CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Downtown Library Residential Mixed-Use
LOCATION	Cathcart Street and Cedar Street Santa Cruz, California
CLIENT	For the Future Housing, Inc.
PROJECT NUMBER	1271-2-1
DATE	June 2, 2022

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Downtown Library Residential Mixed-Use**
Location	**Cathcart Street and Cedar Street Santa Cruz, California**
Client	**For the Future Housing, Inc.**
Client Address	**950 S. Bascom Avenue, Suite 1014 San Jose, California**
Project Number	**1271-2-1**
Date	**June 2, 2022**

Prepared by

Stephen C. Ohlsen, P.E.
Project Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
STEPHEN C. OHLSEN
C 90614
CIVIL
STATE OF CALIFORNIA

John Dye, P.E., G.E.
Senior Principal Engineer
Quality Assurance Reviewer

1259 Oakmead Parkway | Sunnyvale, CA 94085
T 408 245 4600 | F 408 245 4620

1220 Oakland Boulevard, Suite 220 | Walnut Creek, CA 94596
T 925 988 9500 | F 925 988 9501

www.cornerstoneearth.com

TABLE OF CONTENTS

Type of Services | **Geotechnical Investigation**
Project Name | **Downtown Library Residential Mixed-Use**
Location | **Cathcart Street and Cedar Street**
Santa Cruz, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of For the Future Housing, Inc. for the Santa Cruz Library Residential 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:

- Design workshop meeting package plan set titled "Downtown Library Mixed Use" prepared by Ten Over, dated February 3, 2022.
- Entitlements Package Draft plan set titled "Downtown Library Mixed Use" provided by Ten Over, dated April 14, 2022.

1.1 PROJECT DESCRIPTION

The project will include redeveloping the approximately 1¼-acre, generally rectangular site for a new library and multi-family residential mixed-use development. The new residential building will extend one-level below grade and up to eight floors above grade and will likely by of concrete-, steel- and wood-frame construction. The residential tower portion will be located along the eastern half of the building, and the library front along Cedar Street will be up to three floors with no below-grade parking. The below-grade level and first three floors of the residential tower are planned for parking, and the remaining floors are planned for residential units. The western half of the building will be comprised of the library, commercial space, and a daycare. A mezzanine, patio and "green roof" will be located above the library at the fourth floor. A 12-foot-wide alley is planned along the eastern property line between the new residential tower and the existing structures to the east. Appurtenant utilities, bioretention basins, and landscaping are also planned for development.

Structural loads are not available at this time; however, structural loads are expected to be typical for similar mid-rise structures. Estimated cuts and fills up to about 1 to 3 feet are expected for the at-grade building and 10 to 12 feet for the below grade parking garage.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated March 1, 2022 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 April 18 and 19, 2022 with truck-mounted hollow-stem auger drilling equipment and four Cone Penetration Tests (CPTs) advanced on April 11, 2022. The borings were drilled to depths of 60 to 80 feet; the CPTs were advanced to depths of 50 to 80 feet before encountering practical refusal. Seismic shear wave velocity measurements were collected from CPT-2 and CPT-4. Borings EB-1 and EB-3 were advanced adjacent to CPT-1 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; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, Plasticity Index, and washed sieve analyses. 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

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) 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%), 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 Fault.

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)
Montery Bay-Tularcitos	6.3	10.2
Zayante-Vergeles	8.1	13.0
San Gregorio	9.9	15.9
San Andreas (1906)	11.3	18.2
Sargent	12.5	20.1

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

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

We reviewed historical aerial imagery provided online by Historical Aerials (http://www.historicaerials.com). A summary of pertinent surface changes at and near the site is as follows:

- 1952: The project site is occupied by two buildings and three asphalt concrete parking lots located off Center Street.
- 1956: Two buildings have been constructed on the eastern and western portions of the project site.
- 1968: The western building has been demolished and replaced with an asphalt parking area, and the eastern building has been rebuilt with a smaller footprint outside of the project site.
- 1982: The western half of the northern building has been demolished and replaced with asphalt parking lots.
- 2005: The southern building has been rebuilt with a smaller footprint, and no longer occupies the southeastern portion of the site. An asphalt parking area has taken its place, and the project site appears to remain relatively unchanged since the 2005 photo.

3.2 SURFACE DESCRIPTION

Based on the parcel map provided by the County of Santa Cruz, the approximately 1¼-acre site is comprised of two parcels. Parcel 1 is comprised of the asphalt parking lot along Center Street and occupies the majority of the site. Parcel 2 is in the northeastern portion of the site and includes a two-story commercial building to be demolished for the planned development. The site is relatively level but graded to drain to existing storm drainage facilities.

At our exploration locations, surface pavements generally consisted of 2 to 5 inches of asphalt concrete over 5 to 8 inches of aggregate base. Based on visual observations overall, the existing pavements are in poor condition, with areas of significant alligator and transverse cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered existing undocumented fill underlain by native alluvial soil to the maximum depths explored during this investigation. A more detailed description of the subsurface conditions is presented in the following sections.

3.3.1 Undocumented Fills

Below the surface pavements, our borings generally encountered approximately 1 to 5½ feet of undocumented fill. The fills were highly variable in content and generally consisted of loose silty sands and loose poorly graded sand with silt. Abundant brick debris was encountered in Boring EB-1 at a depth of about 3 to 5 feet.

3.3.2 Alluvial Soils

Below the undocumented fill in exploratory Boring EB-1, the native alluvial soils consisted of loose to medium dense silty sand and poorly graded sand with silt to a depth of about 16 feet below existing site grades, followed by dense to very dense poorly graded sand with varying amounts of silt and gravel to the terminal boring depth of 60 feet.

At Boring EB-2, the fill is underlain by native alluvial soils consisting of very stiff lean clay with sand to a depth of about 6 feet, loose to medium dense silty sand and poorly graded sand with silt to about 27 feet below existing site grades, and dense to very dense silty sand and poorly graded sand with varying amounts of silt and gravel to the maximum boring depth of 80 feet.

Boring EB-3 encountered native alluvial soil below the undocumented fill consisting of very stiff lean clay with sand to a depth of about 5 feet followed by loose to medium dense silty sand and poorly graded sands with silt down to about 17 feet. The medium dense sands were underlain by dense to very dense poorly graded sand and silty sand to the terminal boring depth of 60 feet. A soft sandy silt layer was encountered between 32 and 36 feet below grade.

Below the terminal depth of our borings, our CPTs generally encountered poorly graded sands and silty sands to the maximum depth explored of approximately 80 feet. CPT-2 and CPT-4 encountered practical refusal in dense sand layers at depths of approximately 77 and 80 feet below existing grades.

3.3.3 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from about 6 percent under to 13 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

Groundwater was encountered in all three of our exploratory borings at depths ranging from 9 to 11 feet below current grades. CPT pore pressure measurements estimated groundwater depths of approximately 10 to 11 feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

We also reviewed groundwater data available online from the website GeoTracker, https://geotracker.waterboards.ca.gov/. Nearby monitoring well data indicates that groundwater has been measured at depths of approximately 7 to 10 feet below existing site grades at wells located at 1018 Pacific Ave (approximately 300 feet east) between 2004 and 2012.

Based on the above, we recommend a design groundwater depth of 7 feet below current site grades. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE 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 surface 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 site modified peak ground acceleration (PGA_M) was determined in accordance with the 2019 California Building Code (CBC) and Section 21.5 of ASCE 7-16. Therefore, we recommend a site-specific MCE_G peak ground acceleration, PGA_M, of 0.64g for this project.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California but is within a zone mapped as having a high liquefaction potential by the City 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 groundwater depth of 7 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.

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

The results of our CPT analyses (CPT-1 through CPT-4) are presented on Figures 4A to 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 5⅔ to 6½ 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 3¾ to 4¼-inches over a horizontal distance of 30 to 40 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 7-foot-thick layer of non-liquefiable cap is insufficient to prevent ground deformation and significant surficial cracking; therefore, additional settlement and differential movement may occur during a seismic event at the site unless the near surface soils are improved. Ground deformation potential will be mitigated following installation of ground improvement. Additional discussion of ground improvement is presented in the "Foundations" section of this report.

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 current San Lorenzo River runs approximately 800 to 950 feet east of the proposed site. Based on review of Google Earth, the river bottom appears to be at approximately Elevation 4 feet. Therefore, the channel bottom is approximately 9 to 10 feet deep relative to existing site grades. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Based on our analysis, it appears that the potential for lateral spreading is moderate and could potentially result in lateral movement without ground improvement. Additional discussion of ground improvement is presented in the "Foundations" section of this report.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose to medium dense sands based on the work by Robertson and Shao (2010). Based on our analyses, the potential for significant seismic settlement affecting the proposed improvements is low. In addition, we anticipate that the below-grade basement will remove unsaturated sands within the building footprint.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately ⅔ miles inland from the Pacific Ocean shoreline, is approximately 15 to 16 feet above mean sea level, and lies within a mapped tsunami hazard zone. 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; 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.

The Department of Water Resources (DWR), Division of Safety of Dams (DSOD) compiled a database of Dam Failure Inundation Hazard Maps (DSOD, 2015). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is located within a dam failure inundation area for the Newell Reservoir.

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 significant seismic settlements
- Potential for ground deformation and significant surficial cracking
- Potential for lateral spreading
- Undocumented fill and re-development considerations
- Shallow groundwater
- Presence of cohesionless soils
- Shoring and underpinning considerations
- Differential movement at on-grade to on-structure transitions

5.1.1 Potential for Significant Seismic Settlements

As discussed, our liquefaction analysis indicates that there is a very high potential for liquefaction of localized sand layers during a significant seismic event. Our analysis indicates that liquefaction-induced settlement on the order of 5⅔ to 6½ inches could occur, resulting in differential settlement up to 4¼ inches. To mitigate the potential for significant differential movement, we recommend the structure be supported on shallow foundations overlying ground improvement. If conventional shallow footings with ground improvement are considered for the at-grade library, the ground floor slab will either need to be underlain by ground improvement as well, or be designed as a structural slab that is capable of spanning between footings unsupported. A discussion of potential mitigation options is presented in the “Foundations” and “Slabs-on-Grade” sections.

5.1.2 Potential for Ground Deformation and Significant Surficial Cracking

The potential for ground deformation and significant surficial cracking is considered high and is correlated with the high potential for seismic settlement previously discussed. The resulting soil ejecta (sand boils), as observed in the Santa Cruz downtown during the 1989 Loma Prieta earthquake, could potentially occur near the edges of the site or in areas where ground improvement is not performed. Additional settlement and differential movement may occur during a seismic event at the site unless the near surface soils are improved. As discussed above, typical techniques to mitigate the potential ground deformation include ground improvement. and A reinforced concrete mat foundation can also be used to mitigate differential settlement and to provide confinement of liquefiable layers. A discussion of ground deformation mitigation options is presented in the "Foundations" section of this report.

5.1.3 Potential for Lateral Spreading

As discussed, there is a potential for lateral spreading towards the nearby San Lorenzo River. The potential for lateral displacement affecting the proposed improvements is high. As discussed above, we understand that the eastern half of the proposed structure will be supported on a single-level below-grade garage and that the entire structure, at- and below-grade, will be underlain by ground improvement mitigation. As such, we anticipate the potential for lateral displacement affecting the proposed improvements to be mitigated. A discussion of ground improvement options is presented in the "Foundations" section.

5.1.4 Undocumented Fill and Redevelopment Considerations

We encountered approximately 1 to 5½ feet of undocumented fill in our explorations and anticipate that fill may exist across much of the site due to previous development and grading. While we anticipate that most of the fill will be removed during the excavation of the below-grade garage, any at-grade building areas will be underlain by fill that may not provide uniform support for slabs-on-grade or other site improvements. Since the proposed library and residential mixed-use building will be supported on either shallow footings overlying ground improvement or a mat foundation overlying ground improvement at the basement level, in our opinion, a complete over-excavation and removal of existing fill is not required for the structure. However, after the ground improvement has been completed, the upper 2 feet of the at-grade building pad and at the basement level should be re-compacted to repair any damages that may have occurred and provide uniform support for slabs-on-grade or mat foundations. Further recommendations for mitigation of the existing fills are presented in the "Earthwork" section of this report.

Additionally, as discussed, the site is currently occupied by an existing building and appurtenant flatwork, site fixtures, and landscaping. We understand that all the existing improvements will be demolished for the construction of the new building. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, old foundations and slabs, and localized undocumented fills that may be deeper than encountered in our borings. Please refer to the "Earthwork" section below for further recommendations.

