered in our exploratory borings.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The result of the surficial PI test indicated a PI of 14, indicating a low expansion potential to wetting and drying cycles. The result of the PI test in a potentially liquefiable layer indicated a PI of 7.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from approximately optimum to about 10 percent over the estimated laboratory optimum moisture.

3.3 GROUND WATER

Ground water was estimated at depths of 12 to 16 feet below current grades based on pore pressure dissipation tests performed in CPT-1, CPT-2, and CPT-3. Ground water was not available for our exploratory borings due to the rotary wash method used. 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.

Groundwater data available on Geotracker in the project vicinity is recorded at depths of approximately 7 to 11 feet below existing grades. For our analysis, we assumed a high ground water level to be at 6 feet below current grades.

Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

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 2016 edition of the California Building Code. For our liquefaction analysis we used a PGA_M of 0.500g.

4.3 LIQUEFACTION POTENTIAL

The site is not mapped by California Geologic Survey but is mapped in an area of high to very high liquefaction potential by the County of Santa Cruz. 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 6 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.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The results of our CPT analyses (CPT-1 through CPT-4) are presented on Figures 4A through 4D 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 approximately 3½ to 7 inches 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 2⅓ to 4⅔ inches between independent foundation elements.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. We evaluated the potential for surface rupture base on the work of Youd and Garris (1995). Based on our analyses, the potential for liquefaction-induced ground rupture at the site is considered high due to the loose to medium dense sands encountered across the site starting at depths between 6½ and 9 feet below existing site grades. Please refer to the “Conclusions” section of this report for mitigation recommendations.

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.

The closest free face to the site is the San Lorenzo River location adjacent to the west of the site. The river channel is approximately 19 feet deep with an approximately 6 foot high levee separating the river from the property site. The potential for lateral spreading at the site is high based on Lateral Displacement Index (LDI) estimates. We analyzed the site for lateral spreading using analytical methods outlined in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) by calculating Lateral Displacement Index (LDI) values at each CPT location. The LDI is calculated by integrating maximum shear strains versus depth, representing a measure of the potential maximum displacement (Zhang et al., 2004).

Our analysis indicates a significant potential for lateral displacement at the site with LDI values ranging from 1.91 to 7.52 calculated for CPT-1 through CPT-4 in the area of the proposed structure, and potential lateral displacement ranging from approximately 1 to 15 feet. Mitigation options for lateral spreading are presented in subsequent sections of this report.

Provided mitigation measures are taken to improve the loose sandy soils encountered across the site starting at depths of 6½ to 9 feet below existing site grades, the potential for lateral spreading to affect the new construction 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 ground water depth were predominantly medium stiff to hard clays and medium dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low. In addition, the recommended ground improvement will mitigate any potential dry sand settlement.

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 around the northern tip of the Monterey Bay. The site is approximately ½ mile inland from the Pacific Ocean shoreline, and is approximately 16 to 21 feet above mean sea level. The site is mapped by California Geologic Survey as being within a tsunami inundation zone (CGS, 2009). Therefore, the potential for inundation due to tsunami or seiche is considered high.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone A99, described as “Special flood hazard areas subject to inundation by the 1% annual chance flood; areas to be protected from 1% annual chance flood event by a Federal flood protection system under construction; no Base Flood Elevations determined.” 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 and associated ground rupture
- Potential for lateral spreading
- Shallow ground water
- Differential movement at on-grade to on-structure transitions

5.1.1 Potential for Liquefaction-Induced Settlements and Associated Ground Rupture

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Our analysis indicates liquefaction-induced settlement on the order of 3½ to 7 inches, resulting in differential settlements ranging from 2⅓ to 4⅔ inches between independent foundation elements. In addition, the site has a moderate to high potential for ground rupture to occur that could result in ground deformation and settlements in addition to the estimated liquefaction settlements.

To mitigate the total liquefaction-induced settlements and potential for ground rupture, we recommend the proposed structure be supported on shallow foundations over ground improvement. Detailed recommendations are provided in the “Foundations” section of this report.

5.1.2 Potential for Lateral Spreading

As previously discussed, there is a potential for lateral displacement towards the adjacent San Lorenzo River. Potential for lateral spreading appears high for the proposed structure. Typical techniques to mitigate the potential for lateral spreading include ground improvement to construct a shear key or the installation of shear (pin) piers to effectively create a shear key. Additional recommendations are provided in subsequent sections of this report.

5.1.3 Shallow Ground Water

We anticipate that seasonal high ground water exists at depths ranging from approximately 7 to 11 feet below the existing ground surface. Our experience with similar sites in the vicinity indicates that shallow ground water 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 Differential Movement At On-grade to On-Structure Transitions

Some of the at-grade improvements and portions of the structure may transition from on-grade support to overlying the basements. Where the depth of soil cover overlying the basement roof in the at-grade areas is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

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

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

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

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

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. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

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

6.3 REMOVAL OF EXISTING FILLS

As previously discussed, approximately 6½ to 9 feet of undocumented fill was encountered in our exploratory borings. We anticipate the structure will be supported on ground improvement to mitigate the loose to medium dense sands. If ground improvement is installed beneath the building (at footings and under slab-on-grade), undocumented fills may be left in place. 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 beyond the building footprint 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 15 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. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

Excavations extending more than 5 feet below the building subgrade and excavations in pavement and flatwork areas should be slope in accordance with the OSHA soil classification.

6.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the "Temporary Cut and Fill Slopes" section above if space allows. Alternatively, temporary shoring may support the potentially planned cuts up to 10 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 2: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure$^{(2)}$	45 pcf
Restrained Wall – Uniform Earth Pressure$^{(2)}$	25H$^{(1)(3)}$ psf
Passive Pressure – Starting below the bottom of the adjacent excavation$^{(1)(3)}$	350 pcf up to 3,500 psf maximum uniform pressure

(1) H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter
(2) The cantilever and restrained pressures are for drained designs with dewatering. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures
(3) Bottom of adjacent excavation is bottom of mass excavation or bottom of footing excavation, whichever is deeper directly adjacent to the shoring element

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with rotary-wash drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.5.2 Construction Dewatering

Ground water levels are expected to be as high as 6 to 7 feet below current grades; therefore, depending on depths of potential excavations, temporary dewatering may be necessary during construction. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain ground water at least 5 feet below the bottom of the mass excavation, and at least 2 feet below localized excavations such as deepened footings, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Depending on the ground water quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

6.6 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.

Due to the sandy soils likely to be encountered at the subgrade elevation, we suggest recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

6.7 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 are up to about 10 percent over the estimated laboratory optimum in the upper 15 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.

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 10 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.

6.7.2 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.

6.7.3 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. Additional recommendations and laboratory testing should be performed if chemical treatment is desired.

6.7.4 Below-Grade Excavation Stabilization

As the planned basement excavation could potentially extend near the anticipated seasonal high ground water level, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi 500X, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

6.8 MATERIAL FOR FILL

6.8.1 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.

6.8.2 Re-Use of On-Site Site Improvements

We anticipate that moderate quantities of asphalt concrete (AC) grindings and aggregate base (AB) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections, including within below-grade parking garage slab-on-grade areas (provided crushed rock is not required due to the proximity to ground water). AC/AB grindings may not be reused within the habitable building areas including the below-grade garage level. Laboratory testing will be required to confirm the grindings meet project specifications. Due to the existing alligator cracking of some of the AC pavements, it is likely that the grinding operation will leave significant oversize chunks and won’t meet the Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

6.8.3 Potential Import Sources

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

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

6.9 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.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site 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 Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	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)
4 – Using light-weight compaction or walls should be braced

6.10 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.

6.11 SITE DRAINAGE

6.11.1 Surface 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.

6.12 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.

- Some of the near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to potentially 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 ground water is anticipated to be at a depth of less than 10 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.

- The site is not known, to our knowledge, to have pollutants with the potential for mobilization as a result of stormwater infiltration.

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

- Highly infiltrating native soils, such as sand and gravel, may not be protective of groundwater at a project site where infiltration devices are implemented.

- 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.

6.12.1 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.

6.12.1.1 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.

6.12.1.2 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.

6.12.1.3 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.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

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

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2016 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” 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 and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Shear eave velocity measurements performed at CPT-4 to a depth of 80 feet, or practical drilling refusal, resulted in an average shear wave velocity of 712 feet per second (or 217 meters per second). Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS web-based program *U.S. Seismic Design Maps*, located at https://earthquake.usgs.gov/hazards/designmaps/usdesign.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 4: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	D
Site Latitude	36.97273°
Site Longitude	-122.02412°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.500g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.000g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.600g

[1] For Site Class B, 5 percent damped.

Because the potential for liquefaction and the potential for affects to the structure appear high, based on Table 1613.5.2, Site Class Definitions, of the 2016 California Building Code (CBC), the site should be classified as Site Class F. However, as mentioned, we are recommending ground improvement be performed beneath the structure. If ground improvement is performed to minimize seismic settlements as discussed in the “Ground Improvement” Section 7.4 below, in our opinion, Site Classification D in Table 4 of this report, and the presented seismic coefficients, may be used.

7.3 SHALLOW FOUNDATIONS OVER GROUND IMPROVEMENT

In our opinion, ground improvement should be implemented to mitigate the potential for seismic and static settlement of the proposed buildings founded on spread footings. Due to the shallow ground water anticipated, if areas of the structure extend below-grade a mat foundation over ground improvement may be necessary. In our opinion, ground improvement will mitigate the settlements to tolerable levels and, as long as the recommendations in the “Earthwork” section and subsequent sections are followed, the proposed structure may be supported on shallow foundations.

7.3.1 Spread Footings

Removal of disturbed soils due to ground improvement should be performed during site grading prior to excavating footings. Spread footings should bear on engineered fill overlying ground improvement, be at least 24 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Bearing pressures will be dependent on the final designed ground improvement technique and spacing; however, substantial improvement in bearing capacity would be expected. On a preliminary basis, an average allowable bearing pressure of 4,000 to 6,000 psf for combined dead plus live loads may be used in preliminary designs. We recommend the above allowable pressure be confirmed with the design-build ground improvement contractor.

Ground improvement and the replacement of disturbed near-surface soils as engineered fill would be designed to reduce total settlement due to static and seismic conditions to meet the structural requirement.

7.3.2 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 350 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.3 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.

7.3.4 Reinforced Concrete Mat Foundations

Based on our experience, we estimate a mat foundation underlain by ground improvement may be designed for maximum average allowable contact pressures of 3,000 to 4,000 pounds per square foot (psf) for dead plus live loads. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

The above bearing pressures estimates should be evaluated further once a design-build ground improvement contractor has been chosen. Recommendations for ground improvement are provided in the following sections.

7.3.5 Mat Foundation Settlement

Ground improvement should be designed to reduce total settlement due to static and seismic conditions to tolerable levels. As discussed in the “Ground Improvement” section below, the ground improvement design should be such that the total foundation settlement (static and seismic) are reduced to 1½ inches or less with no more than 1 inch for either the static or seismic component.

7.3.6 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. As the mat foundation will be underlain by ground improvement, the modulus of subgrade reaction will be affected by the ground improvement method and the bearing pressures across the mat. Once ground improvement design has been confirmed and initial bearing pressures determined, please forward the contact pressures plan for the mat (to scale and in color).

7.3.7 Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 350 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.8 Moisture Protection Considerations for Mat Foundations

For moisture protection recommendations, please refer to Section 8.3.

7.3.9 Hydrostatic Uplift and Waterproofing

Where portions of the structures extend below the design ground water level, including bottoms of slabs-on-grade and mat foundations, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design ground water should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design ground water level, a drainage system may be added as discussed in the “Retaining Wall” section.

In addition, the portions of the structures extending below design ground water should be waterproofed to limit moisture infiltration, including mat foundation/thickened slab areas, all construction joints, and any retaining walls. We recommend that a waterproof specialist design the waterproofing system.

7.4 GROUND IMPROVEMENT

7.4.1 Ground Improvement Requirements

Ground improvement should consist of densification techniques to improve the ground's resistance to liquefaction, reduce static settlement, and improve bearing capacity and seismic performance. Densification techniques could potentially consist of vibro replacement (i.e. stone columns), granular compaction piles (i.e. rammed aggregate), grouted displacement columns (i.e. CLSM), or similar densification techniques. The intent of the ground improvement design would be to increase the density of the potentially liquefiable sands by laterally displacing and/or densifying the existing in-place soils. The degree to which the density is increased will depend on the improvement method and spacing. Ground improvement can also be used to reduce static settlements and increase bearing capacity.

Vibro replacement and granular compaction piles are similar in that a probe is vibrated into the ground to the design depth and a compacted open-graded gravel column is constructed from the bottom up. The surrounding soils are densified by the displacement of the soil as well as the vibrations from consolidating and expanding the gravel column laterally. One of the disadvantages of these densification pile types are the noise and vibration (and sometimes dust) produced during construction. The vibrations may cause noise and vibrations that can be heard or felt off-site. Pre-drilling through surficial materials may reduce noise and vibration, and should be anticipated for improvement areas adjacent to the site that may be sensitive to vibrations.

CLSM columns are formed in displaced soil cavities and displace liquefiable and compressible soil with cemented Controlled Low Strength Material. CLSM column ground improvement can mitigate liquefaction and settlement of heavy foundations and slabs. CLSM columns are ideal for sensitive project sites such as those near critical structures that require low noise and no vibration construction methods, unreinforced masonry walls, occupied offices, sensitive soil (e.g. Bay Mud), and hazardous/contaminated soil sites where deep ground improvement is required.