5.1.5 Shallow Groundwater

Shallow groundwater was measured at depths ranging from approximately 9 to 11 feet below the existing ground surface. As discussed above, we recommend a design groundwater depth of 7 feet below existing grades. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement and basement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches will be required. Foundations extending below the design groundwater level should be designed to resist hydrostatic pressures and be waterproofed. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.6 Presence of Cohesionless Soils

As mentioned, the site is underlain by cohesionless, sandy soils with low fines content. The sandy soils are not likely to stand vertical when excavated and excavation sidewalls for foundations, utility trenches, temporary slopes, basement excavation, etc. may cave in or accumulate significant amount of slough. Grading and excavation contractors should be made aware of this condition and plan on forming footings, preparing subgrade just prior to concrete placement, and other similar construction issues as relates to temporary shoring, utility excavations, etc. These issues are addressed within the “Earthwork” and “Foundations” sections of this report.

5.1.7 Shoring and Underpinning Considerations

For a one level below-grade basement, an approximately 12 to 15 feet deep excavation will likely be required for a shallow foundation excavation. Locally deeper excavations will be required if auto stacker pits are considered or for elevator pits. The adjacent buildings, sidewalks, streets and utilities along the sides of the site should be supported by temporary shoring until the permanent basement walls have been constructed. The primary considerations in selecting a suitable shoring system typically include 1) control of vertical and lateral ground surface or wall movements, 2) constructability, 3) dewatering and 4) cost. There are several possible methods of providing lateral support for the excavation, including a soldier pile and lagging retaining system, soldier pile tremie concrete (SPTC) walls or mixed-in-place soil/cement walls.

All systems would require tiebacks or internal bracing for lateral support. A soldier pile and lagging retaining system is more flexible and pervious than either an SPTC or mixed-in-place soil/cement wall. The latter two types of walls would be relatively rigid and could significantly limit lateral deflections and ground movement related to the shoring. In addition, SPTC or mixed-in-place soil/cement walls are relatively impervious and would reduce the volume of water pumped to dewater the site. The disadvantages of these systems are cost and space requirements, as they may require 2 to 3 feet around the perimeter of the site. A combination of these systems could be used depending on the performance desired along the various excavation faces. For example, some of the basement walls may encounter more permeable silt and sand layers that may be susceptible to sloughing or caving and would likely require greater volume of groundwater pumping. Where movements could be detrimental to adjacent existing buildings/improvements or it is not practical to install underpinning, the stiffer shoring systems could be used. The shoring system selected should be designed by a shoring designer or structural engineer experienced in the specific type of construction.

If the excavation extends below the level of an adjacent building foundation, lateral support should be provided to prevent loss of ground beneath existing slab-on-grade floors. Where adjacent foundations are above an imaginary 1:1 (horizontal to vertical) line extending up from the base of the excavation, they should be underpinned unless the shoring can be designed to provide lateral and/or vertical support for the structure. Additional design and construction considerations for the shoring system include the following items:

1. Soldier pile and lagging wall below the groundwater may experience difficulties with seepage, localized flowing sand and possible increased wall movement.
2. Adjacent structures may need to be underpinned to protect from ground movement associated with the proposed shoring system. Slant piles will likely be an acceptable method to underpin adjacent structures, although other methods are available. Underpinning will likely need to extend into competent soil below the excavation level.
3. The shoring will need to extend deep enough to reduce the potential for base heave, groundwater piping, and/or bearing failure.
4. Tie-backs in the upper loose to medium dense sands will likely require a smooth-cased tieback method and pressure grouting to develop sufficient bond strengths.
5. Internal bracing may be required in areas where tie-back encroachment is not feasible or allowed by adjacent property owners.
6. The contractor should establish survey points on the shoring and on adjacent improvements within 25 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding improvements during construction. In addition, a thorough crack survey of the adjacent buildings should be performed by the project surveyor prior to the start of construction and immediately after its completion.

Recommendations for design of temporary shoring, tie-back anchors, dewatering and underpinning are presented in the following sections of this report.

5.1.8 Differential Movement at On-Grade to On-Structure Transitions

We anticipate areas adjacent to the planned basement that will have flatwork areas that may transition from on-grade support to overlying the basement. 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.

If flush shoring is not utilized and engineered fill is placed behind retaining walls extending to near finished grade, we recommend consideration be given to dowels between the pavement and building or subslabs beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. Hinge slabs and subslabs should be considered at these transitions at garage entrances. 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). At this time, we do not anticipate any at-grade portions of the building to extend beyond the basement limits. We should be consulted if this changes.

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. 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. Based on our site observations, surficial stripping should extend about 4 to 6 inches below existing grade in vegetated areas that will have at-grade improvements.

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 RE-COMPACTION OF UNDOCUMENTED FILLS

As the building will be supported on ground improvement elements, we recommend that the upper 2 feet of the building pad be over-excavated following the ground improvement installation to re-compact areas disturbed by the ground improvement process and to provide a uniform support for the proposed slab-on-grade or mat foundation. Depending on the final building pad elevation and foundation type, the depths of the over-excavation may be modified in the field.

Provided the fills meet the "Material for Fill" requirements below, the shallow fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 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 20 feet at the site may be classified as OSHA Soil Type C materials. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

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 planned cuts up to 15 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	40 pcf
Restrained Wall – Uniform Earth Pressure	25H*
Passive Pressure – Starting at 2 feet below the bottom of the excavation	375 pcf up to 3,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

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 hollow-stem auger 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 groundwater) 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

Groundwater levels are expected to be as high as 5 to 7 feet above the planned excavation bottom; therefore, temporary dewatering will 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 groundwater 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 groundwater 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.

6.5.3 Underpinning

Where foundations for adjacent buildings are above an imaginary 1:1 line drawn up from the bottom of the proposed basement excavation, they should be underpinned, or the shoring should be designed to provide vertical and lateral support for adjacent structures. If underpinning is required, we judge slant piles or offset augercast piles will be acceptable methods to underpin adjacent structures. On a preliminary basis, underpinning piles/piers may be designed using an ultimate frictional resistance of 800 pounds per square foot, provided they are embedded at least 15 feet below the basement excavation level. The underpinning designer should apply an appropriate factor of safety to the above ultimate capacity, as required. To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be drilled or excavated concurrently. We recommend underpinning piers should be preloaded prior to dry packing. We should observe the installation of the underpinning piers to check that adequate embedment has been achieved.

If slant piles are used, they should be designed by the underpinning contractor, and we should review the geotechnical aspects of the underpinning design.

6.6 AT-GRADE 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 recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

6.7 WET SOIL STABILIZATION GUIDELINES

Native soil 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 13 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 site conditions.

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 8 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 geosynthetic (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.

6.7.4 Below-Grade Excavation Stabilization

The proposed building excavation will extend into saturated silt and sand with varying strength. Due to the high moisture content of these materials, it will likely become unstable under the weight of track-mounted or rubber-tired construction equipment. To provide a firm base for construction of the foundation, it may be necessary to remove approximately 12 to 18 inches of native soil below the foundation level and replace it with a bridging layer, such as crushed rock and a layer of stabilization fabric, such as Mirafi HP 370A or approved equivalent. 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. Lime and/or cement treatment can also be considered for the upper 12 to 18 inches of exposed basement soils, which would likely require 4 to 5 percent lime or cement to create a bridging layer. Lastly, a layer of lean cement-sand slurry layer ("rat slab") may be considered or a combination of the two. Temporary dewatering to a depth of at least 3 to 5 feet below the bottom of the building excavation is recommended during construction.

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 significant quantities of asphalt concrete (AC) grindings and aggregate base (AB). 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 groundwater). AC grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

6.8.3 Potential Import Sources

Imported soil for use as general fill 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 and 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 "Wet Soil Stabilization Guidelines" section of this report.

Table 2: 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
	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 (Marshall)	NA

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

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.

- The near-surface soils at the site are clayey and categorized as Hydrologic Soil Group C, and is expected to have infiltration rates on the order of ½ to 1 inch per hour.

- Seasonal high groundwater is not mapped in the area but was encountered as high as 8 feet below grade in our explorations, and therefore is expected to be within 10 feet below the base of infiltration measures.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

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

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

- 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 soil.

- 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.

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 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.

- 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: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

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

7.1.1 Site Location and Provided Data For 2019 CBC Seismic Design

The project is located at latitude 36.972057° and longitude -122.026600°, which is based on Google Earth (WGS84) coordinates at the approximate center of 600 Cedar Street in Santa Cruz, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2019 CBC SEISMIC DESIGN CRITERIA

As discussed in the “Subsurface” of our report, our CPT and exploratory borings encountered medium dense to dense sands and soft to very stiff silt and clay deposits to a depth of 80 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-4, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 225 meters per second (738 feet per second), for the upper 100 feet.

7.2.1 2019 CBC Seismic Design

As our borings encountered deep alluvial soils with shear wave velocity for the upper 30 meters between 600 and 1200 feet per second, per section 20.3.2 of ASCE 7-16, we have classified the site as Soil Classification D, which is described as a “stiff soil” profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as “determined” for the purposes of estimating the seismic design parameters from the code. Our site-specific ground motion hazard analysis considered a V_{S30} of 225 m/s (738 ft/s).

We note that due to the potential for liquefaction and the potential for affects to the proposed structures appear high, based on Section 20.3.1 of ASCE 7-16, the site should be classified as Site Class F and a site response analysis in accordance with Section 21.1 of ASCE 7-16 shall be performed, unless the proposed structures meet the following exception:

> **EXCEPTION:** For structures that have fundamental periods of vibration equal to or less than 0.5s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

If ground improvement is performed under the entirety of the building, including under the at-grade and below-grade portions, to mitigate liquefaction settlement estimates in accordance with recommendations provided in the “Ground Improvement” sections below, in our opinion, a Site Classification of D is still valid even if the structures’ periods are greater than 0.5 seconds. If ground improvement is not performed, then additional geotechnical analysis and review will need to be performed to see if a site-specific response analysis is required.

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see Figure 5. The recommended design spectral accelerations and associated periods are provided in graphically on Figure 6.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

As discussed in the “Conclusions” section, we recommend that the proposed structure be supported on shallow foundations overlying ground improvement to mitigate the potential for liquefaction-induced settlement and lateral spreading. Ground improvement can be used to mitigate the settlement to tolerable levels and, provided the recommendations in the “Earthwork” section and subsequent sections below are followed, the proposed structures may be supported on shallow foundations. We recommend a design-build ground improvement contractor design the mitigation using an appropriate ground improvement technique to meet the project requirements. Foundation recommendations are presented in the following sections.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings – At-Grade

Provided ground improvement is performed in accordance with recommendations in this report, we anticipate that the at-grade portions of the buildings and at-grade improvements may be supported on conventional footings. Continuous and/or spread footings should bear on uniformly spaced ground improvement elements, 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 ground improvement technique and spacing; however, substantial improvement in bearing capacity and reduction in settlement would be expected. On a preliminary basis, we expect allowable bearing pressures of at least 4,000 psf for combined dead plus live loads would be feasible with a one-third increase for all loads, including wind and seismic.

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 a tolerable level as discussed below.

8.2.2 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 375 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.3 Conventional Shallow 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.

Due to the presence of clean sand and silts, at-grade footing excavation walls will likely not stand vertical and will need to be sloped to a minimum 1:1 inclination or Stay-Form or similar may need to be placed within the footing excavations as they are excavated during construction of the foundation elements. Granular material encountered in the footing bottoms will likely be disturbed to a depth of 6 to 8 inches following excavation and will need to be compacted to 90 percent relative compaction prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. We should re-observe the footing excavations in granular materials after reinforcing steel has been placed and just prior to concrete placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material after we have observed the footing excavation to protect the granular material prior to steel placement.

8.2.4 Hydrostatic Uplift and Waterproofing

Where the structure will extend below the design groundwater level, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design groundwater should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design groundwater level, a drainage system may be added as discussed in the “Retaining Wall” section.

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

8.2.5 Reinforced Concrete Mat Foundations – At-Grade or Basement Level

Provided ground improvement is performed in accordance with recommendations in this report, the proposed at-grade and below-grade structures may be supported on a mat foundation bearing on uniformly spaced ground improvement elements and designed in accordance with the recommendations below. Reinforced concrete mat foundations should be designed in accordance with the 2019 California Building Code.

On a preliminary basis, the mat should be designed for a maximum *average* allowable bearing pressure of 2,000 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 4,000 psf. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement. If the actual average areal bearing pressure is higher than presented above, or if there are other aspects of design not accounted for in this report, please notify us so that we may revise our recommendations.

8.2.6 Mat Modulus of Soil Subgrade Reaction

The modulus of soil subgrade reaction is a model element that represents the response to a specific loading condition, including the magnitude, rate, and shape of loading, given the subsurface conditions at that location. Design experts recommend using a variable modulus of soil subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mats. This will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. We have assumed that the average areal mat pressure will be approximately 1,000 to 1,200 psf. Based on this assumed pressure, we calculated a preliminary modulus of subgrade reaction value for the mat foundation for unimproved ground.

For preliminary SAFE runs (or equivalent analysis), we recommend an initial modulus of soil subgrade reaction of 5 pounds per cubic inch (pci) for the mat foundation. As discussed above, the modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. As noted, this value represents the assumed soil response due to static and seismic deflection before ground improvement elements are considered. Updated modulus values for improved ground should be provided by the design-build contractor based on the type of ground improvement and estimated spacing.

8.2.7 Hydrostatic Uplift and Waterproofing

Mat foundations that extend below the recommended design groundwater level of 7 feet, should be designed to resist potential hydrostatic uplift pressures. Basement walls extending below design groundwater should be designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design groundwater level, a drainage system may be added as discussed in the "Retaining Wall" section.

In addition, the portions of the structures extending below design groundwater should be waterproofed to limit moisture infiltration, including mat foundation, all construction joints, and any basement retaining walls. We recommend that a waterproofing specialist design the waterproofing system.

8.3 GROUND IMPROVEMENT

As discussed above, conventional shallow footings or a rigid mat foundation supporting the mixed-use building may be used in combination with ground improvement. We recommend that ground improvement be performed within the at-grade and below-grade building footprint and extend to a tip elevation of at least Elevation 11 feet (WSG84 datum) to mitigate liquefaction settlement and lateral spreading. Ground improvement can be used to improve the subsurface soils such that the total combined static and seismic settlements are reduced to less than 1½ inches with ½ to ¾ inches differential settlement over a horizontal distance of 30 feet, enabling the structures to be supported on spread footings. Ground improvement should provide adequate confining improvement around all foundations. Ground improvement options should also include an increase in allowable bearing pressures and should reduce settlement to within the tolerances stated above.

8.3.1 General

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), grouted displacement columns (i.e. CLSM), or similar densification techniques.

Considering the close proximity to existing commercial properties and the potential presence of impacted soil at the 425 Pacific Street parcel, we assume that grouted displacement columns would be the preferred ground improvement method. The intent of the ground improvement design beneath the proposed building would be to increase the density of the potentially liquefiable sands within 25 feet from the surface by laterally displacing and/or densifying the existing in-place soils.

Grouted displacement 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 upper 2 feet of the working pad will likely need to be re-compacted after ground improvement installation, due to surface disturbance, potential localized ground heave and removal and re-compaction of undocumented fill. For this reason, we do not recommend preparation of the building pad or the construction of utilities prior to ground improvement.

The diameter of these ground improvement elements would be 24 to 30 inches and spacing would be proposed by the ground improvement contractors based on their experience and documented case histories of improvement performed on other projects with similar soil conditions which we would review as part of their submittal. The spacing would be estimated to improve the sands to obtain a post treatment $(N_1)_{60cs}$ of at least 20 to 25 blows/foot. The spacing would also be selected to reduce the total static settlement to 1½ inches with a differential settlement of ¾ inches over a horizontal distance of 30 feet. We would recommend a modulus test at the on-set of construction to verify that the ground improvement will control the static settlement. This recommendation is predicated on our working with and reviewing the ground improvement contractor’s submittal documentation on their proposed spacing and installation methodology and case histories from other similar projects. We would also independently observe installation in the field and prepare a signed and stamped close-out letter with confirms that installed ground improvement meets our recommendations.

8.3.2 Ground Improvement Design Guidelines

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, 5) case histories showing pre and post improvement $(N_1)_{60cs}$ or Q_{C1cs} values for projects with similar site conditions, 6) estimate of static settlement and modulus to meet settlement goals. We recommend that all displacement columns be capped with a minimum 6-inch-thick compacted gravel pad to facilitate load transfer and to decouple the footings from underlying ground improvement elements. The actual gravel pad thickness should be confirmed by the design-build contractor. We should be retained to review the ground improvement contractor’s plan and densification 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 previous performance testing. 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 the ground improvement on the approved layout, and 4) over-excavation and re-compact top of building pad, as required, prior to construction of remainder of pad and the foundations.

The degree to which the soil density is increased will depend on the improvement method and spacing. Even though the above methods are designed to mitigate different existing soil conditions, ground improvement should provide an additional increase in bearing capacity and soil stiffness at the individual improvement locations.

8.3.3 Ground Improvement Performance Testing

Foundation areas must meet the above total settlement criteria, which will include all settlement estimated from static loads. Analysis of settlement for static loading should include compression within the treatment area due to structural loads, and seismic settlement estimated for below the zone of treatment. Ground improvement must also provide adequate support for the design bearing capacity.

Verification testing should include at least two modulus tests within the building footprint. 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 CLSM column 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.

We recommend that at least two test array including pre- and post-installation CPT testing be performed. Performance testing typically consists of CPTs performed within each test array to confirm soil strength and density increases were achieved to meet the settlement criteria. 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 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

As discussed, we recommend that ground improvement be installed under at-grade footings. We also recommend the ground improvement elements be considered within slab-on-grade areas as well. As an alternative, the ground floor slab could be designed as a structural slab that is capable of spanning unsupported between footings and grade beams to reduce slab settlement or distress following a design level earthquake.

It should be noted that if ground improvement is not performed within the slab-on-grade areas, slab settlement or deflection will occur following a design-level earthquake. The at-grade library and retail slabs would need to be designed to tolerate some deflection where the slabs transition from on-footing support to ground-only support. Since seismic settlement could theoretically range from approximately 4 to 5 inches, loss of support could occur below the slab on-grade that results in voids beneath the slab and localized cracking at transition areas. If required, these voids could be filled with grout following an earthquake. The following recommendations assume at-grade slabs will be underlain by appropriately spaced ground improvement elements.

9.1 AT-GRADE INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, any at-grade 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. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 PARKING STRUCTURE SLAB-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. The garage slab should also be supported on at least 6 inches of select fill consisting of one of the following placed and compacted in accordance with the “Compaction” section of this report:

- Class 2 aggregate base,
- ¾-inch clean, crushed rock
- recycled AC/AB grindings
- cement-treated soil, consisting of at least 4 percent quicklime or cement by dry weight

Basement level slabs should be water-proofed and designed to resist hydrostatic pressures, as needed. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 15-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
No. 200	0 – 5

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.4 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian 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.

As discussed, there is a potential for differential settlement due liquefaction and sand venting, especially around the perimeter of the building of the at-grade portion of the building. To reduce the potential for differential sidewalk movement relative to the ground improvement supported structure during a significant seismic event, flatwork should be reinforced, include construction and control joints spaced no greater than 6 feet on center, and be dowelled across the building entrances. Alternatively, a row of ground improvement could be extended beyond the building footprint that would provide some support to sidewalk areas during a significant seismic event in addition limiting the effect of increased pore pressure along the building foundation.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgement considering the shallow clay soil conditions blanketing portions of the site.

Table 3: Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	13.5	17.5

[1] Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 5

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and Pedestrian Pavements" section above.

Table 4: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 10	6.0	6.0
Maximum ADTT = 100	6.5	6.0

[1] Subgrade design R-Value = 5

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi. 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.

10.2.1 Stress Pads for Trash Enclosures

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

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

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

Table 5: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	40 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	40 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.

11.2 SEISMIC LATERAL EARTH PRESSURES

11.2.1 Basement Walls

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We checked seismic earth pressures for the proposed restrained and unrestrained (cantilever) retaining walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.8.3 using the Design level earthquake. 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).

Because 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 resultant force 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. We recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

Because the wall is restrained, or will act as a restrained wall, and will be designed for 40 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

11.3 WALL DRAINAGE

11.3.1 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.

11.3.2 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 to vertical basement wall drainage panels, the drainage path must be maintained.

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.

11.3 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.

11.4 FOUNDATIONS

In general, conventional at-grade site retaining walls may be supported on a continuous conventional footing. Strip footings should bear on natural, undisturbed soil or entirely on engineered fill, and extend at least 18 inches below the lowest adjacent grade. Basement walls should be supported on perimeter foundations underlain by ground improvement, as discussed in the “Foundations” section.

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

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of For the Future Housing, Inc. specifically to support the design of the Downtown Library Residential 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 groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

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

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

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

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

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

SECTION 13: REFERENCES

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

ATC Hazards by Location, Hazards by Location, 2020, https://hazards.atcouncil.org/

American Concrete Pavement Association (ACPA, 2006). *Design of Concrete Pavement for Streets and Roads.*

ASCE 7-16. American Society of Civil Engineers. (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.*

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

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

California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

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

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

Cetin, K.O., Bilge, H.T., Wu, J., Kammerer, A.M., and Seed, R.B., Probablilistic Model for the Assessment of Cyclically Induced Reconsolidation (Volumetric) Settlements, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 135, No. 3, March 1, 2009.

Federal Emergency Management Administration (FEMA), 2009, FIRM City of Santa Cruz, California, Community Panel #06087C0332E.

GitHub, *usgs/shakemap-scenarios*, 2020, https://github.com/usgs/shakemap-scenarios/tree/master/rupture_sets/BSSC2014

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

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

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

Lew, M. et al, 2010, Seismic Earth Pressures on Deep Building Basements, Proceedings, SEAOC Convention, Indian Wells, CA.

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

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

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

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

State of California Department of Transportation, Highway Design Manual, Latest Edition.

U.S. Geological Survey, *Unified Hazard Tool*, 2020, https://earthquake.usgs.gov/hazards/interactive/

U.S. Geological Survey, *Building Seismic Safety Council 2014 Event Set*, ArcGIS Web Application, 2020, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=14d2f75c7c4f4619936dac0d14e1e468

U.S. Geological Survey, *USGS Earthquake Hazards Program*, 2020, https://earthquake.usgs.gov/nshmp-haz-ws/apps/spectra-plot.html

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

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228)*. KMZ files available at:* www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

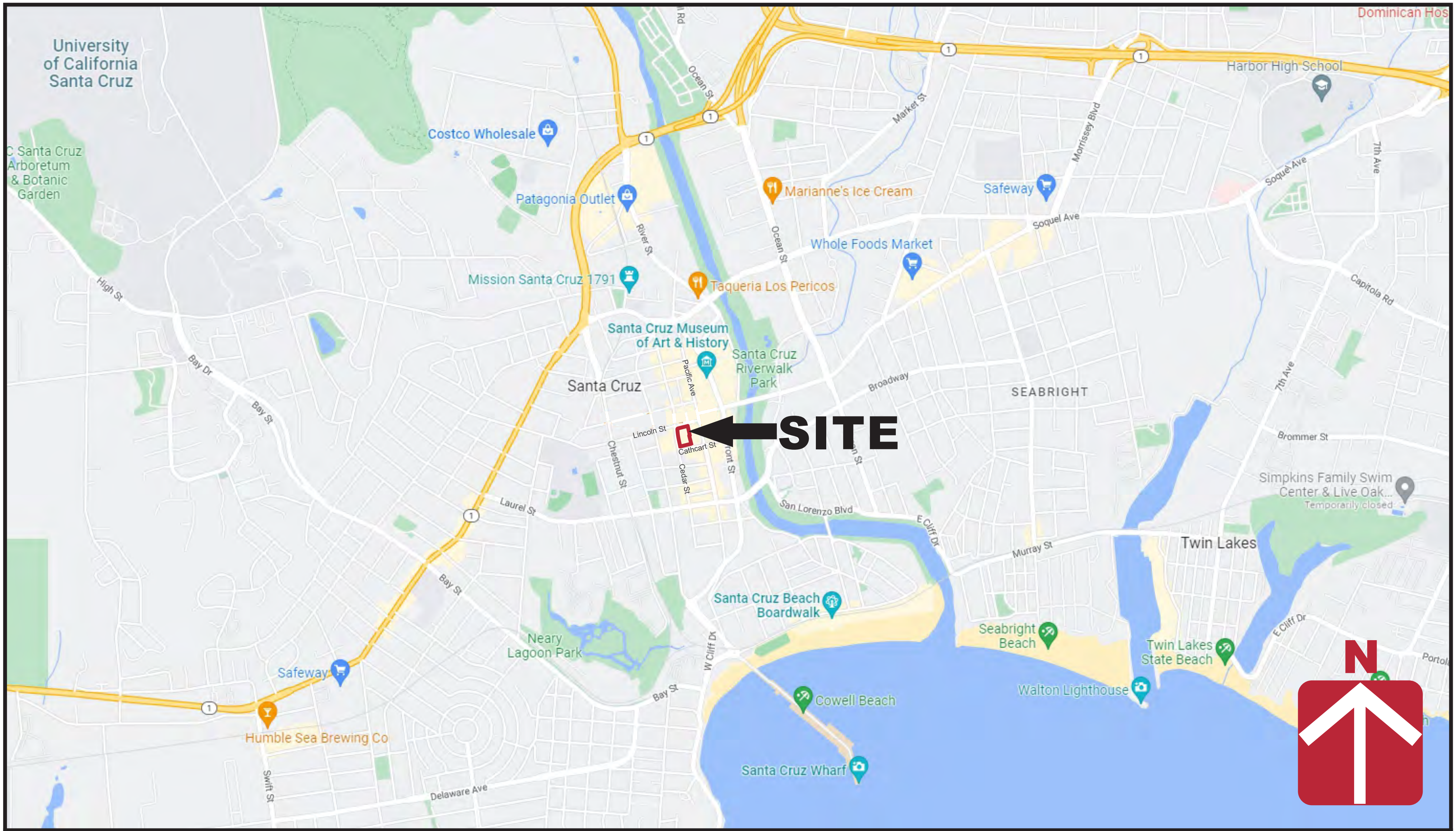
Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