The CLSM columns are separated from the bottom of the footing using a minimum 6-inch layer of crushed rock or other material "cushion". No connectivity of the CLSM columns and overlying structural element is allowed. In some cases, a Ground Anchor may be used in a higher strength column to resist uplift forces. Lateral resistance is provided by footing, mat, or slab bottom friction at the concrete to cushion layer interface or passive resistance of the side walls. The target strengths of the CLSM are usually between 500 to 1,000 psi at 28 days, depending on load demands. The CLSM strength is tested using standard sampling and loading methods.

Based on the chosen ground improvement technique, the upper 1 to 2 feet or more of the working pad will likely need to be re-compacted after ground improvement installation, due to surface disturbance and potential ground heave. For this reason, we do not recommend preparation of the final pad, placement of non-expansive fill, or the construction of utilities prior to ground improvement.

Contractors to perform recommended ground improvement should have adequate experience for the proposed methods to address the requirements herein. All construction quality control and quality assurance records should be supplied to the design team for review on completion of the ground improvement. Adequate quality control readings must be available at the time of installation so that real time oversight can be provided. The instrumentation provided will depend on the ground improvement method chosen. Once a method is chosen, the geotechnical engineer should modify the project design guideline specification for the appropriate method.

7.4.2 Ground Improvement Design Guidelines

The ground improvement columns will extend from building subgrade to near the bottom of the potentially liquefiable layers as necessary to meet the design criteria, estimated to be as deep as 30 to 40 feet below existing grades. The ground improvement design should reduce the total (static plus seismic) settlement to 1½ inches or less, with no more than 1-inch of static nor 1-inch of seismic settlement allowed as a component of the total settlement. This total settlement is preliminary and this criteria should be confirmed collaboratively with the structural engineer and owner.

We anticipate a ground improvement element spacing of about 4 to 6 feet on center beneath spread footing foundations and 5 to 8 feet on center within slab-on-grade areas, including mats, to meet the performance criteria given above. Due to the variability and uncertainty of ground conditions, we recommend that ground improvement element spacing not exceed 6 feet in foundation areas, and 8 feet in slab-on-grade improvement areas. We anticipate a tighter spacing will likely be required for the CLSM column methodology, as vibratory consolidation of sandy soils is typically more effective laterally at densification than non-vibratory displacement column construction.

Research indicates that pore pressure migration can affect even improved areas, and it is common to continue densification improvement to a distance outside of the building area. For that reason, ground improvement should be designed to provide adequate confinement around all foundations at the perimeter of the structure (at least one row of columns beyond the foundation limits) in addition to the foundation elements.

We recommend that the ground improvement design include, but not be limited to: 1) drawings showing the ground improvement layout, spacing and diameter, 2) the foundation layout plan, 3) proposed ground improvement length, 4) top and bottom elevations. We should be retained to review the ground improvement contractor’s plan and settlement estimates prior to construction, and to review and confirm that the contractor’s ground improvement design will satisfactorily meet the design criteria based on the performance testing. Following the completion of the Ground Improvement Performance Testing indicated below, a final ground improvement design report and calculation package, including support for the ground improvement design and indicating that the design criteria will be met, should be submitted to the design team for review and approval.

Ground improvement would generally be constructed as follows: 1) clear the site of existing demolition debris, 2) mass grading to the building pad subgrade elevation, 3) install performance test arrays to confirm the design spacing achieves the densification requirements, verified by CPT testing and additional liquefaction analyses, 4) install the ground improvement on the approved layout, and 5) re-compact top of building pad, as required, prior to construction of remainder of pad and the foundations.

7.4.3 Ground Improvement Performance Testing

On a preliminary basis, foundation and slab areas must meet the above total settlement criteria, which will include all settlement estimated from static loads and seismic shaking. Analysis of settlement for static loading should include compression within the treatment area due to structural loads, and long-term consolidation estimated for below the zone of treatment. Analysis of settlement for seismic loading should include settlement due to liquefaction strain, as well as any dry sand settlement. Ground improvement must also provide adequate support for the design bearing capacity.

Performance testing typically consists of a pre-construction test section to confirm design spacing with post-installation CPT testing to confirm that suitable ground improvement has occurred to meet the design criteria. If the design criteria have not been met, then additional testing may be required. Verification testing involves carrying out pre- and post-array penetration testing of the soil equidistant between treatment points for the analysis of liquefaction, and comparison with measurements before treatment. We recommend that liquefaction analysis methods used include the methods proposed by Idriss and Boulanger (2014). Because of detrimental effects of pore pressure on the results of testing, we recommend that testing of ground improvement test arrays occur no sooner than two weeks after their installation. This should be incorporated into project planning, as well as the possibility that additional arrays and testing may be required if proposed spacing is inadequate.

Verification testing also includes the performance of a modulus test at each array location. To validate the parameters selected for a specific project, a modulus load test is performed on a test pier typically constructed in locations chosen in coordination with the geotechnical engineer. Modulus tests are conducted to a pressure equal to at least 150% of the maximum design top of pier stress to assure a reasonable level of safety which supports long term settlement control and demonstrates that the ground improvement element has adequate strength. Performing modulus testing beyond the limit state top of pier stress meets the intent of the building code with respect to shallow foundation support. Modulus testing should be performed in general accordance with ASTM D1143.

For the proposed mixed-use structure at the site, we recommend that at least two test arrays including liquefaction and modulus testing be performed.

We should observe and monitor installation of the test arrays and production ground improvement on a full-time basis and review the post-test array settlement analyses provided by the contractor.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE OVER GROUND IMPROVEMENT

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. 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 PODIUM GARAGE SLABS-ON-GRADE

Garage slabs-on-grade should be at least 5 inches thick and if constructed with minimal reinforcement intended for shrinkage control only, should have a minimum compressive strength of 3,000 psi. If the slab will have heavier reinforcing because the slab will also serve as a structural diaphragm, the compressive strength may be reduced to 2,500 psi at the structural engineer's discretion. If there will be areas within the garage that are moisture sensitive, such as equipment and elevator rooms, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. 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.

8.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

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

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

- The 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.

8.4 EXTERIOR FLATWORK

8.4.1 Pedestrian Concrete Flatwork

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

SECTION 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 on a design R-value of 5. The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 5: Asphalt Concrete Pavement Recommendations, Design R-value = 5

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

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

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

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). Recommendations for garage slabs-on-grade were provided in the “Concrete Slabs and Pedestrian Pavements” section above. 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.

Table 6: PCC Pavement Recommendations, Design R-value = 5

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

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.

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.

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 7: 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

Basement walls should be designed as restrained walls. 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 Basement Walls

The 2016 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). If the walls are greater than 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Basement walls are not free to deflect, and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the basement wall will be restrained (use 45 pcf + 8H psf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $1.7H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 24 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained "at-rest" pressure) from our report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

10.2.2 Site Walls

The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

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.

10.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and 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. In addition, where drainage panels will connect from a horizontal application for at-grade areas to vertical basement wall drainage panels, the drainage path must be maintained. We are not aware of manufactured corner protection suitable for this situation; therefore, we recommend that a section of crushed rock be placed at the transitions. The crushed rock should be at least 3 inches thick, extend at least 12 inches horizontally over the top of the basement roof and 12 inches down from the top of the basement wall, and have a layer of filter fabric covering the crushed rock.

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

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 Swenson specifically to support the design of the Front Street Santa Cruz Mixed-Use project in Santa Cruz, 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.

Swenson may have provided Cornerstone with plans, reports and other documents prepared by others. Swenson 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.

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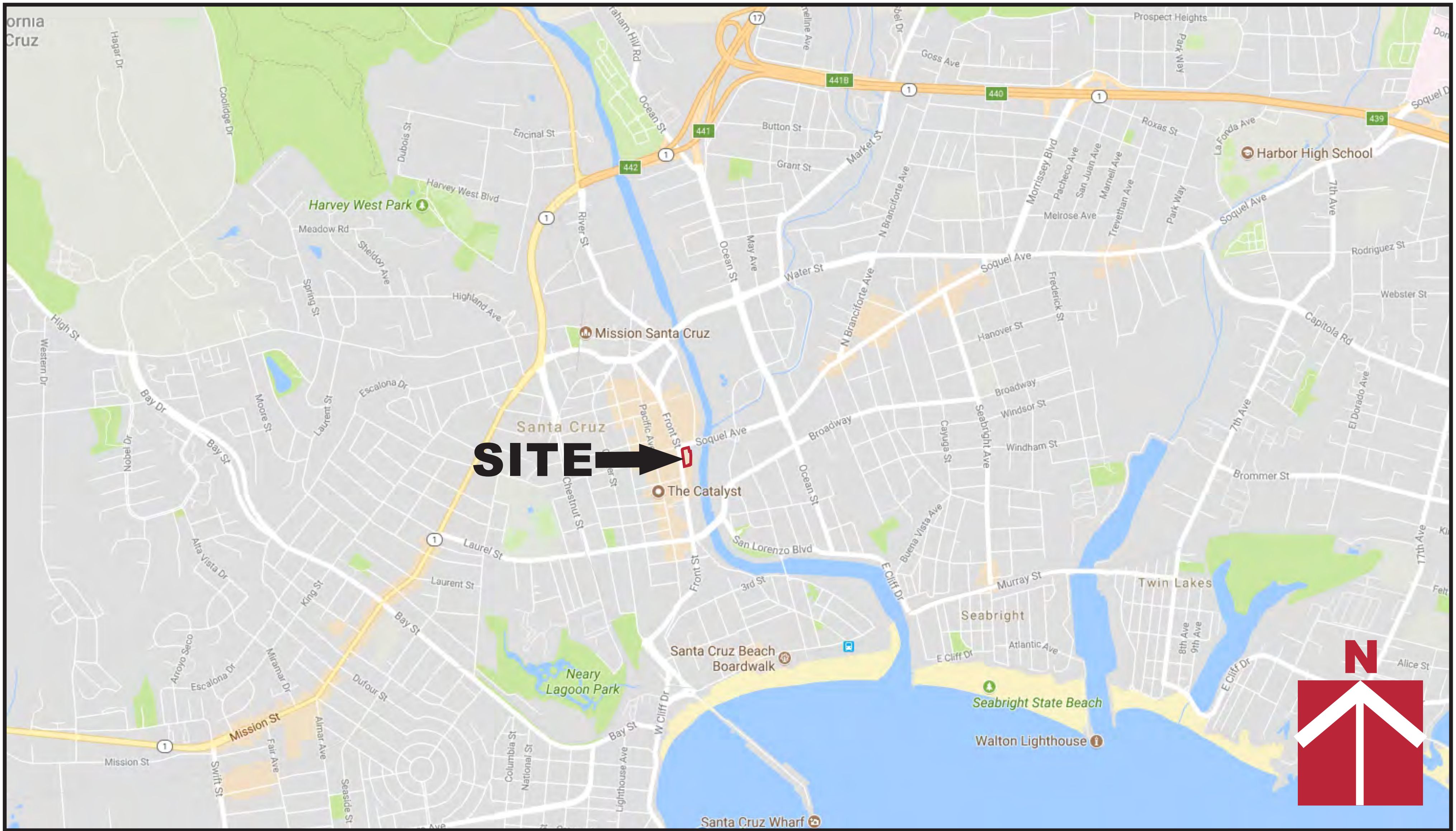

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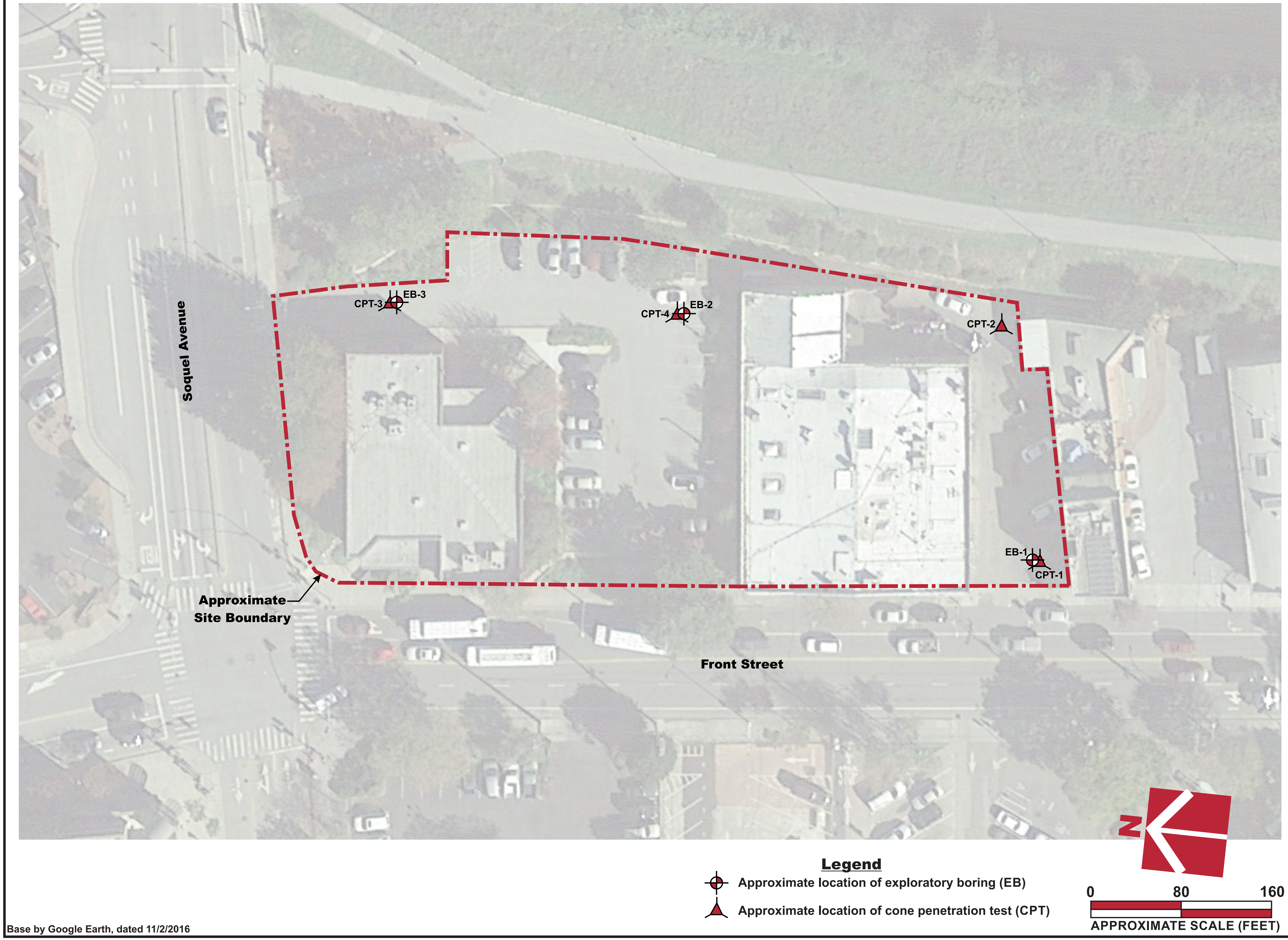
Vicinity Map