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

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

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

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

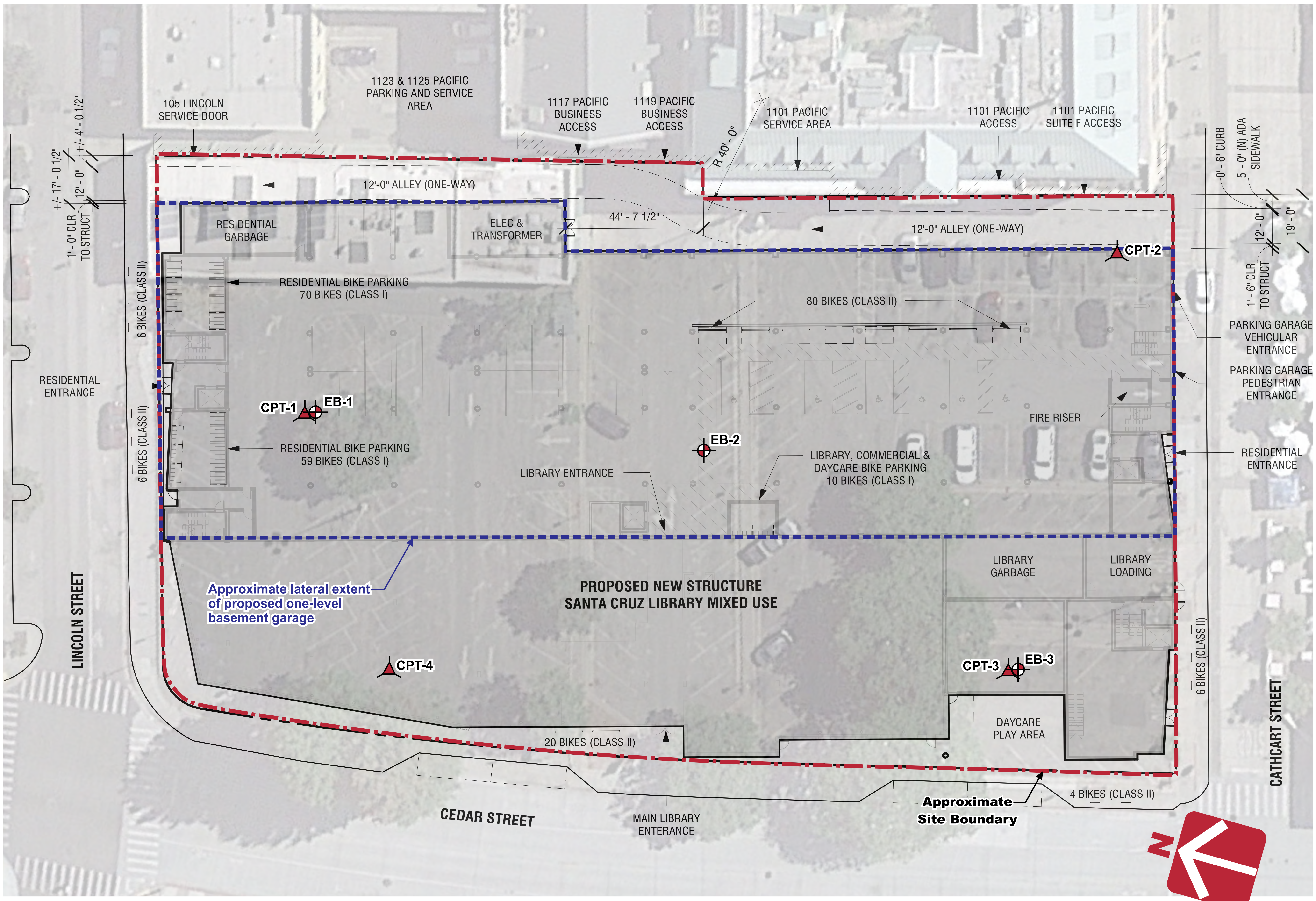


CORNERSTONE EARTH GROUP

Vicinity Map

Downtown Library Mixed-Use
600 Cedar Street
Santa Cruz, CA

Project Number	1271-2-1
Figure Number	Figure 1
Date	June 2022
Drawn By	RRN



Legend

⊕ Approximate location of exploratory boring (EB)

▲ Approximate location of cone penetration test (CPT)

0 30 60

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 09/26/2020
Overlay by Tenover, Site Plan - A1.2, dated 04/14/2022

Project Number
1271-2-1

Figure Number
Figure 2

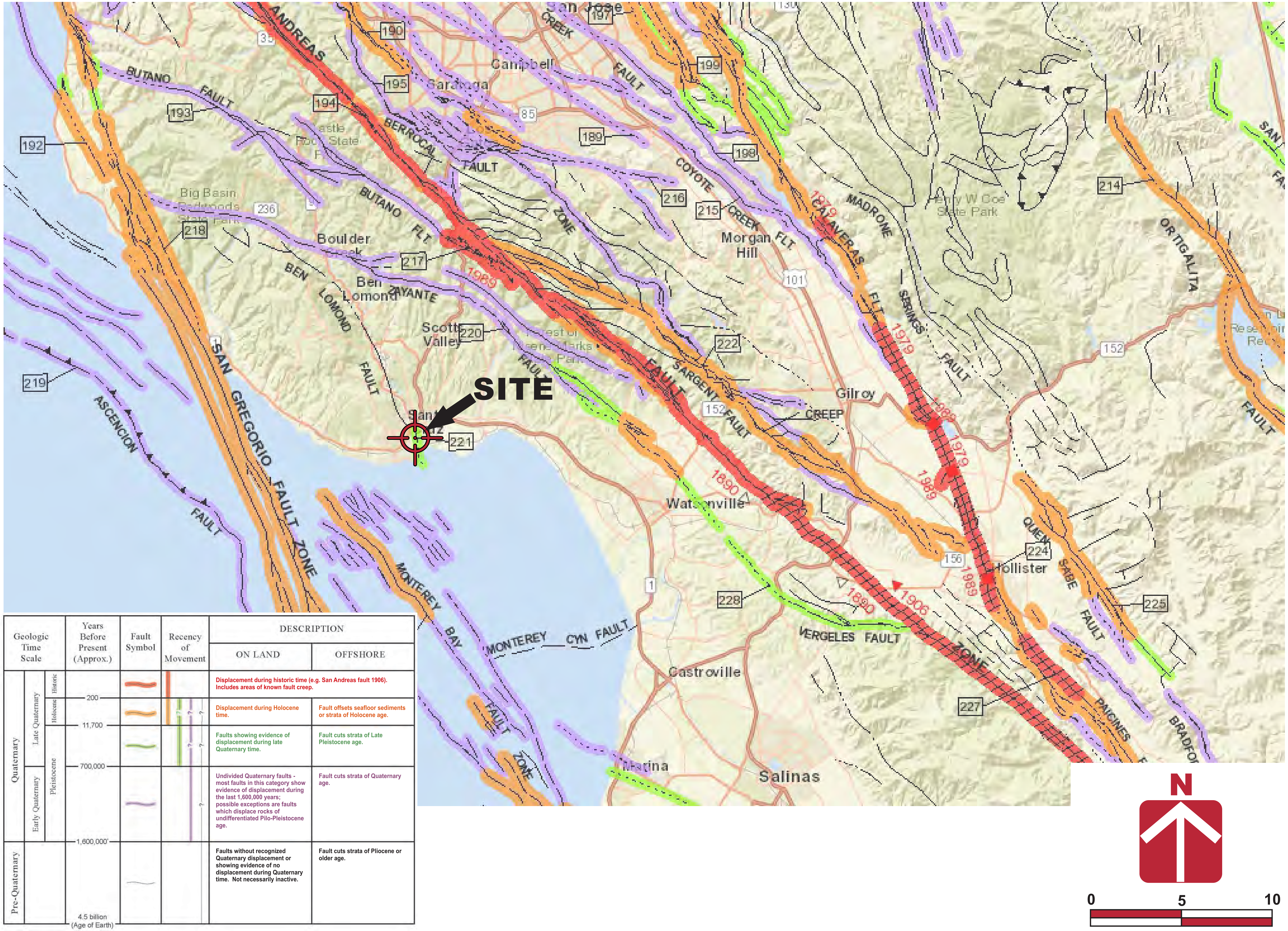
Drawn By
RRN

Date
June 2022

Site Plan

Downtown Library Mixed-Use
600 Cedar Street
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	200	~		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700	~	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000	~	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000	~	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			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)

Project Number
1271-2-1

Figure Number
Figure 3

Date
June 2022

Drawn By
RRN

Regional Fault Map

Downtown Library Mixed-Use
600 Cedar Street
Santa Cruz, CA

CORNERSTONE
EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	**Downtown Library Res Mixed-Use**
Project No.	**1271-2-1**
Project Manager	**SCO**

SEISMIC PARAMETERS

Controlling Fault	**Monterey Bay-Tularcitos**	
Earthquake Magnitude (Mw)	**7.26**	
PGA (Amax)	**0.637**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**11**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.10 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

6.45 **(Inches)**

TOTAL SEISMIC SETTLEMENT 6.6 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 3.13 **L/H** 90.0

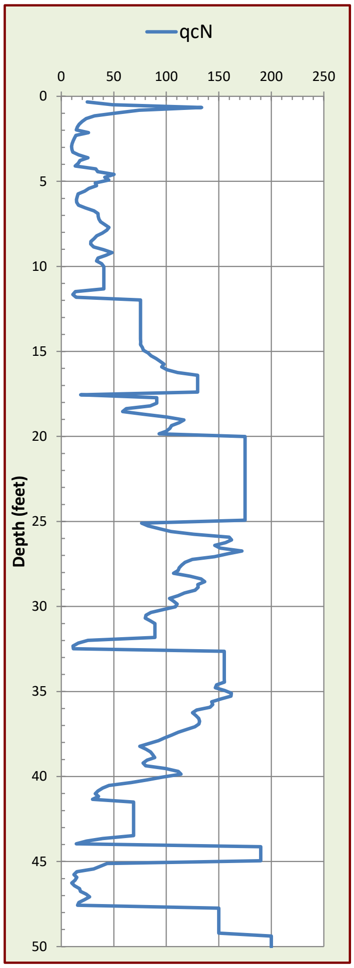
LDI[1]**Corrected for Distance** 0.51 (4 < L/H < 40)

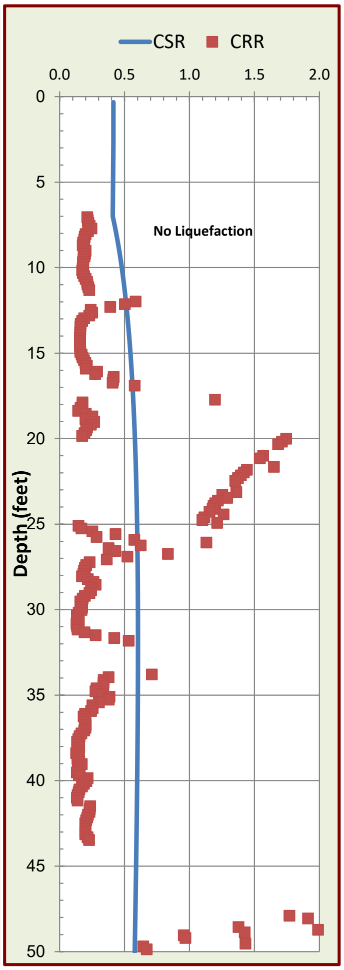
EXPECTED RANGE OF DISPLACEMENT

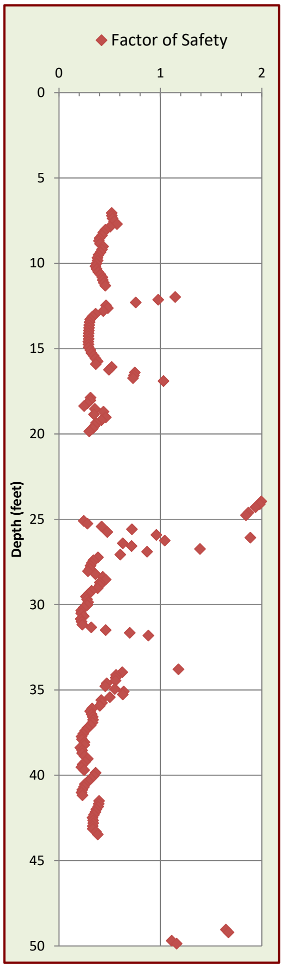
0.3 **to** **1.0** **feet**

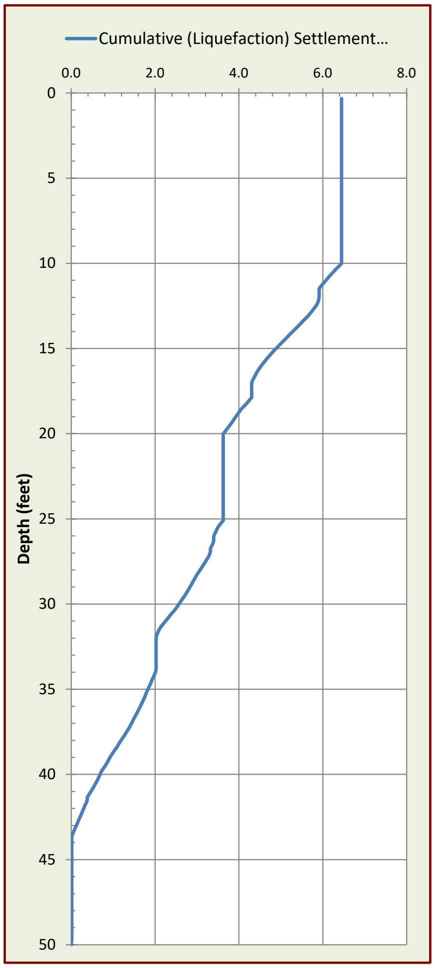
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title: **Downtown Library Res Mixed-Use**
Project No.: **1271-2-1**
Project Manager: **SCO**

SEISMIC PARAMETERS

Controlling Fault: **Monterey Bay-Tularcitos**
Earthquake Magnitude (Mw): **7.26**
PGA (Amax): **0.637** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **11**
Design Water Depth (feet): **7**
Ave. Unit Weight Above GW (pcf): **120**
Ave. Unit Weight Below GW (pcf): **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**
0.17 **(Inches)**
LIQUEFACTION SETTLEMENT FROM 50 **FEET**
6.86 **(Inches)**

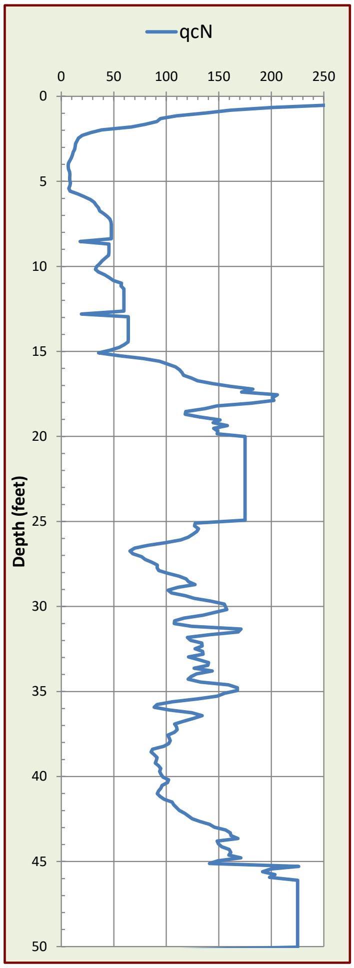
TOTAL SEISMIC SETTLEMENT 7.0 **INCHES**

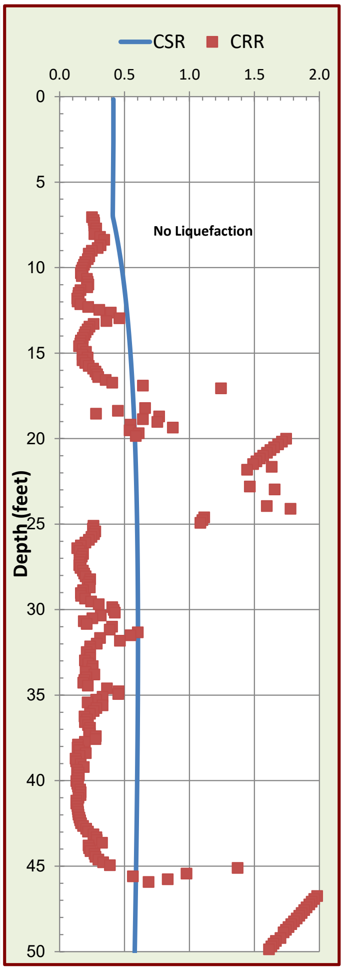
POTENTIAL LATERAL DISPLACEMENT

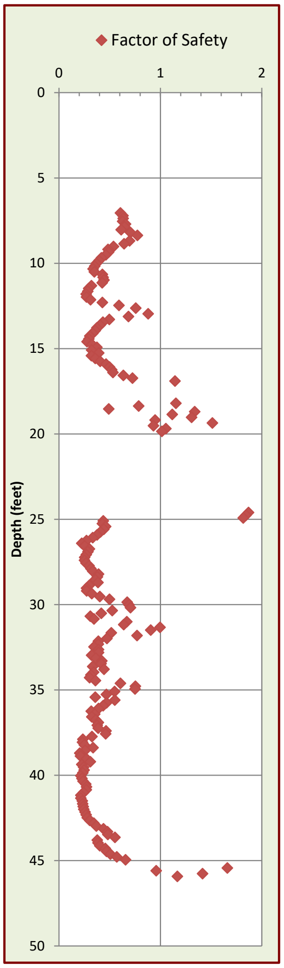
LDI² 2.38 **L/H** 90.0
LDI¹Corrected for Distance 0.39 (4 < L/H < 40)

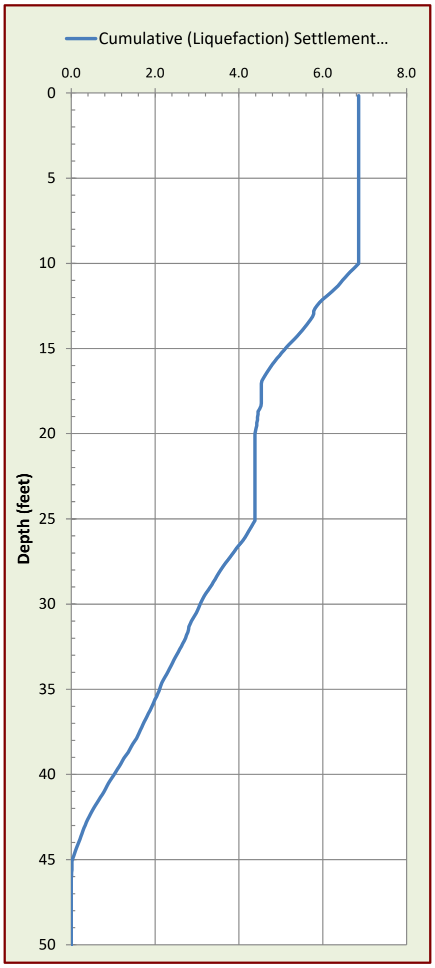
EXPECTED RANGE OF DISPLACEMENT
0.2 to 0.8 feet

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title **Downtown Library Res Mixed-Use**

Project No. **1271-2-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **Monterey Bay-Tularcitos**

Earthquake Magnitude (Mw) **7.26**

PGA (Amax) **0.637** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **9.3**

Design Water Depth (feet) **7**

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

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

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.22 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

6.17 **(Inches)**

TOTAL SEISMIC SETTLEMENT 6.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 2.82 **L/H** 104.4

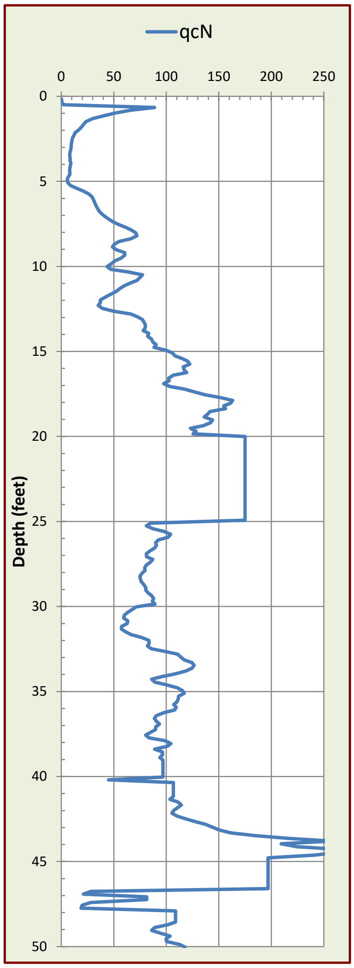
LDI[1]**Corrected for Distance** 0.41 (4 < L/H < 40)

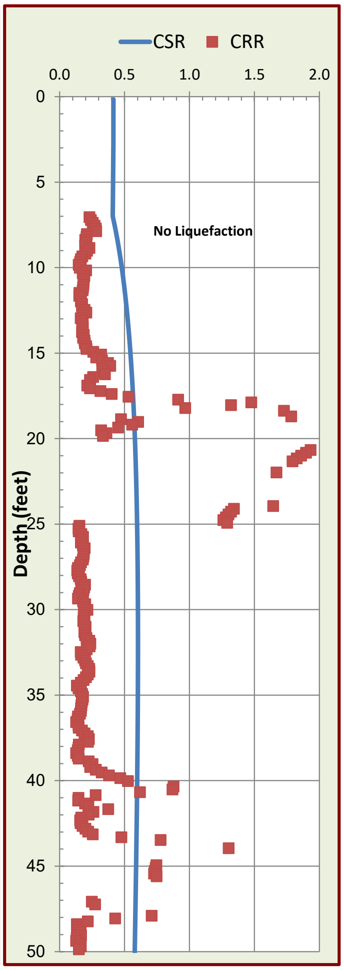
EXPECTED RANGE OF DISPLACEMENT

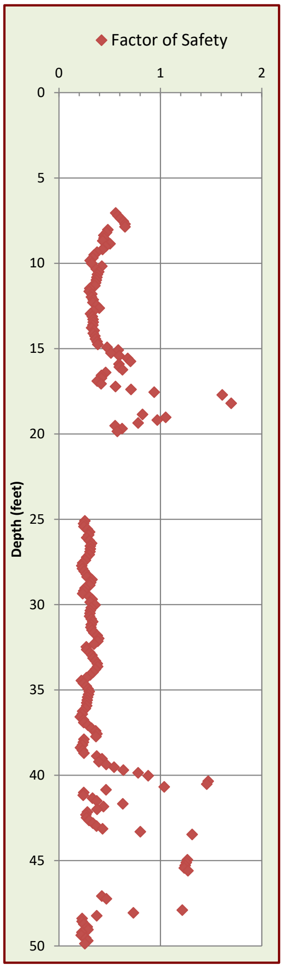
0.2 **to** **0.8** **feet**

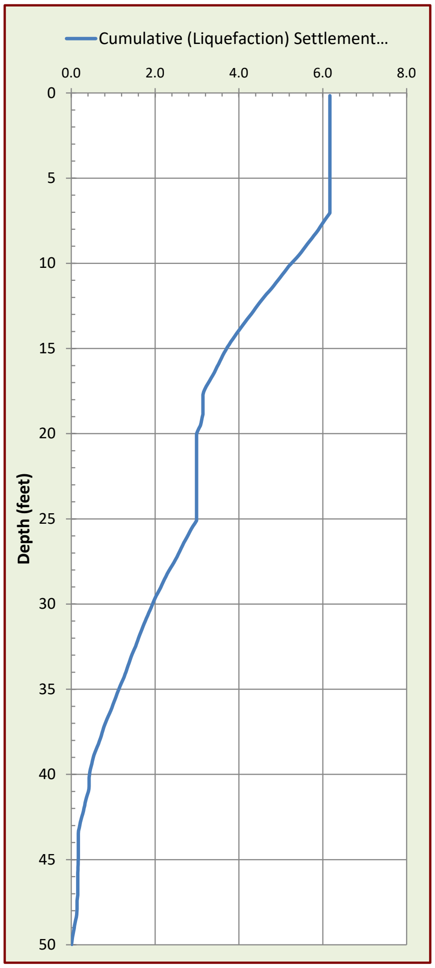
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	**Downtown Library Res Mixed-Use**
Project No.	**1271-2-1**
Project Manager	**SCO**