Front Street Mixed Use
Santa Cruz, CA

Project Number	100-49-1
Figure Number	Figure 1
Date	October 2017
Drawn By	RRN



Base by Google Earth, dated 11/2/2016

Project Number
100-49-1

Figure Number
Figure 2

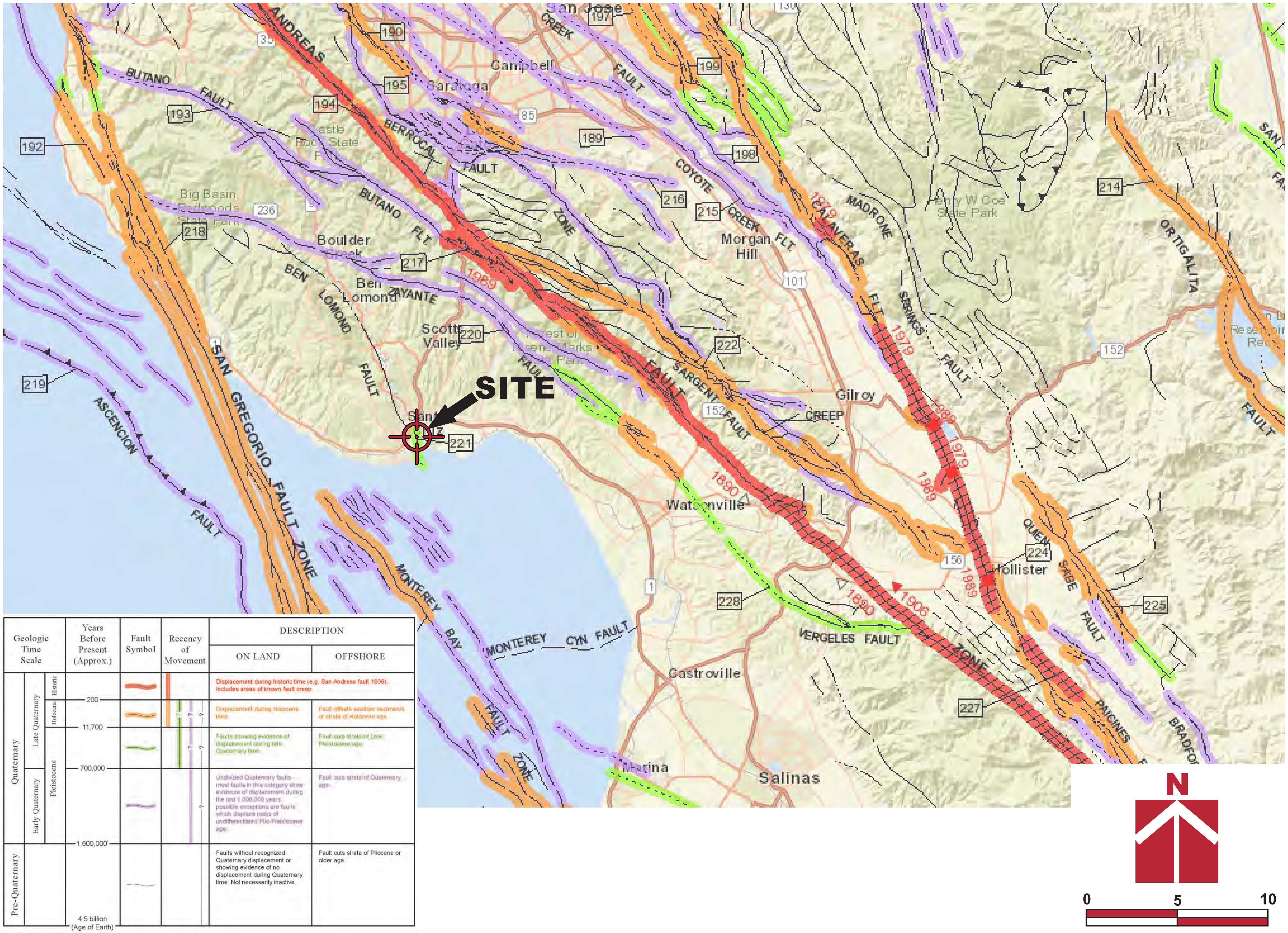
Drawn By
RRN

Date
October 2017

Site Plan

Front Street Mixed Use
Santa Cruz, 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 Plio-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)

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Regional Fault Map

Front Street Mixed Use
Santa Cruz, CA

Project Number 100-49-1

Figure Number Figure 3

Date October 2017

Drawn By RRN

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FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title **Front Street Mixed-Use**

Project No. **100-49-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.5** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **12.8**

Design Water Depth (feet) **6**

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

Ave. Unit Weight Below GW (pcf) **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.06 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

5.42 **(Inches)**

TOTAL SEISMIC SETTLEMENT 5.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 2.70 **L/H** 14.5

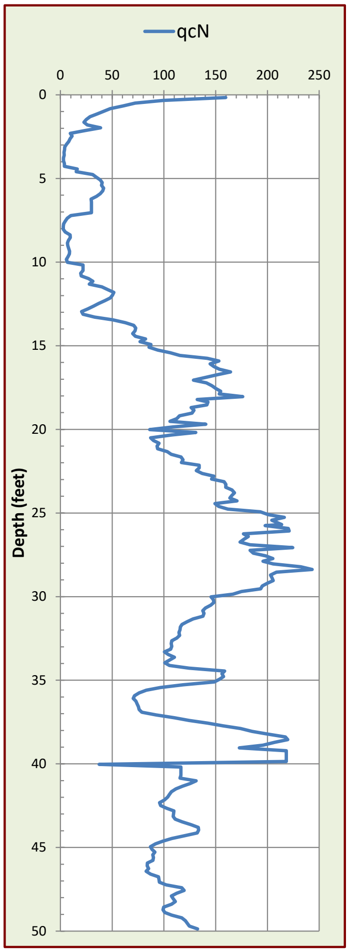
LDI[1]$_{\text{Corrected for Distance}}$ 1.91 (4 < L/H < 40)

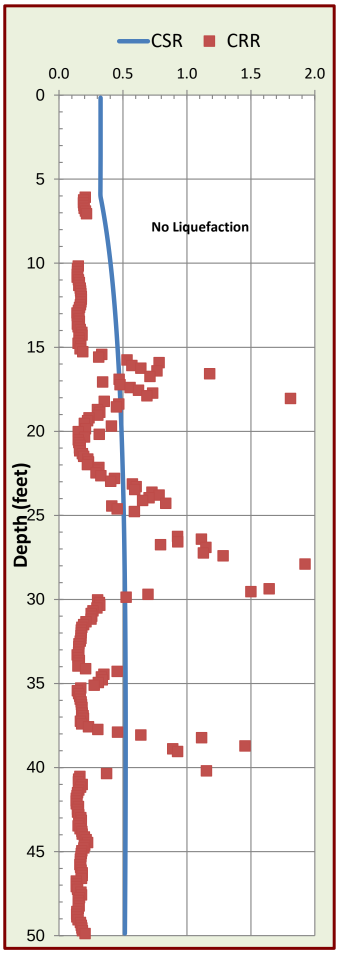
EXPECTED RANGE OF DISPLACEMENT

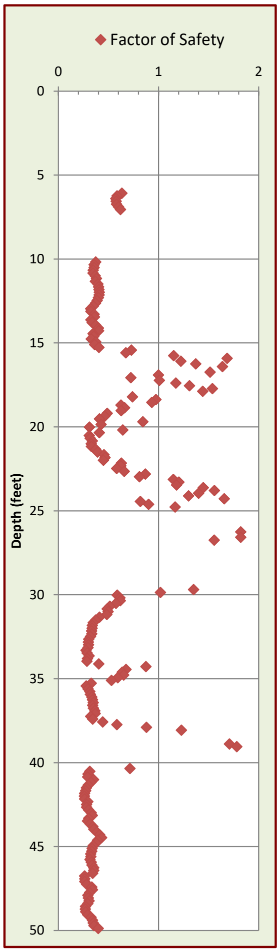
1.0 **to** **3.8** **feet**

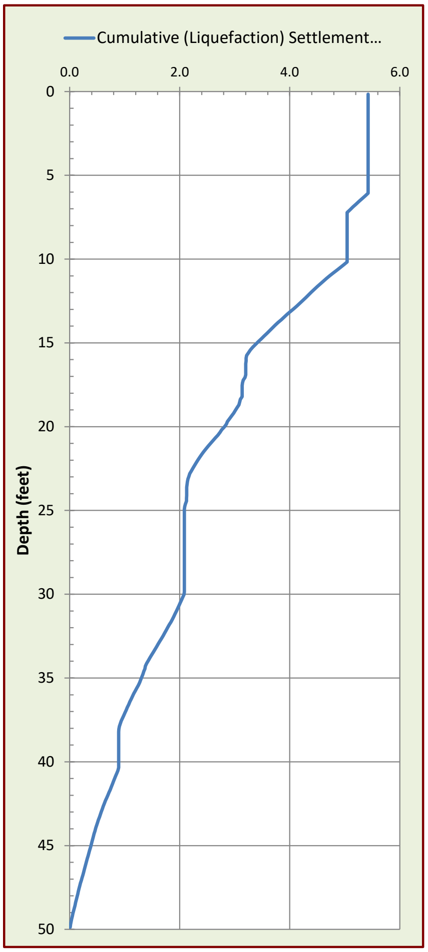
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title **Front Street Mixed-Use**

Project No. **100-49-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.5** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **12.1**

Design Water Depth (feet) **6**

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

Ave. Unit Weight Below GW (pcf) **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

3.48 **(Inches)**

TOTAL SEISMIC SETTLEMENT 3.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 1.25 **L/H** 5.0

LDI[1]$_{\text{Corrected for Distance}}$ 2.07 (4 < L/H < 40)