SEISMIC PARAMETERS

Controlling Fault	**Monterey Bay-Tularcitos**	
Earthquake Magnitude (Mw)	**7.26**	
PGA (Amax)	**0.637**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.3**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.13 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

6.06 **(Inches)**

TOTAL SEISMIC SETTLEMENT 6.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 4.13 **L/H** 107.8

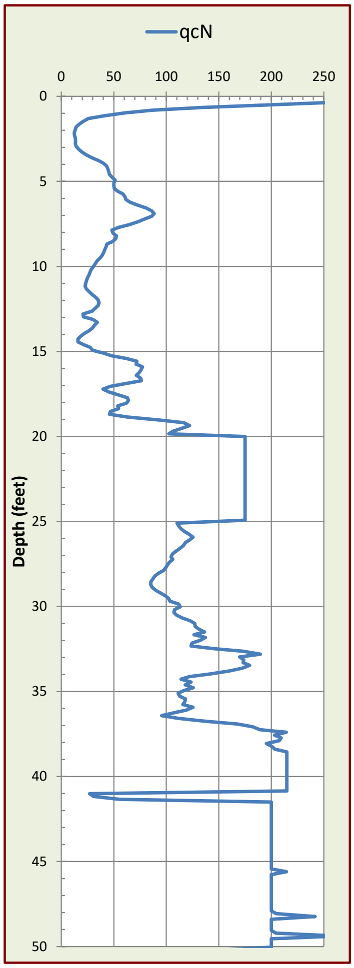
LDI[1]**Corrected for Distance** 0.59 (4 < L/H < 40)

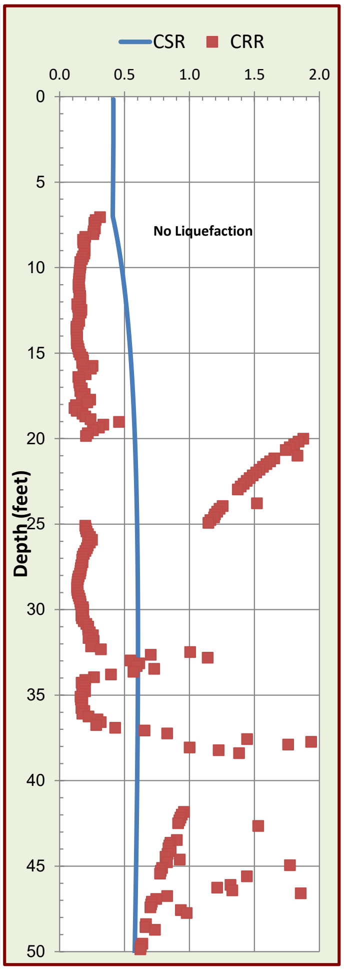
EXPECTED RANGE OF DISPLACEMENT

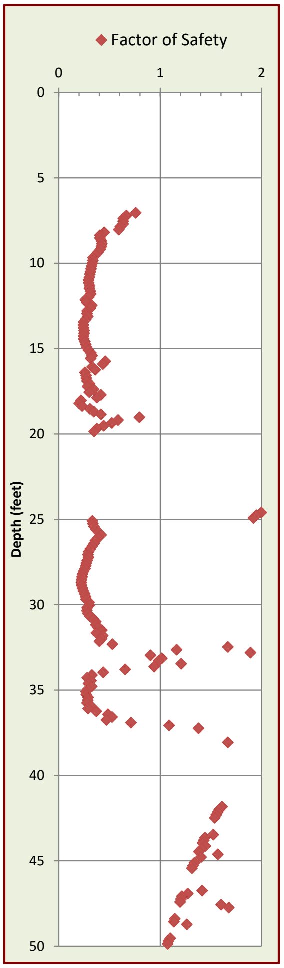
0.3 to 1.2 feet

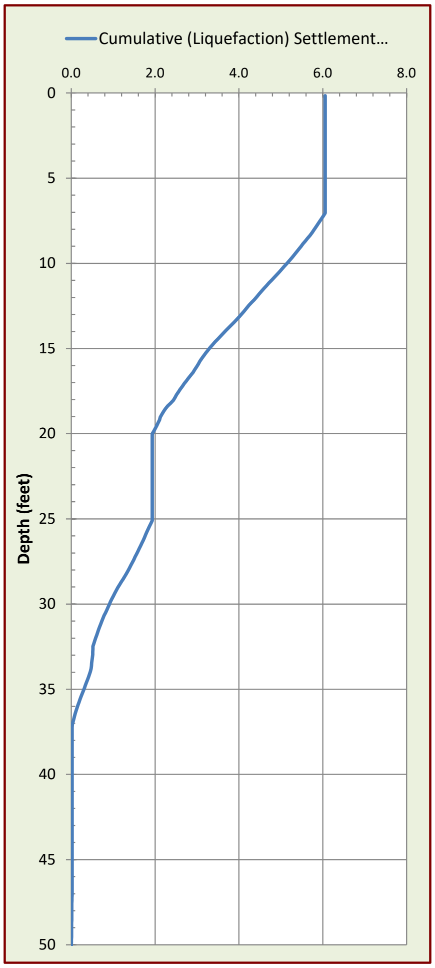
[1] Not Valid for L/H Values < 4 and > 40.

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









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Three 8-inch-diameter exploratory borings were drilled on April 18 and 19, 2022, to depths of 60 to 80 feet. Four CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on April 11, 2022, to depths ranging from 50 to 80 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil , are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand-held GPS unit, 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 are included as part of this appendix.

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

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

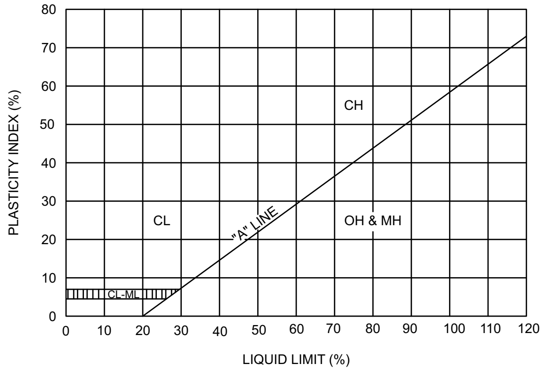
SAMPLER TYPES

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

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PROJECT NAME 600 Cedar Street
PROJECT NUMBER 1271-2-1
PROJECT LOCATION Santa Cruz, CA
DATE STARTED 4/15/22 **DATE COMPLETED** 4/15/22
GROUND ELEVATION **BORING DEPTH** 59.8 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.9723520° **LONGITUDE** -122.0265925°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
∇ **AT TIME OF DRILLING** 11 ft.
NOTES
▼ **AT END OF DRILLING** 11 ft.

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

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

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 1.0 2.0 3.0 4.0
	0	5 inches asphalt concrete over 5 inches aggregate base							
		Silty Sand (SM) [Fill] loose, moist, brown, fine to medium sand, some fine subangular gravel	3	MC-1B	86	9			
		abundant brick fragments	8	MC-2B	87	15		43	
	5								
		Silty Sand (SM) loose, moist, brown, fine to medium sand	13	MC-3B	92	10			
		Poorly Graded Sand with Silt (SP) loose, moist, gray and brown, fine to coarse sand	11	MC-4B	92	6			
	10								
▼		becomes wet							
		Poorly Graded Sand (SP) loose, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	13	MC-5B	112	12			
	15	becomes medium dense	15	SPT					
		Poorly Graded Sand with Silt (SP-SM) dense to very dense, moist, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	50/3"	SPT-7		20			
			43	SPT					
	20								
			35	SPT-9		14		5	
			35	SPT					
	25		50/6"	SPT-11		14			

Continued Next Page

BORING NUMBER EB-1

PROJECT NAME 600 Cedar Street

PROJECT NUMBER 1271-2-1

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) dense to very dense, moist, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	38	SPT					
			medium dense at 28 feet	19	SPT-13		13			
	30									
				66	SPT-14		13			
	35									
				36	SPT-15		17			
	40									
				79	SPT					
	45									
				43	SPT-17		15			
	50									
				53	SPT					
	55									

Continued Next Page

BORING NUMBER EB-1

PAGE 3 OF 3

PROJECT NAME 600 Cedar Street
PROJECT NUMBER 1271-2-1
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)	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) dense to very dense, moist, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	50 3"	☒ SPT-19		15			
	60		Bottom of Boring at 59.8 feet.							
	65									
	70									
	75									
	80									
	85									

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

BORING NUMBER EB-2

PAGE 1 OF 3

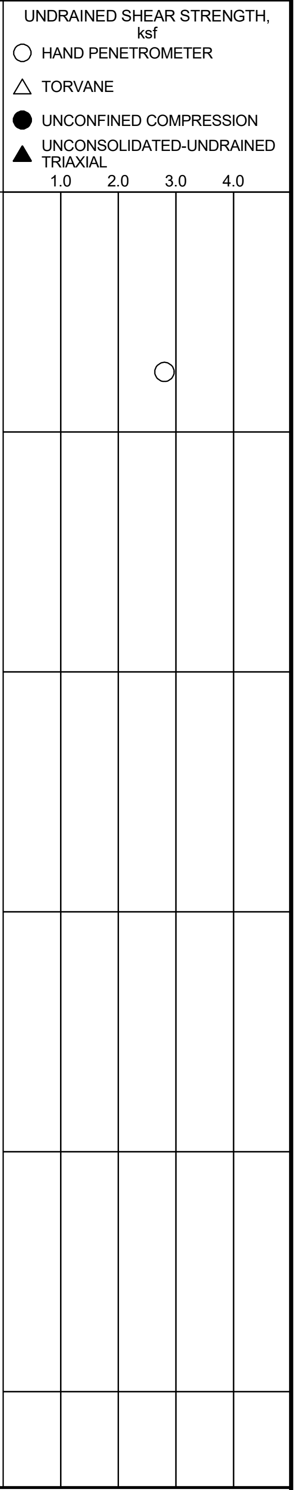
CORNERSTONE EARTH GROUP

PROJECT NAME 600 Cedar Street
PROJECT NUMBER 1271-2-1
PROJECT LOCATION Santa Cruz, CA
DATE STARTED 4/19/22 **DATE COMPLETED** 4/19/22
GROUND ELEVATION **BORING DEPTH** 80 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.9720089° **LONGITUDE** -122.0265345°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
▽ **AT TIME OF DRILLING** 11 ft.
NOTES
▼ **AT END OF DRILLING** 11 ft.

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	2 inches asphalt concrete over 8 inches aggregate base						
		Silty Sand (SM) [Fill] loose, moist, brown, fine sand	15	MC-1B	79	19		
		Lean Clay with Sand (CL) very stiff, moist, brown, fine sand, low plasticity	7	MC-2B	87	21		
	5		15	MC-3B	96	18		
		Silty Sand (SM) loose, moist, brown, fine sand						
		Poorly Graded Sand with Silt (SP-SM) medium dense, moist, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	22	MC				
	10							
▼		becomes wet						
			11	SPT-5		25		7
	15		16	SPT				
		Poorly Graded Sand (SP) dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel	38	SPT-7		12		2
	20							
		becomes medium dense	23	SPT-8		15		
	25							

Continued Next Page



BORING NUMBER EB-2

PAGE 2 OF 3

CORNERSTONE EARTH GROUP

PROJECT NAME 600 Cedar Street

PROJECT NUMBER 1271-2-1

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 to dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	22	SPT					
				44	SPT-10		14			
	30									
			Silty Sand (SM) very dense to dense, moist, gray and brown, fine to coarse sand							
				62	SPT-11		20			
	35									
				35	SPT-12		22			
	40									
			Poorly Graded Sand with Silt (SP-SM) dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel							
				40	SPT					
	45									
			Silty Sand (SM) medium dense to dense, moist, gray and brown, fine to coarse sand							
				18	SPT-14		26			
	50									
				38	SPT					
				27	SPT-16		19			
	55									

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 3 OF 3

PROJECT NAME 600 Cedar Street

PROJECT NUMBER 1271-2-1

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
		Poorly Graded Sand with Silt (SP-SM) very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel						
			50/3"	SPT-17		15		
	60							
			68	SPT				
	65							
			60	SPT-19		15		
	70							
			50/6"	SPT				
	75							
			75	SPT-21		15		
	80	Bottom of Boring at 80.0 feet.						
	85							

BORING NUMBER EB-3

PAGE 1 OF 3

CORNERSTONE
EARTH GROUP

PROJECT NAME 600 Cedar Street
PROJECT NUMBER 1271-2-1
PROJECT LOCATION Santa Cruz, CA
DATE STARTED 4/18/22 **DATE COMPLETED** 4/18/22
GROUND ELEVATION **BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.9716944° **LONGITUDE** -122.0266914°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
∇ **AT TIME OF DRILLING** 9 ft.
NOTES
▼ **AT END OF DRILLING** 9 ft.