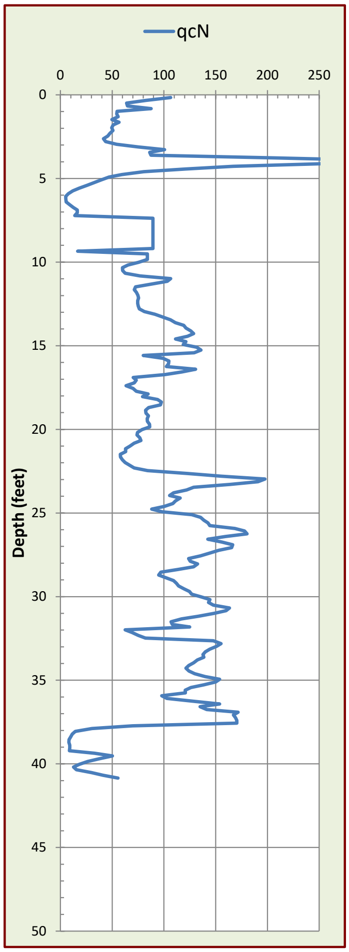
EXPECTED RANGE OF DISPLACEMENT

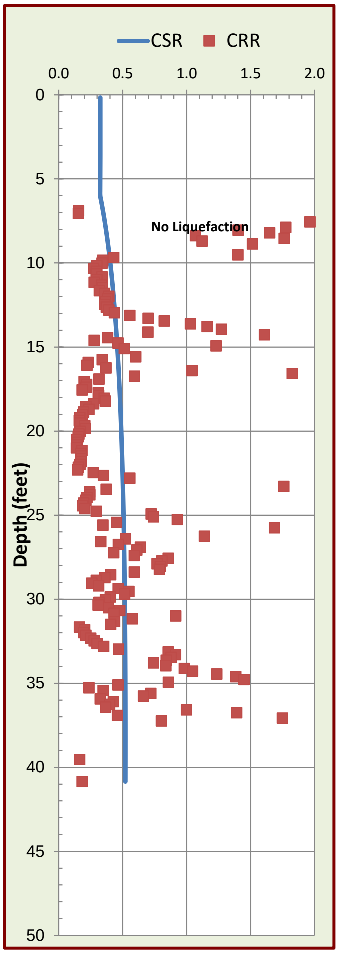
1.0 **to** **4.1** **feet**

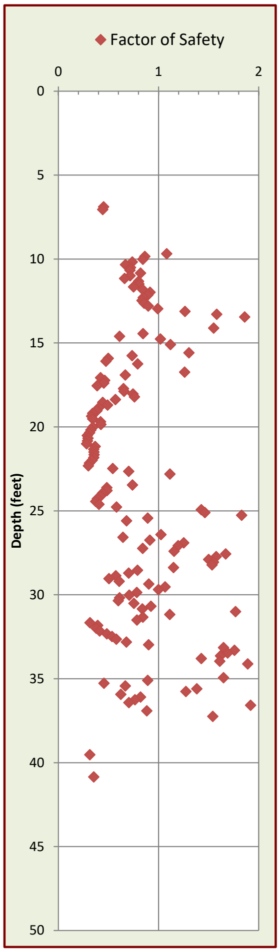
---

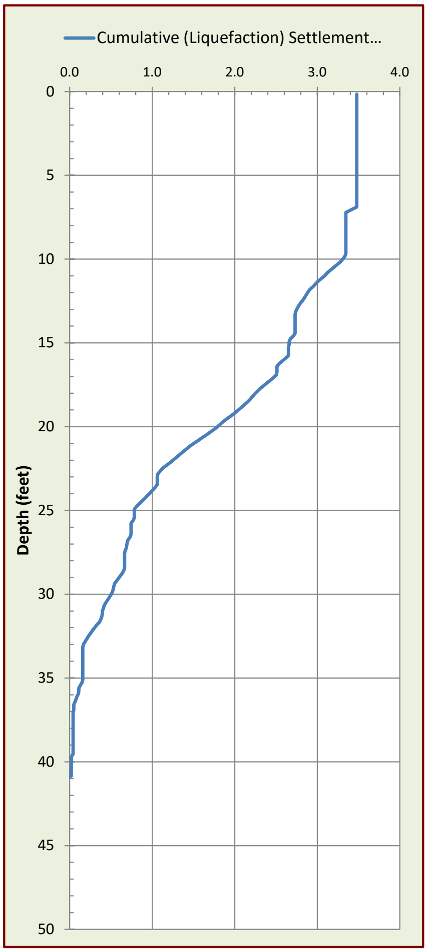
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title **Front Street Mixed-Use**

Project No. **100-49-1**

Project Manager **MFR**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.5** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **16.2**

Design Water Depth (feet) **6**

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

Ave. Unit Weight Below GW (pcf) **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

6.71 **(Inches)**

TOTAL SEISMIC SETTLEMENT 6.7 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 6.04 **L/H** 7.1

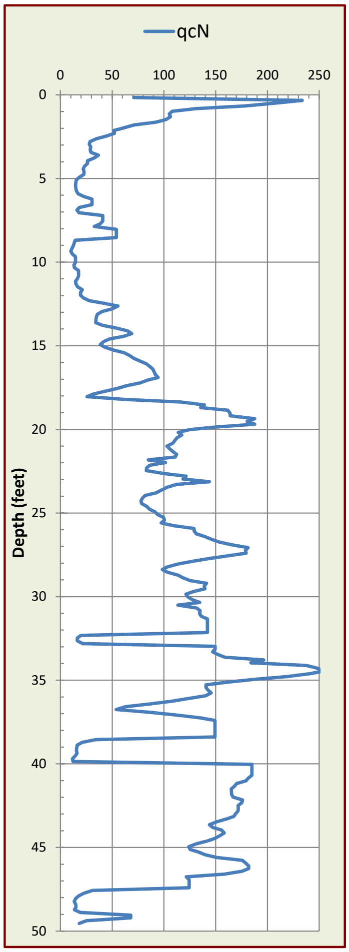
LDI[1]**Corrected for Distance** 7.52 (4 < L/H < 40)

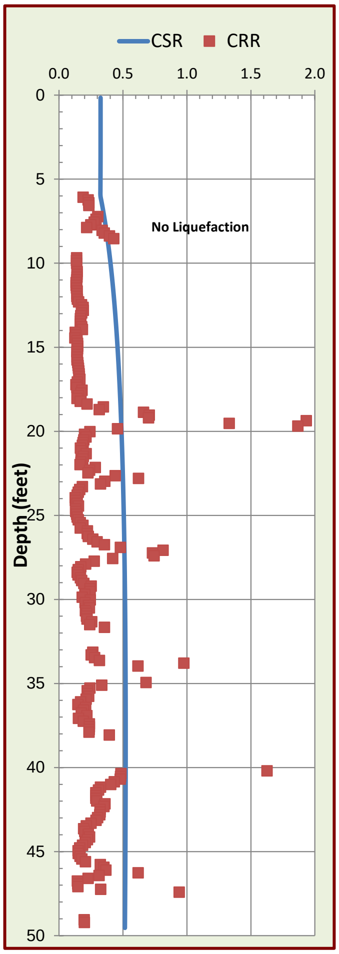
EXPECTED RANGE OF DISPLACEMENT

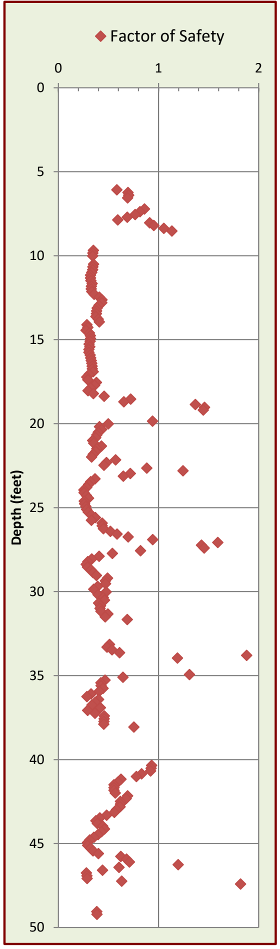
3.8 **to** **15.0** **feet**

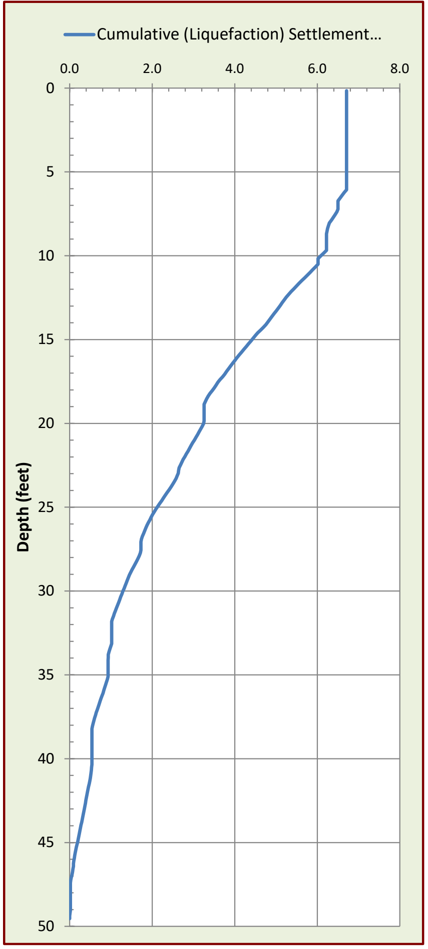
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	**Front Street Mixed-Use**
Project No.	**100-49-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.5**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**16.2**
Design Water Depth (feet)	**6**
Ave. Unit Weight Above GW (pcf)	**125**
Ave. Unit Weight Below GW (pcf)	**120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

5.51 **(Inches)**

TOTAL SEISMIC SETTLEMENT 5.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 3.55 **L/H** 5.8

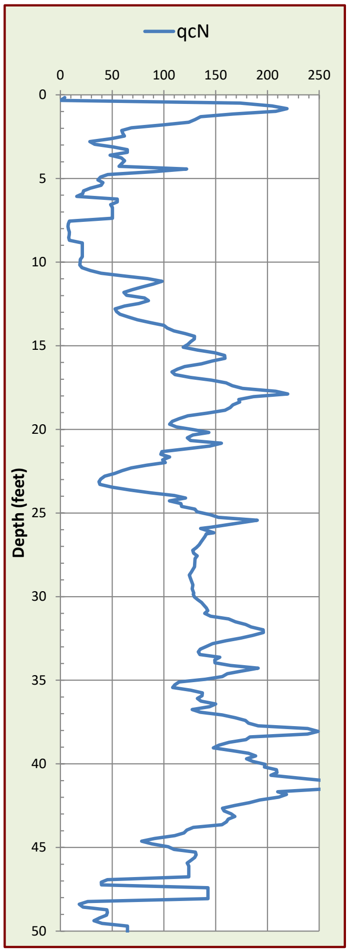
LDI[1]**Corrected for Distance** 5.25 (4 < L/H < 40)

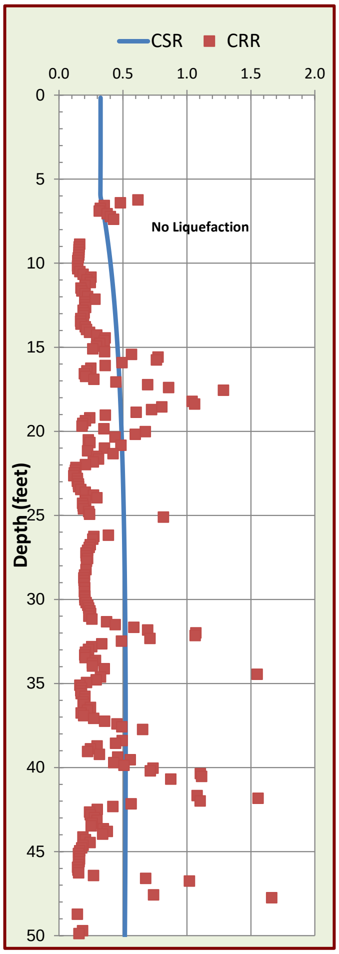
EXPECTED RANGE OF DISPLACEMENT

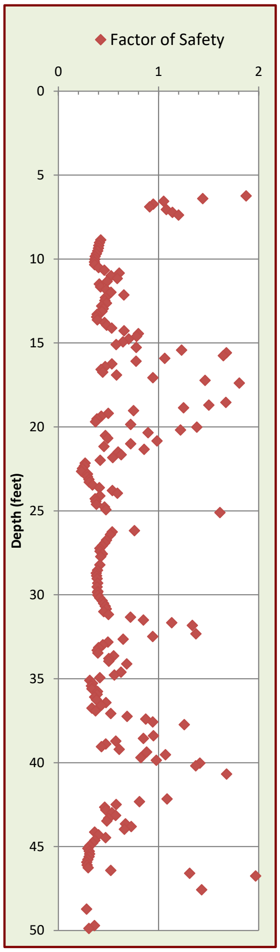
2.6 **to** **10.5** **feet**

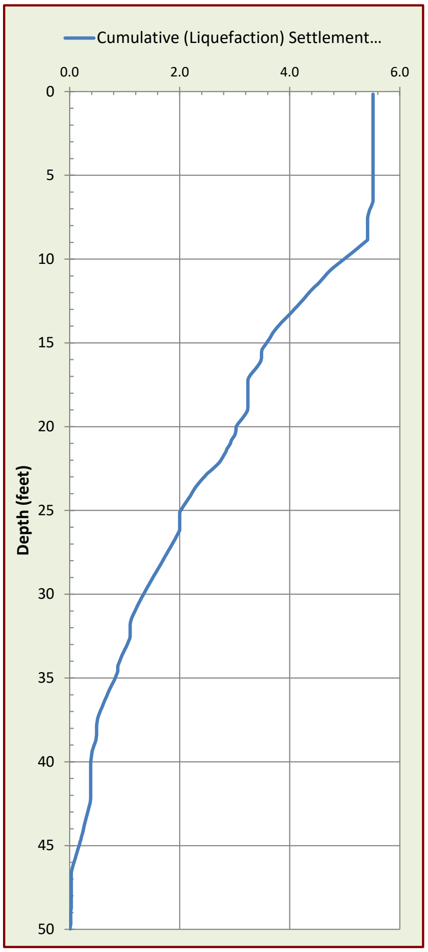
[1] Not Valid for L/H Values < 4 and > 40.