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

ELEVATION (ft) | DEPTH (ft) | SYMBOL

DESCRIPTION

Poorly Graded Sand with Silt (SP-SM) [Fill]
loose, moist, brown, fine to medium sand, fine subangular gravel

Lean Clay with Sand (CL)
very stiff, moist, brown, fine sand, low plasticity
Liquid Limit = 33, Plastic Limit = 23

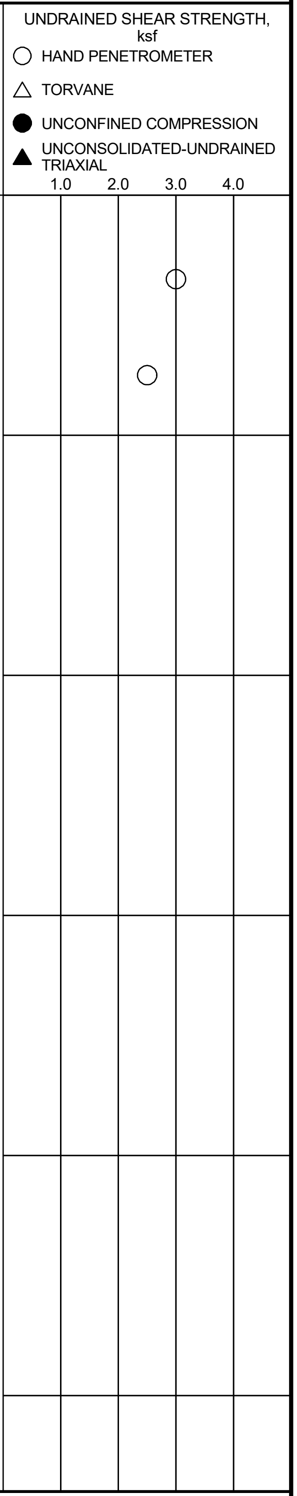
Silty Sand (SM)
loose, moist, brown, fine to medium sand

Poorly Graded Sand with Silt (SP-SM)
loose, moist, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel
becomes wet

becomes medium dense

Poorly Graded Sand (SP)
very dense, wet, gray and brown, fine to coarse sand

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
9	MC-1B	82	20	10	75
5	MC-2B	94	18		
9	MC-3B	90	6		
13	MC-4B	94	23		
17	MC				
20	MC-6B	116	8		
25	MC				
50 6"	SPT-8		13		2
52	SPT-9		18		



Continued Next Page

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/16/22 07:23 - P:\DRAFTING\GINT FILES\1271-2-1 CEDAR AVE.GPJ

BORING NUMBER EB-3

PAGE 2 OF 3

CORNERSTONE EARTH GROUP

PROJECT NAME 600 Cedar Street

PROJECT NUMBER 1271-2-1

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
			Poorly Graded Sand (SP) very dense, wet, gray and brown, fine to coarse sand							
				57	SPT					
	30									
			Sandy Silt (ML) soft, wet, gray, fine sand, low plasticity							
				16	SPT-11		36			○
	35									
			Silty Sand (SM) dense, moist, gray and brown, fine to coarse sand							
				38	SPT-12		23			
	40									
			Poorly Graded Sand with Silt (SP-SM) dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel							
				33	SPT					
	45									
				39	SPT-14		16			
	50									
			becomes very dense	63	SPT					
	55									

Continued Next Page

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME 600 Cedar Street
PROJECT NUMBER 1271-2-1
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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
			Poorly Graded Sand with Silt (SP-SM) very dense, wet, gray and brown, fine to coarse sand, some fine subangular to subrounded gravel	75	☒ SPT-16		17			
	60		Bottom of Boring at 60.0 feet.							
	65									
	70									
	75									
	80									
	85									

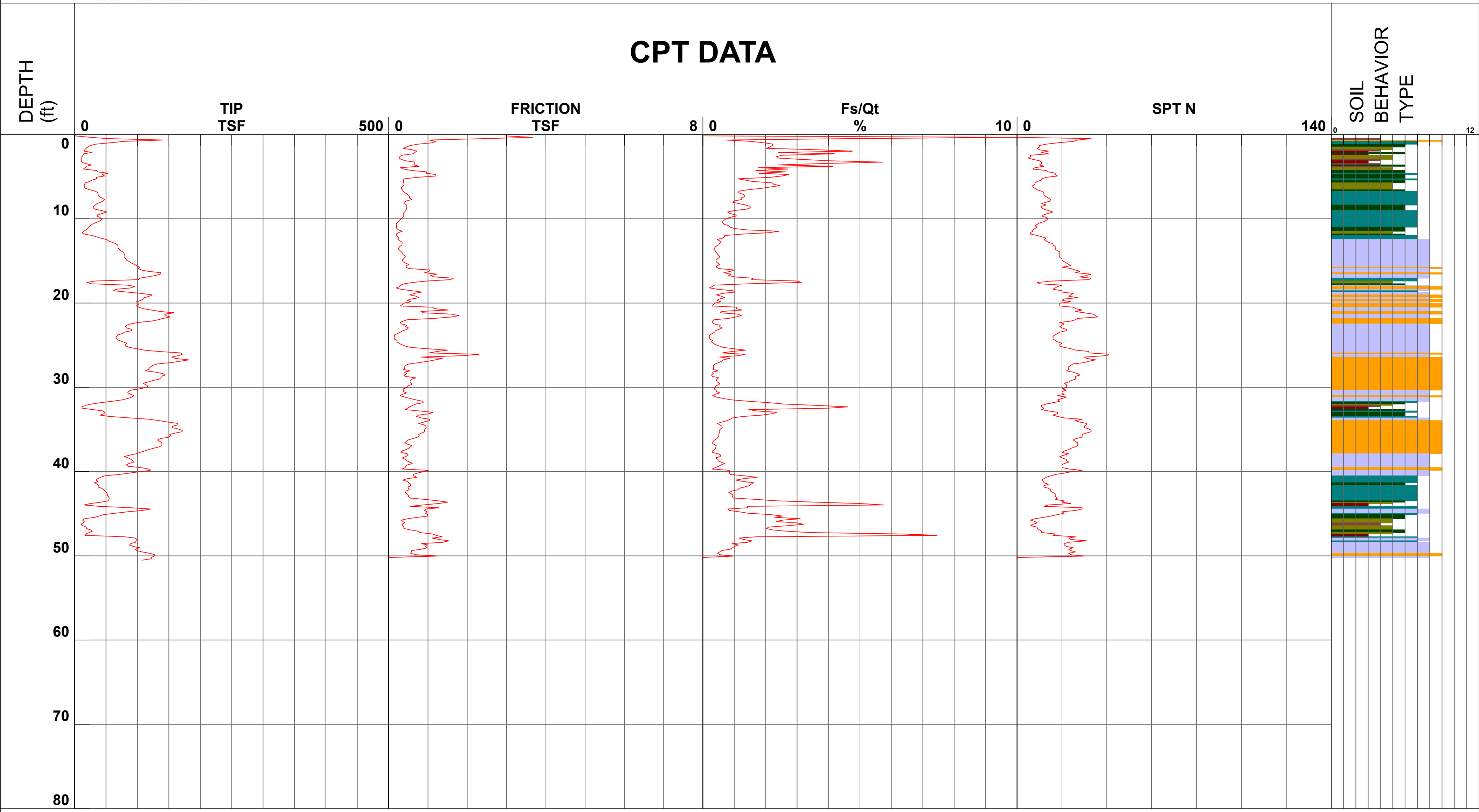
Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

Project 600 Cedar Street
Job Number 1271-2-1
Hole Number CPT-01
EST GW Depth During Test 11.00 ft
Operator AJ-GM-BH
Cone Number DDG1596
Date and Time 4/11/2022 11:47:36 AM
Filename SDF(703).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8

CPT DATA



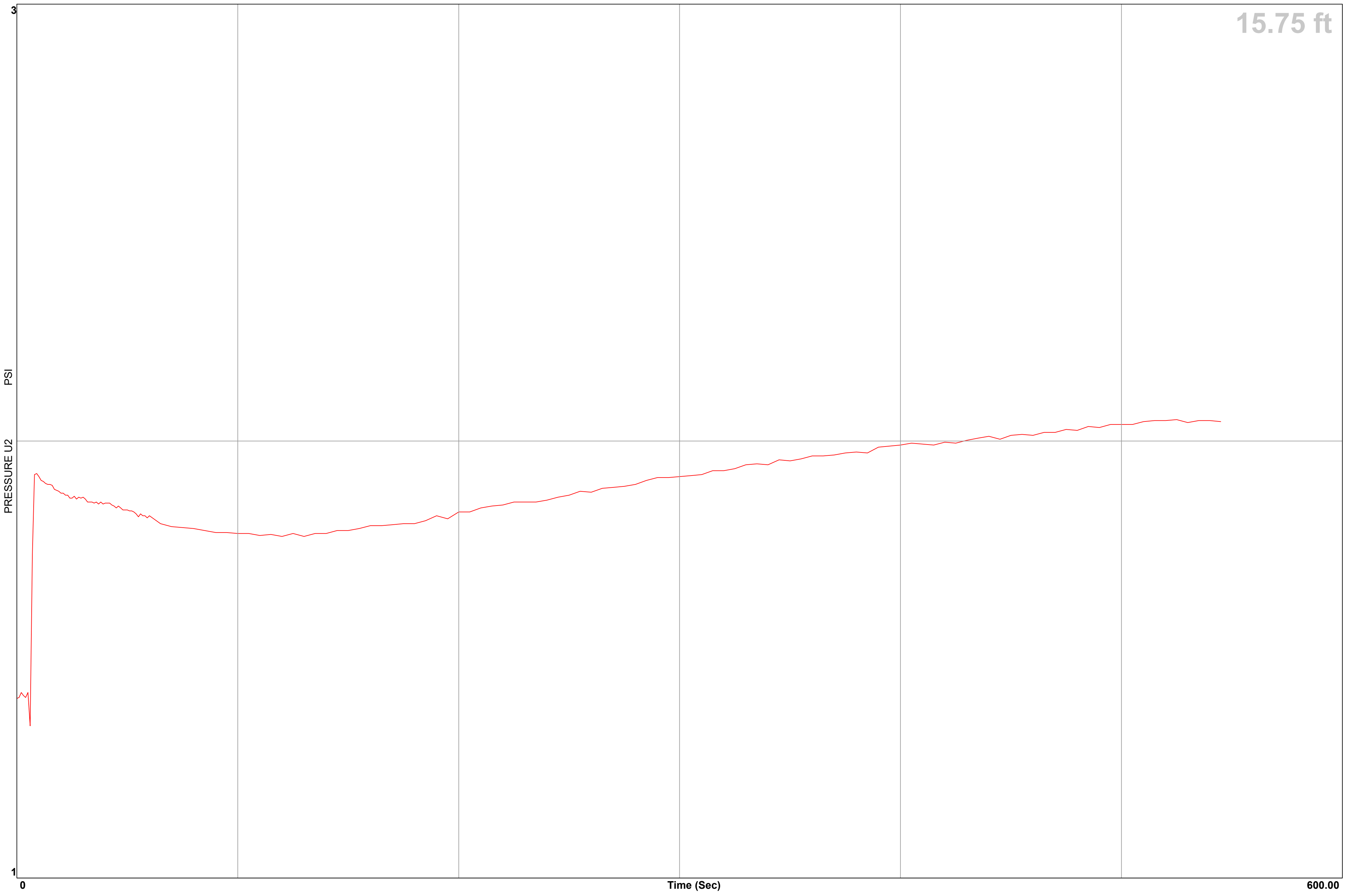
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	600 Cedar Street	**Operator**	AJ-GM-BH		
Job Number	1271-2-1	**Cone Number**	DDG1596	**GPS**	
Hole Number	CPT-01	**Date and Time**	4/11/2022 11:47:36 AM		
Equilized Pressure	2.0	**EST GW Depth During Test**	11.0		