[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 track-mounted, rotary-wash drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Three 6-inch-diameter exploratory borings were drilled on September 13 and 14, 2017 to depths of 30 to 99½ feet. Four CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on August 25, 2017, to depths ranging from approximately 41 to 82 feet, or practical drilling refusal. 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. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

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

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

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

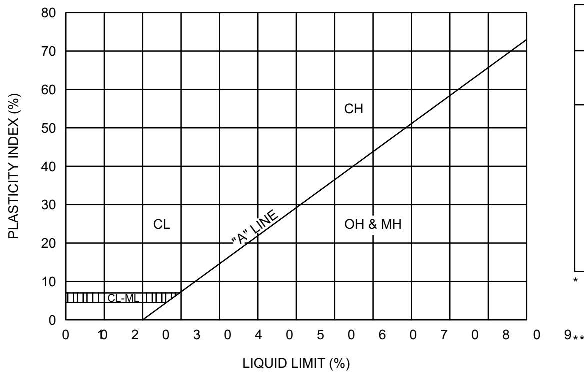
SAMPLER TYPES

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

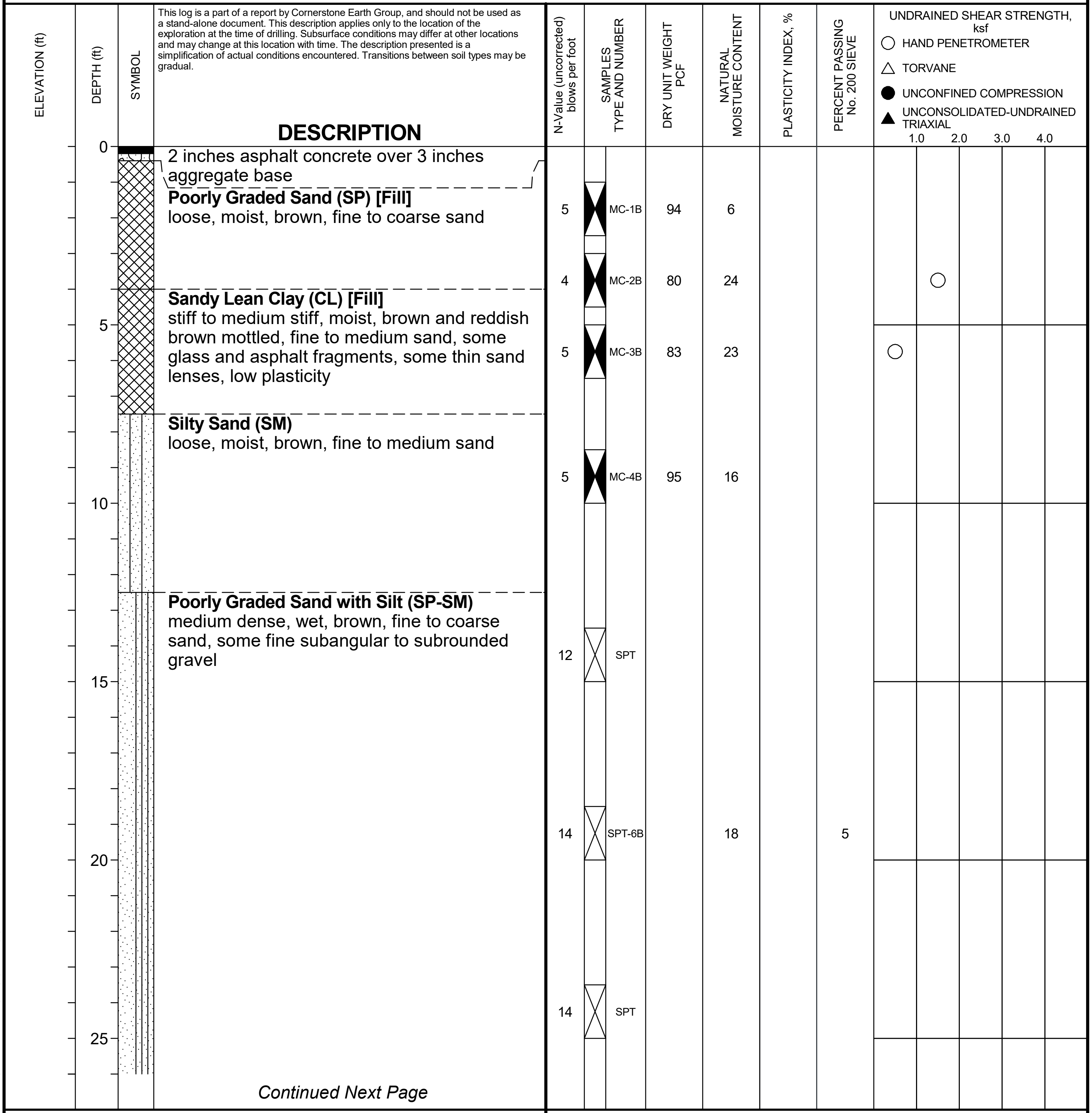
PAGE 1 OF 3

CORNERSTONE EARTH GROUP

PROJECT NAME Front Street Mixed Use
PROJECT NUMBER 100-49-1
PROJECT LOCATION Santa Cruz, CA
GROUND ELEVATION ____ **BORING DEPTH** 70 ft.
LATITUDE ____ **LONGITUDE** ____
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** Not Encountered
▼ **AT END OF DRILLING** Not Encountered

DATE STARTED 9/14/17 **DATE COMPLETED** 9/14/17
DRILLING CONTRACTOR Britton Exlporation Services, Inc.
DRILLING METHOD CME 55 Track Rig, 4 inch rotary wash
LOGGED BY SCO
NOTES ____

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	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
2 inches asphalt concrete over 3 inches aggregate base						
Poorly Graded Sand (SP) [Fill] loose, moist, brown, fine to coarse sand	5	MC-1B	94	6		
	4	MC-2B	80	24		
Sandy Lean Clay (CL) [Fill] stiff to medium stiff, moist, brown and reddish brown mottled, fine to medium sand, some glass and asphalt fragments, some thin sand lenses, low plasticity	5	MC-3B	83	23		
Silty Sand (SM) loose, moist, brown, fine to medium sand	5	MC-4B	95	16		
Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel	12	SPT				
	14	SPT-6B		18		5
	14	SPT				

Continued Next Page

BORING NUMBER EB-1

PROJECT NAME Front Street Mixed Use
PROJECT NUMBER 100-49-1
PROJECT LOCATION Santa Cruz, 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)	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
		Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel							
	30		17	SPT-8B		16			
	35		15	SPT					
		Silty Sand (SM) medium dense, moist, grayish brown, fine to coarse sand, some fine subangular to subrounded gravel							
	40		11	SPT-10B		29			
		Clayey Sand (SC) medium dense, moist, grayish brown, fine to medium sand							
		Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel							
	45		16	SPT					
	50		17	SPT-12		19		12	
	55								

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CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 3 OF 3

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, 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
		▨	**Silty, Clayey Sand (SC-SM)** loose, moist, grayish brown, fine to medium sand, some fine subangular to subrounded gravel							
				7	☒ SPT-13		33			
	60									
			Silty Sand (SM) very dense, moist, grayish brown, fine to coarse sand, some fine subangular to subrounded gravel							
	65									
				54	☒ SPT-14		18			
	70		Bottom of Boring at 70.0 feet.							
	75									
	80									
	85									

BORING NUMBER EB-2

PAGE 1 OF 4

CORNERSTONE
EARTH GROUP

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, CA

DATE STARTED 9/13/17 **DATE COMPLETED** 9/13/17

GROUND ELEVATION **BORING DEPTH** 99.5 ft.

DRILLING CONTRACTOR Britton Exlporation Services, Inc.

LATITUDE **LONGITUDE**

DRILLING METHOD CME 55 Track Rig, 4 inch rotary wash

GROUND WATER LEVELS:

LOGGED BY SCO

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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
	0		3½ inches asphalt concrete over 2 inches aggregate base							
			Sandy Lean Clay (CL) [Fill] hard, moist, brown and reddish brown mottled, fine to medium sand, some fine subangular gravel, low plasticity	25	MC-1B	114	15			>4.5 ○
			Clayey Sand with Gravel (SC) [Fill] medium dense, moist, gray and brown mottled, fine to coarse sand, fine subangular to subrounded gravel	34	MC-2B	118	12			
	5			17	MC-3B	96	12			
			Silty Sand (SM) loose, moist, brown, fine to medium sand	6	MC-4B	83	20		29	
	10									
			Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel	18	SPT-5		24			
	15									
			loose	7	SPT					
	20									
				26	SPT-7B		16			
	25									

Continued Next Page

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, 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
			Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel							
				19	SPT					
	30									
				18	SPT-9		20			
	35									
			Clayey Sand (SC) loose, moist, grayish brown, fine to medium sand							
				6	SPT					
	40									
			Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand							
				20	SPT-11		23			
	45									
				13	SPT					
	50									
			Clayey Sand (SC) loose to medium dense, moist, grayish brown, fine to medium sand							
				8	SPT-13B		40		39	
	55									

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 3 OF 4

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, 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)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
		Clayey Sand (SC) loose to medium dense, moist, grayish brown, fine to medium sand							
	60	**Silty Sand (SM)** medium dense, moist, grayish brown, fine to coarse sand	17	SPT					
	65	**Clayey Sand (SC)** loose, moist, grayish brown, fine to medium sand	9	SPT-15		35			
		Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to medium sand							
	70	**Sandy Silt (ML)** medium stiff, moist, grayish brown, fine sand, low plasticity	14	SPT					
	75	**Silty Sand (SM)** medium dense, moist, grayish brown, fine to coarse sand, some fine subangular to subrounded gravel	22	SPT-17		28			
		Poorly Graded Sand with Silt (SP-SM) medium dense, moist, brown, fine to coarse sand	27	SPT					
	80 85	**Silty Sand (SM)** medium dense, moist, gray, fine sand	19 16	SPT-19 ○ NR		29			

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BORING NUMBER EB-2

PAGE 4 OF 4

CORNERSTONE EARTH GROUP

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, CA

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

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
			Silty Sand (SM) medium dense, moist, gray, fine sand							
				50	☒ SPT-20B		17			
	90		**Poorly Graded Sand with Silt (SP-SM)** medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel							
	95									
				88	☒ SPT-21B		12			
	100		Bottom of Boring at 99.5 feet.							
	105									
	110									
	115									

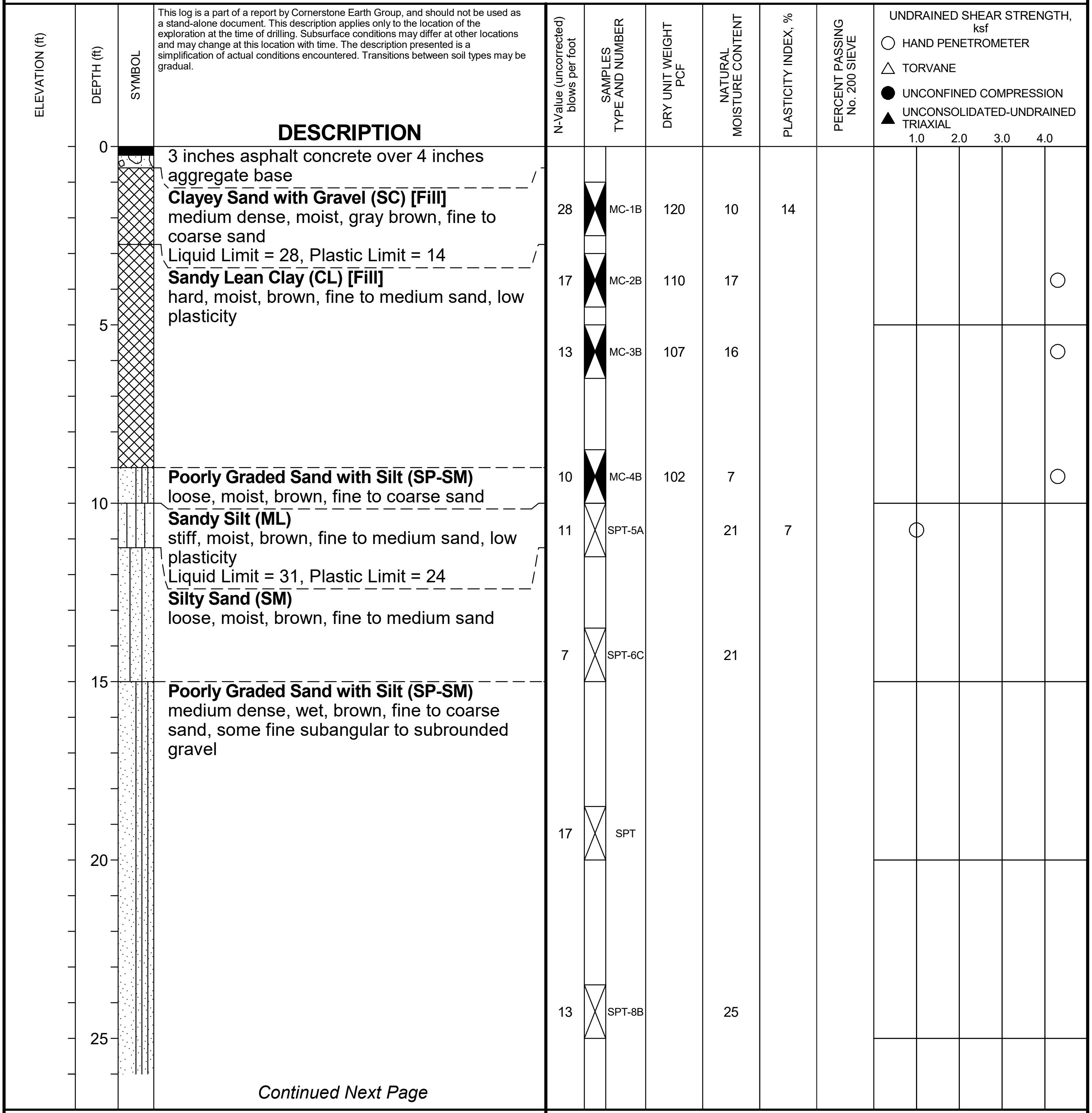
CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 2

PROJECT NAME Front Street Mixed Use
PROJECT NUMBER 100-49-1
PROJECT LOCATION Santa Cruz, CA
DATE STARTED 9/14/17 **DATE COMPLETED** 9/14/17
GROUND ELEVATION **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Britton Exlporation Services, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME 55 Track Rig, 4 inch rotary wash
GROUND WATER LEVELS:
LOGGED BY SCO
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

3 inches asphalt concrete over 4 inches aggregate base

Clayey Sand with Gravel (SC) [Fill]
medium dense, moist, gray brown, fine to coarse sand
Liquid Limit = 28, Plastic Limit = 14

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

Poorly Graded Sand with Silt (SP-SM)
loose, moist, brown, fine to coarse sand

Sandy Silt (ML)
stiff, moist, brown, fine to medium sand, low plasticity
Liquid Limit = 31, Plastic Limit = 24

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

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

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
28	MC-1B	120	10	14	
17	MC-2B	110	17		
13	MC-3B	107	16		
10	MC-4B	102	7		
11	SPT-5A		21	7	
7	SPT-6C		21		
17	SPT				
13	SPT-8B		25		

Continued Next Page

BORING NUMBER EB-3

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Front Street Mixed Use

PROJECT NUMBER 100-49-1

PROJECT LOCATION Santa Cruz, CA

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
			Poorly Graded Sand with Silt (SP-SM) medium dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel	12	SPT-9		15			
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

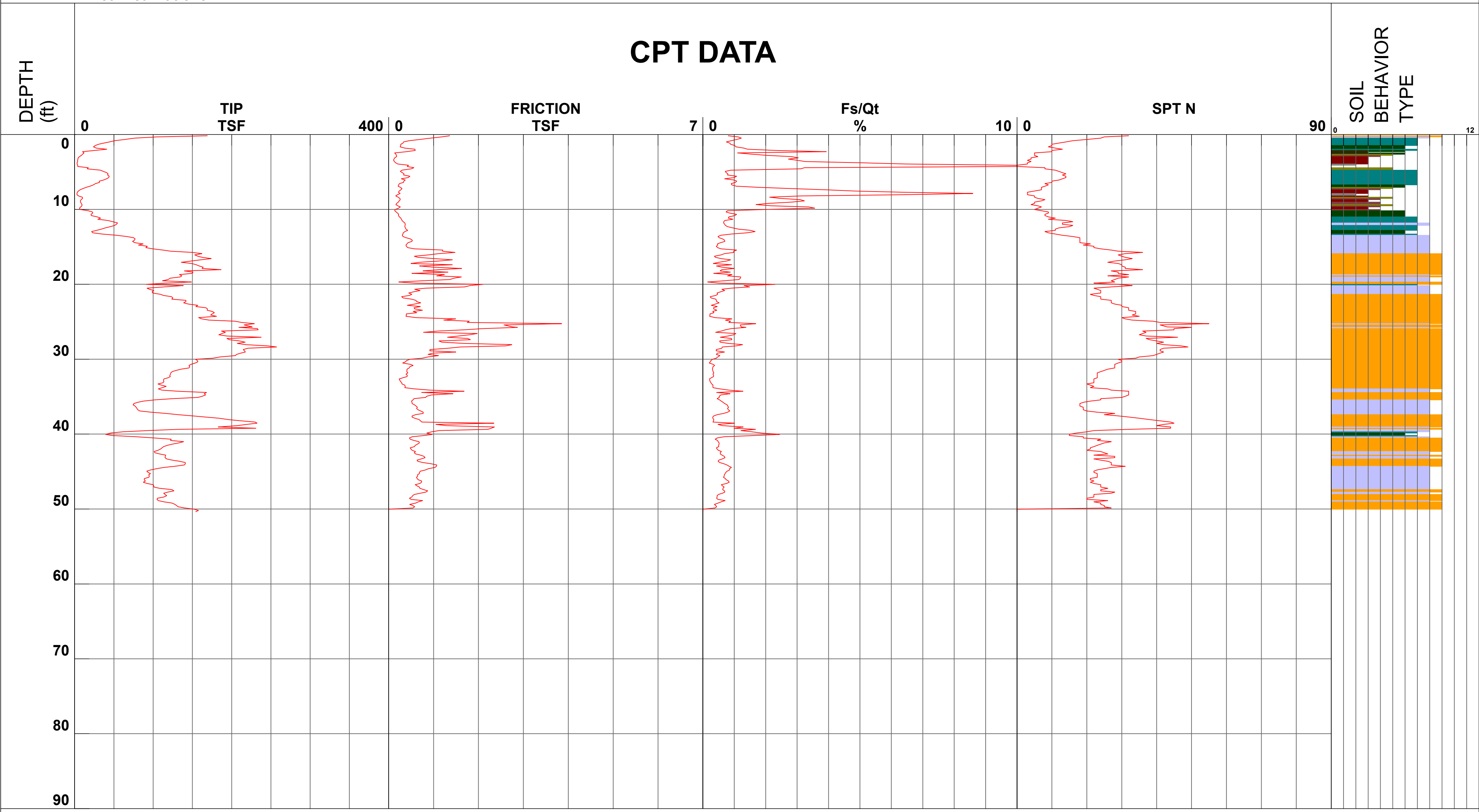
Project: Front Street Mixed Use
Job Number: 100-49-1
Hole Number: CPT-01
EST GW Depth During Test: 12.80 ft

Operator: RC-AS
Cone Number: DDG1418
Date and Time: 8/25/2017 10:32:44 AM

Filename: SDF(016).cpt
GPS:
Maximum Depth: 50.36 ft

Net Area Ratio .8

CPT DATA



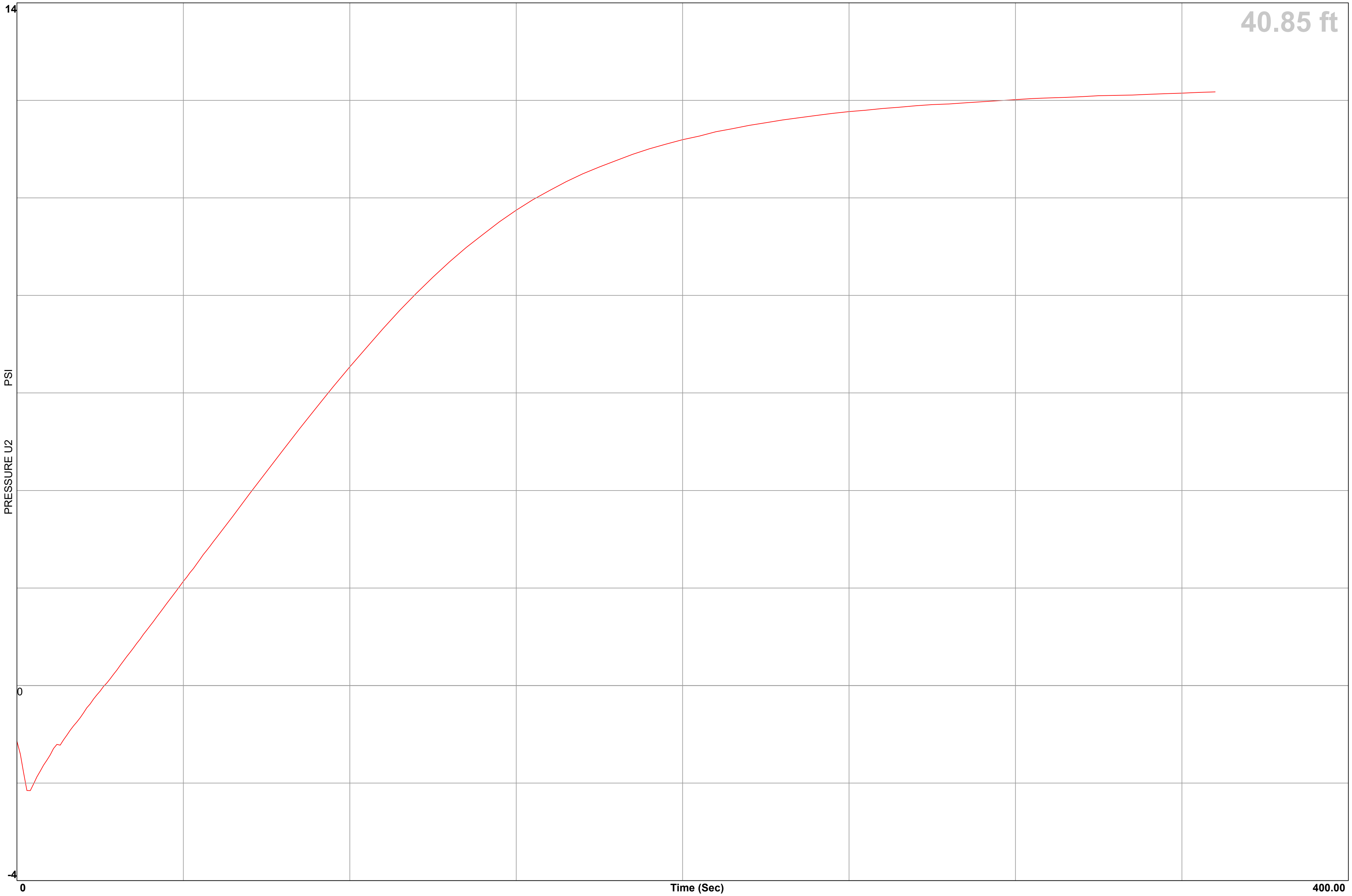
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey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

Cone Size 10cm squared

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

Cornerstone Earth Group

Location	Front Street Mixed Use	**Operator**	RC-AS		
Job Number	100-49-1	**Cone Number**	DDG1418	**GPS**	
Hole Number	CPT-01	**Date and Time**	8/25/2017 10:32:44 AM		
Equilized Pressure	12.1	**EST GW Depth During Test**	12.8		



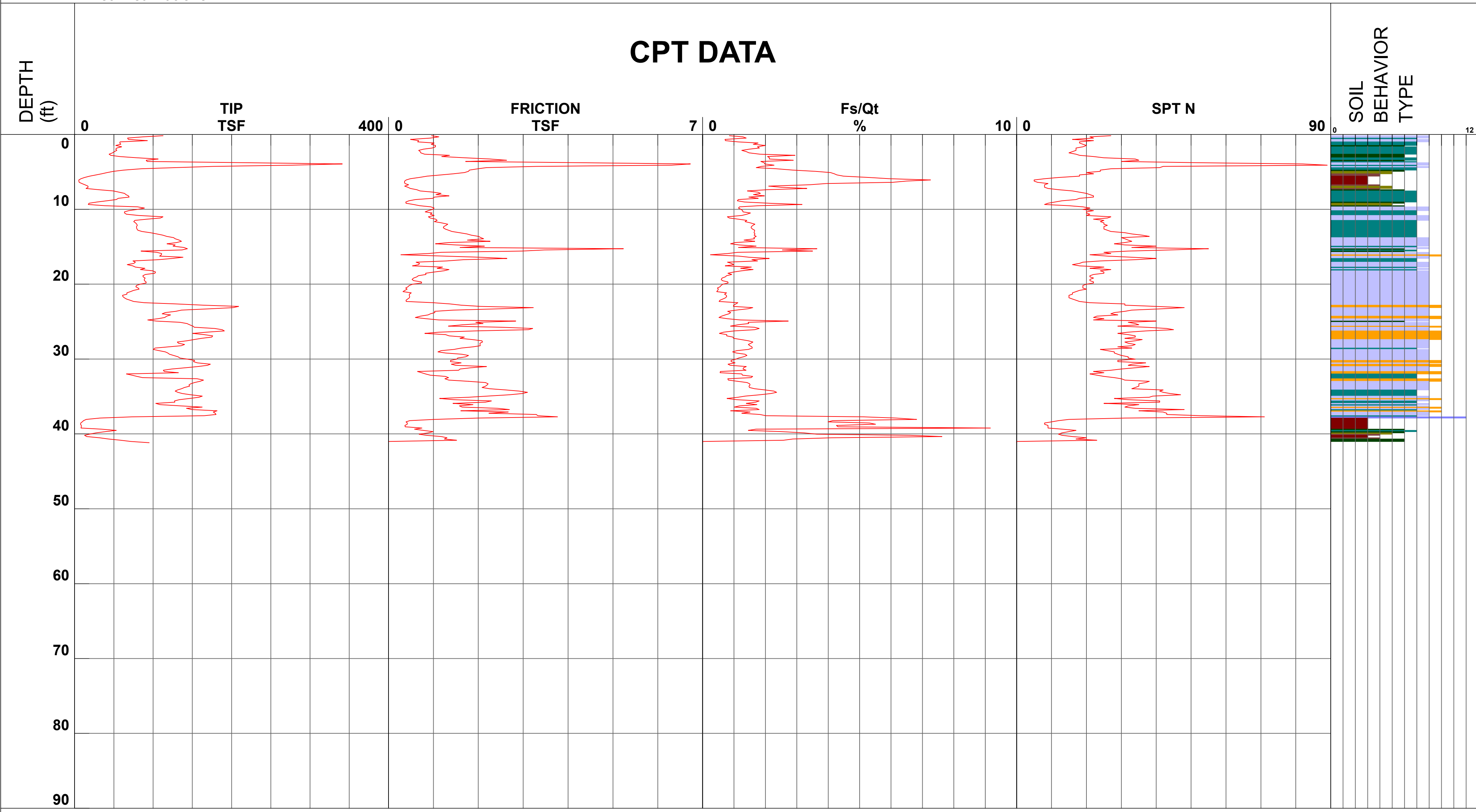
Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

Project	Front Street Mixed Use	Operator	RC-AS	Filename	SDF(022).cpt
Job Number	100-49-1	Cone Number	DDG1333	GPS	
Hole Number	CPT-02	Date and Time	8/25/2017 12:19:53 PM	Maximum Depth	41.17 ft
EST GW Depth During Test		12.10 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

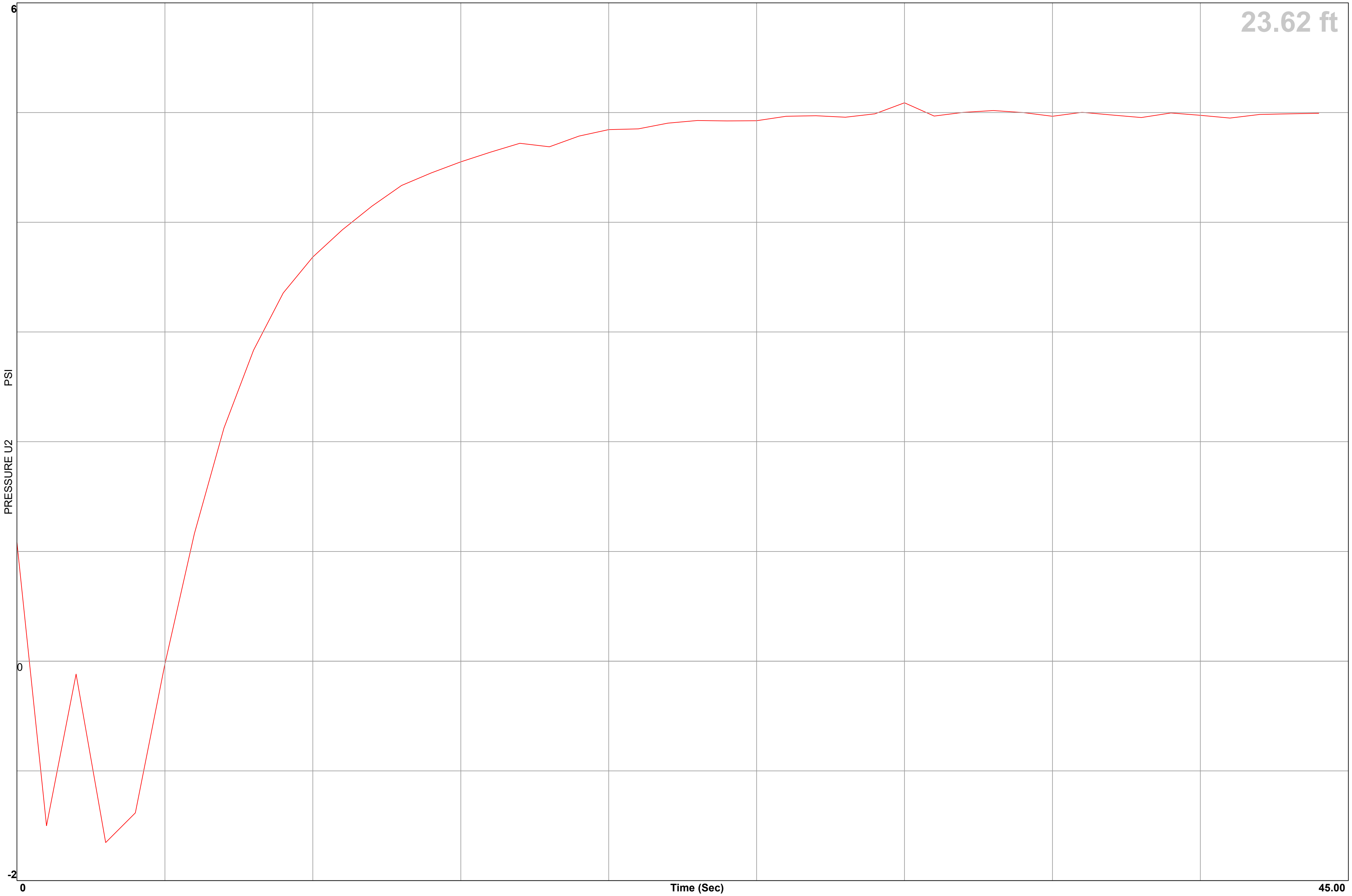
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Cornerstone Earth Group

Location	Front Street Mixed Use	**Operator**	RC-AS		
Job Number	100-49-1	**Cone Number**	DDG1333	**GPS**	
Hole Number	CPT-02	**Date and Time**	8/25/2017 12:19:53 PM		
Equilized Pressure	4.9	**EST GW Depth During Test**	12.1		

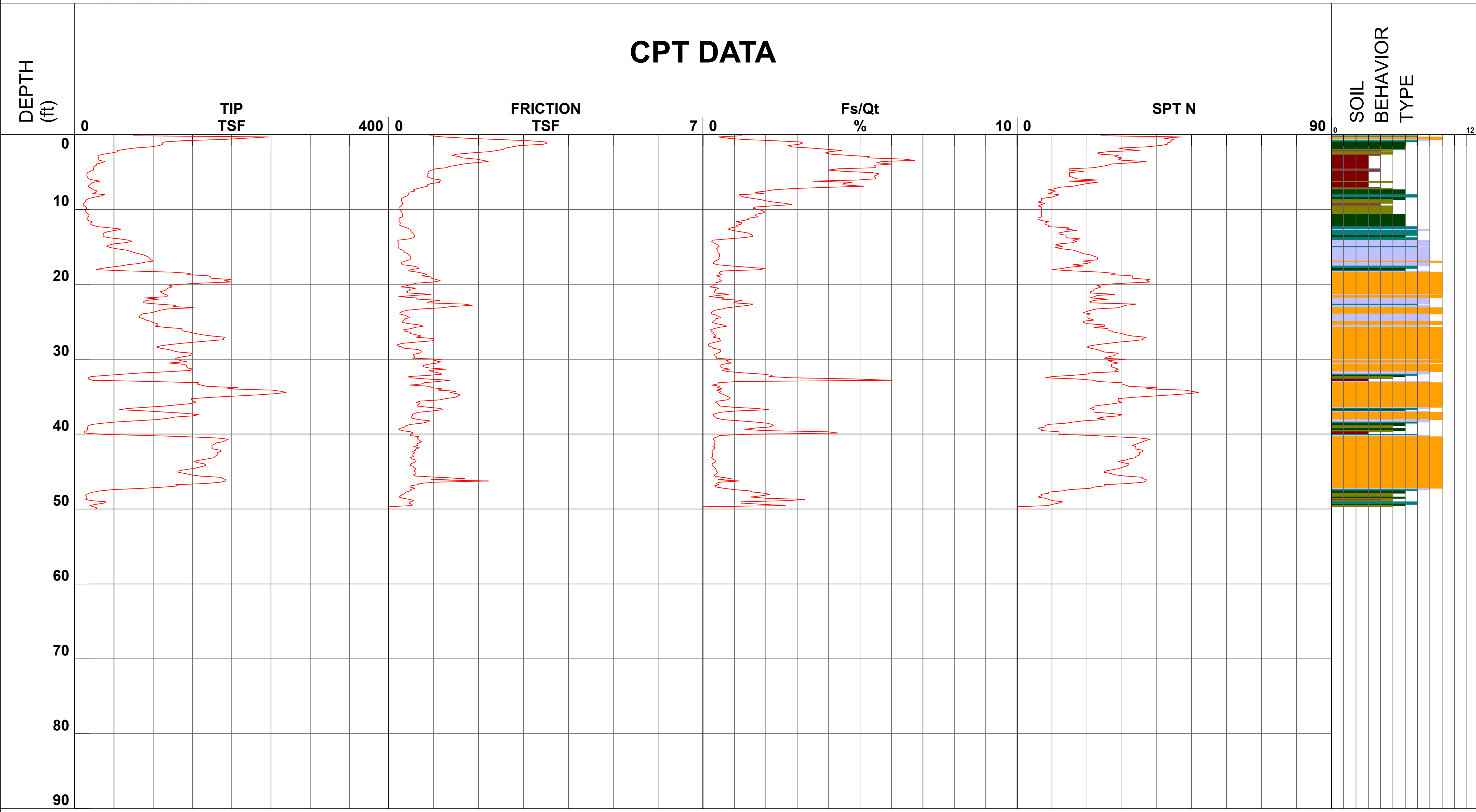


Cornerstone Earth Group

Project	Front Street Mixed Use	Operator	RC-AS	Filename	SDF(014).cpt
Job Number	100-49-1	Cone Number	DDG1418	GPS	
Hole Number	CPT-03	Date and Time	8/25/2017 7:18:11 AM	Maximum Depth	50.03 ft
EST GW Depth During Test		16.20 ft			

Net Area Ratio .8

CPT DATA



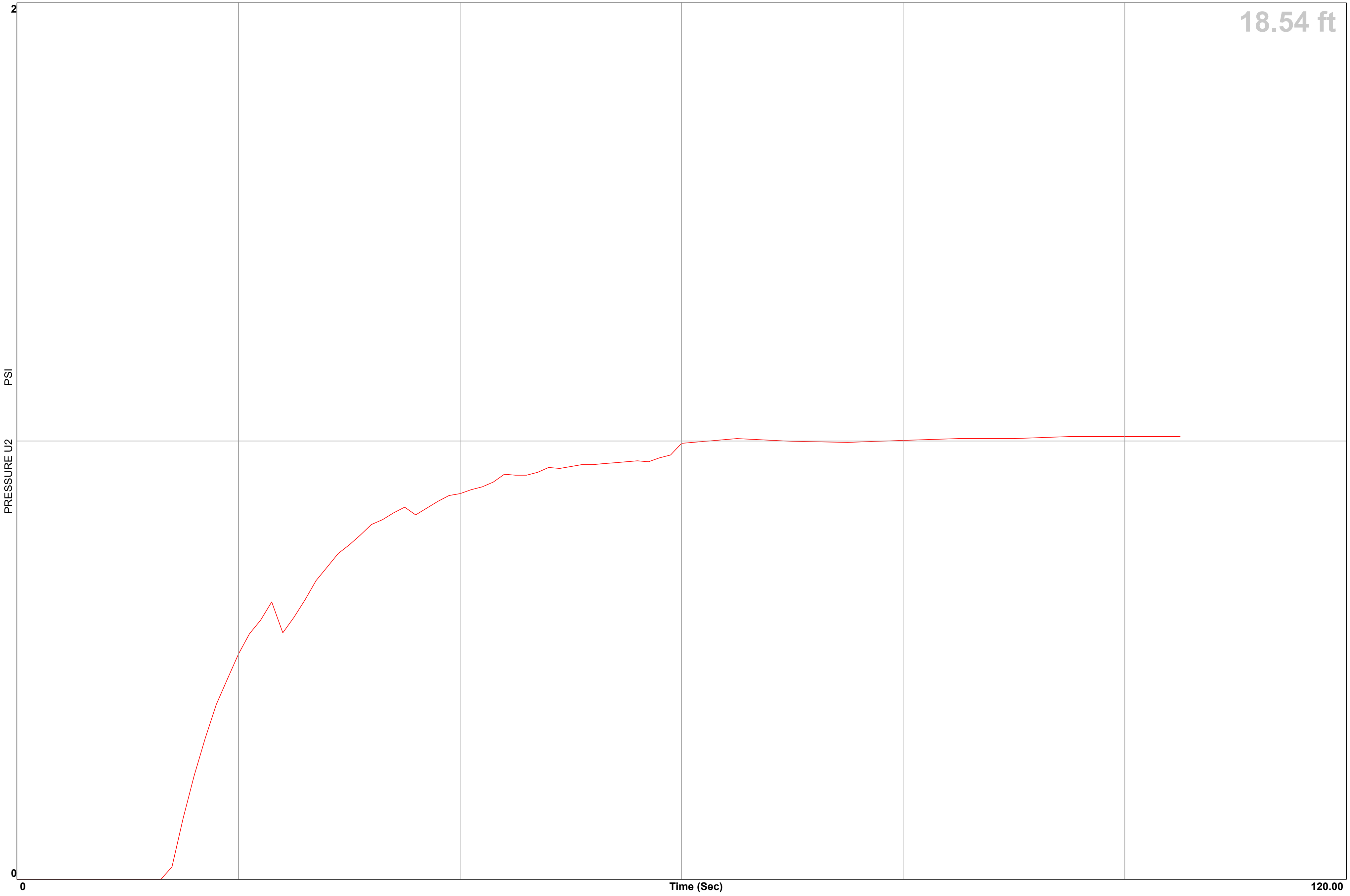
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Cornerstone Earth Group

Location	Front Street Mixed Use	**Operator**	RC-AS		
Job Number	100-49-1	**Cone Number**	DDG1418	**GPS**	
Hole Number	CPT-03	**Date and Time**	8/25/2017 7:18:11 AM		
Equilized Pressure	1.0	**EST GW Depth During Test**	16.2		

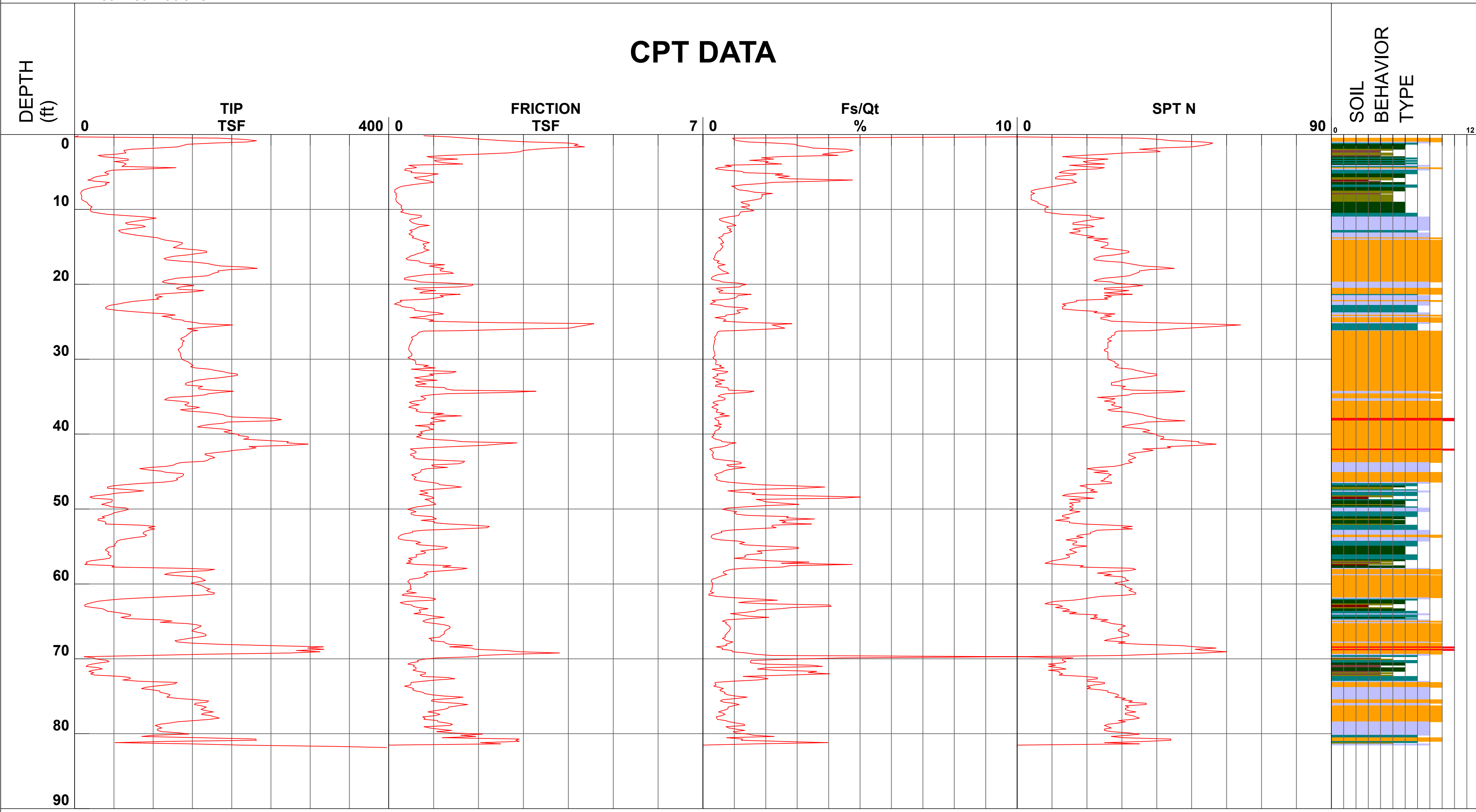


Cornerstone Earth Group

Project Front Street Mixed Use
Job Number 100-49-1
Hole Number CPT-04
EST GW Depth During Test 12.00 ft
Operator RC-AS
Cone Number DDG1418
Date and Time 8/25/2017 8:36:28 AM
Filename SDF(015).cpt
GPS
Maximum Depth 81.86 ft

Net Area Ratio .8

CPT DATA



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

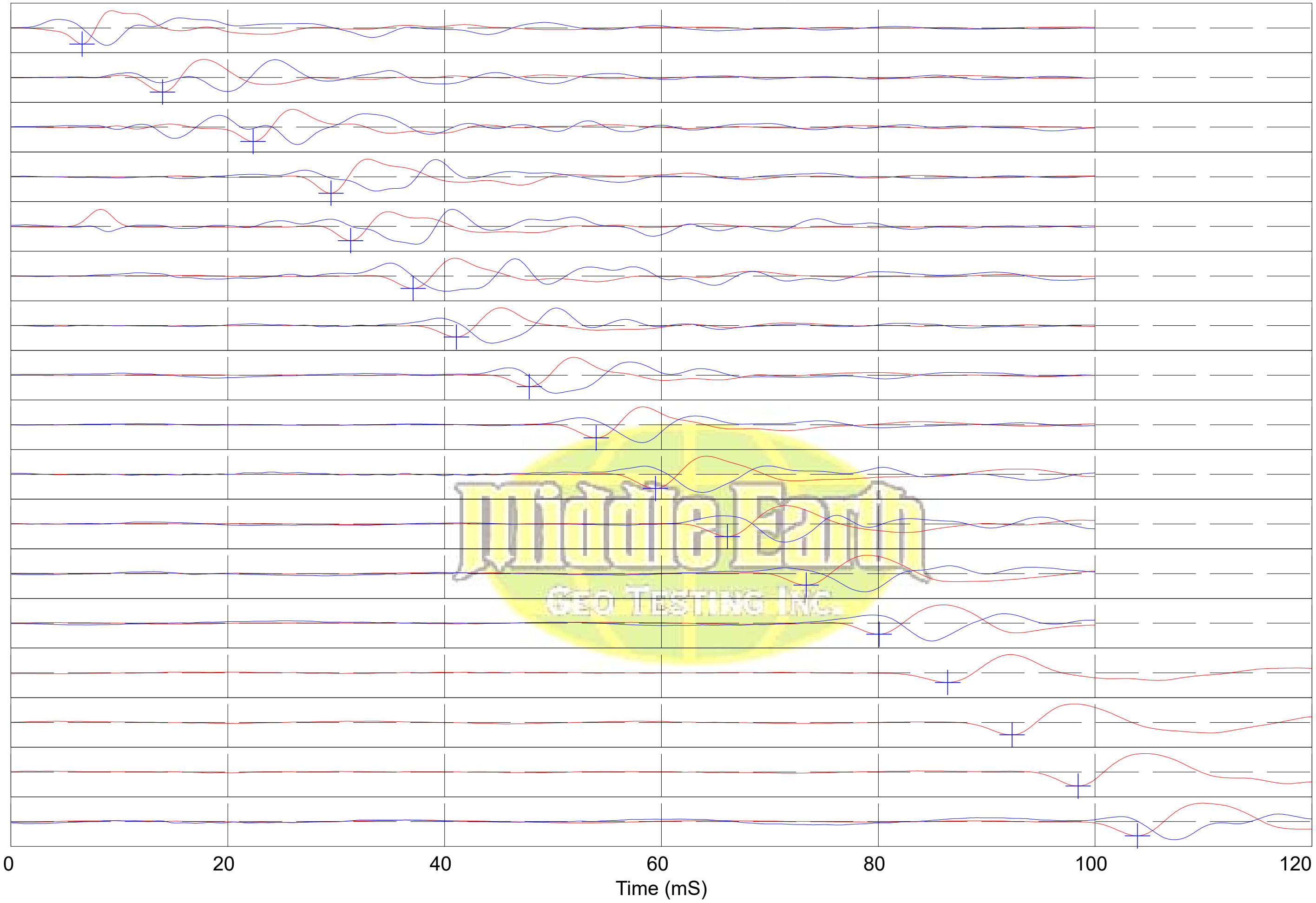
Cone Size 10cm squared

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

CPT-04

Cornerstone Earth Group

Front Street Mixed Use



Depth 4.92ft	Ref*	Arrival 6.56mS	Velocity*
Depth 10.01ft	Ref 4.92ft	Arrival 13.98mS	Velocity 532.35ft/S
Depth 14.93ft	Ref 10.01ft	Arrival 22.34mS	Velocity 531.70ft/S
Depth 20.01ft	Ref 14.93ft	Arrival 29.53mS	Velocity 670.49ft/S
Depth 21.33ft	Ref 20.01ft	Arrival 31.33mS	Velocity 702.91ft/S
Depth 25.10ft	Ref 21.33ft	Arrival 37.11mS	Velocity 632.86ft/S
Depth 30.02ft	Ref 25.10ft	Arrival 41.09mS	Velocity 1208.24ft/S
Depth 34.94ft	Ref 30.02ft	Arrival 47.81mS	Velocity 720.92ft/S
Depth 40.03ft	Ref 34.94ft	Arrival 53.98mS	Velocity 814.16ft/S
Depth 44.95ft	Ref 40.03ft	Arrival 59.45mS	Velocity 891.55ft/S
Depth 50.03ft	Ref 44.95ft	Arrival 66.09mS	Velocity 760.11ft/S
Depth 54.95ft	Ref 50.03ft	Arrival 73.35mS	Velocity 673.23ft/S
Depth 60.04ft	Ref 54.95ft	Arrival 80.07mS	Velocity 753.06ft/S
Depth 65.12ft	Ref 60.04ft	Arrival 86.40mS	Velocity 800.18ft/S
Depth 70.05ft	Ref 65.12ft	Arrival 92.34mS	Velocity 825.82ft/S
Depth 74.97ft	Ref 70.05ft	Arrival 98.43mS	Velocity 805.04ft/S
Depth 80.05ft	Ref 74.97ft	Arrival 103.90mS	Velocity 927.32ft/S

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

HOLE NUMBER: CPT-04

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.

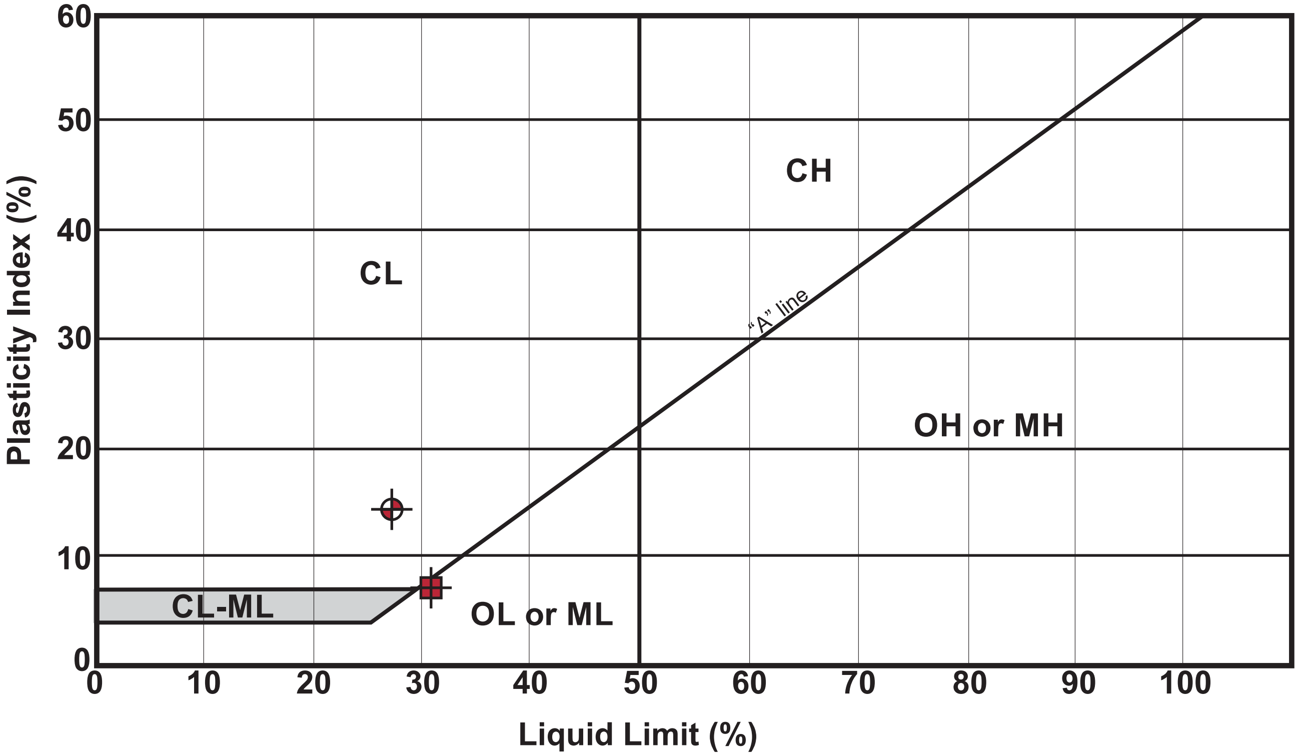
Moisture Content: The natural water content was determined (ASTM D2216) on 32 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 12 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 four samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

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-3	2.0	10	28	14	14	—	Clayey Sand (SC) (CL fines) [Fill]
⊞	EB-3	10.5	21	31	24	7	—	Sandy Silt (ML)

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Plasticity Index Testing Summary

Front Street Mixed Use
Santa Cruz, CA

Project Number
100-49-1

Figure Number
Figure B1

Date
October 2017

Drawn By
FLL