Page 1 of 1

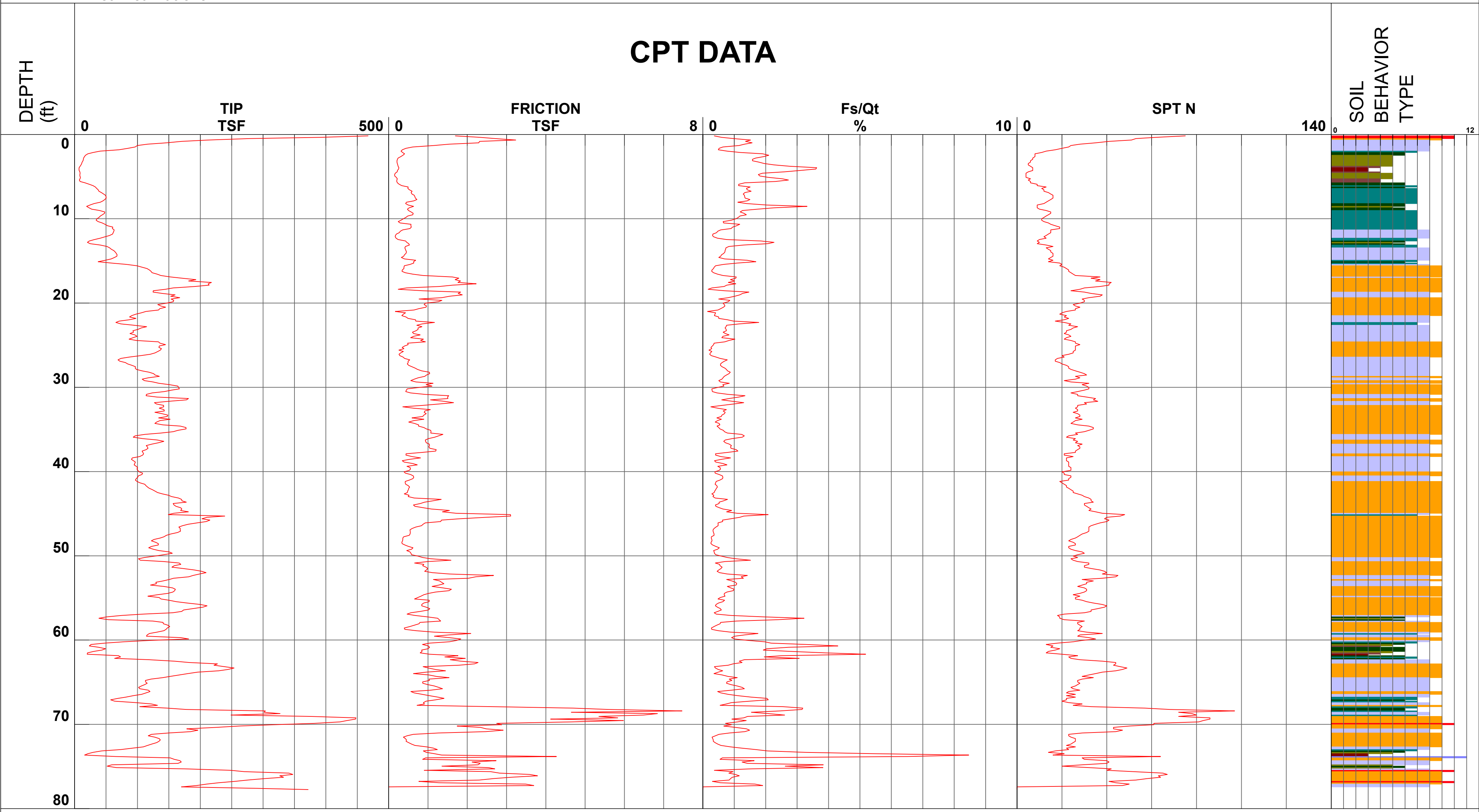
Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project 600 Cedar Street
Job Number 1271-2-1
Hole Number CPT-02
EST GW Depth During Test 10.00 ft
Operator AJ-GM-BH
Cone Number DDG1596
Date and Time 4/11/2022 7:59:09 AM
Filename SDF(700).cpt
GPS
Maximum Depth 77.75 ft

Net Area Ratio .8

CPT DATA



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

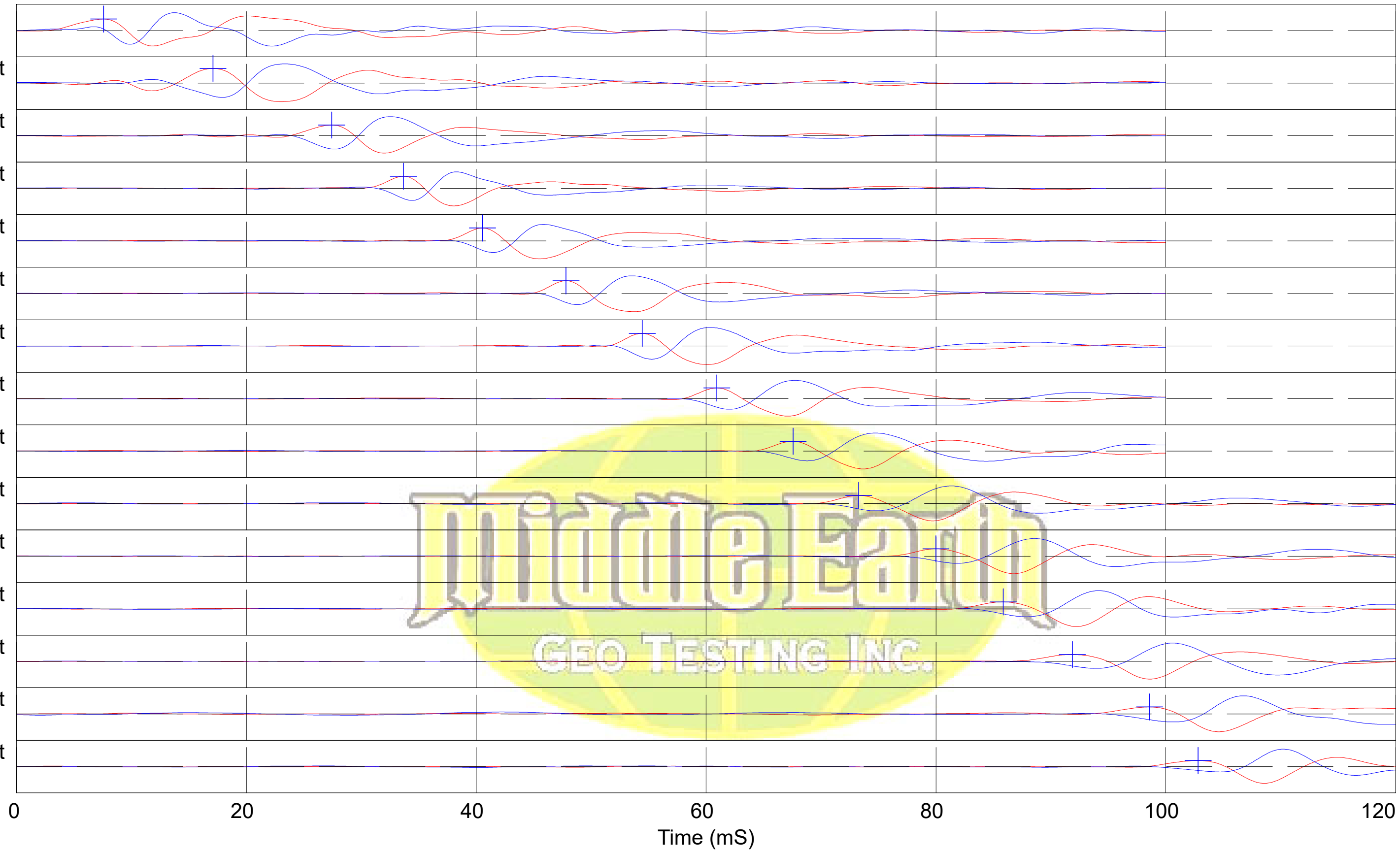
Cone Size 15cm squared

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

CPT-02

Cornerstone Earth Group

600 Cedar Street



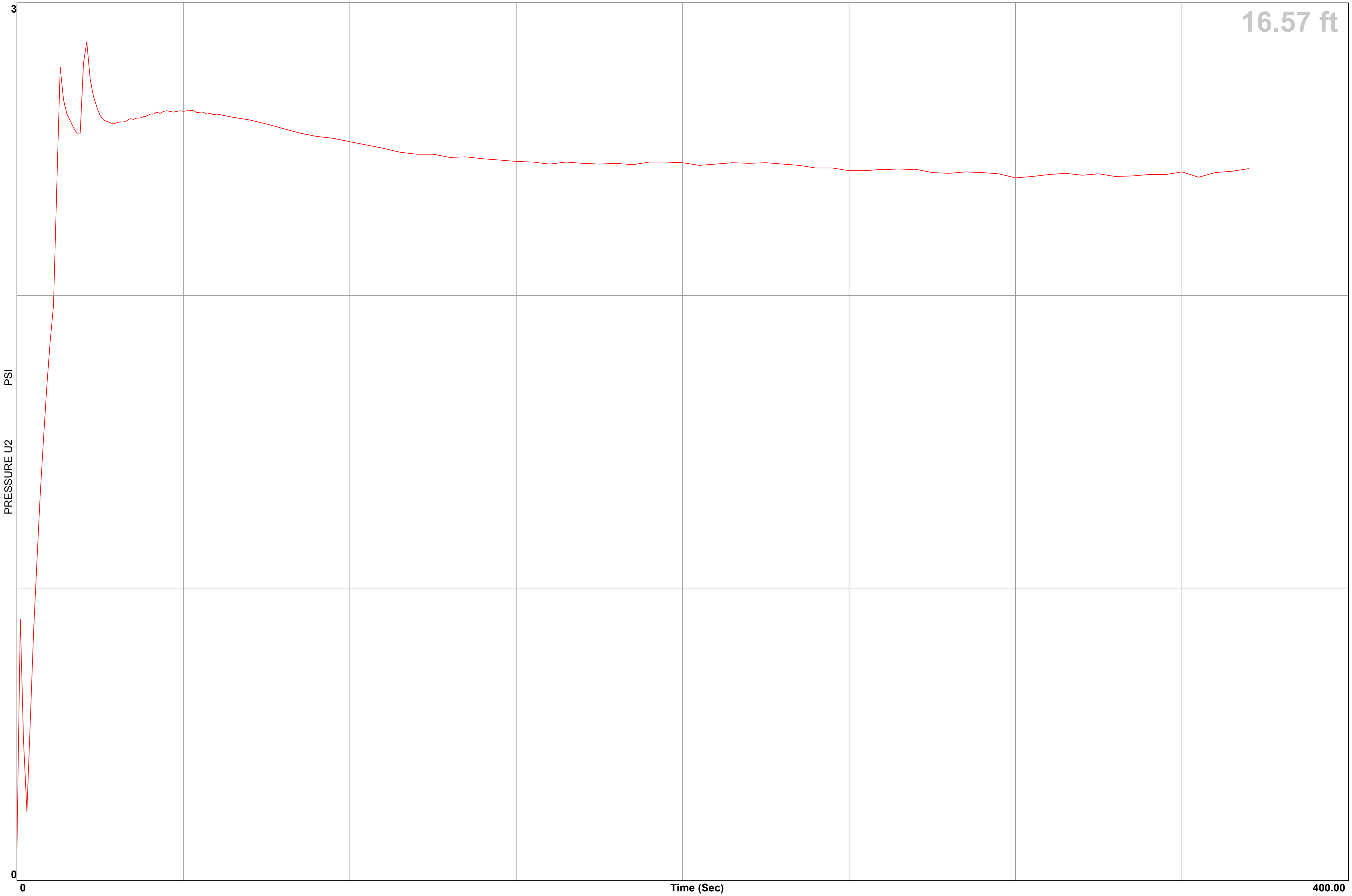
Depth 4.99ft	Ref*	Arrival 7.58mS	Velocity*
Depth 10.01ft	Ref 4.99ft	Arrival 17.11mS	Velocity 410.08ft/S
Depth 15.03ft	Ref 10.01ft	Arrival 27.42mS	Velocity 439.89ft/S
Depth 20.01ft	Ref 15.03ft	Arrival 33.67mS	Velocity 756.39ft/S
Depth 25.03ft	Ref 20.01ft	Arrival 40.54mS	Velocity 706.60ft/S
Depth 30.02ft	Ref 25.03ft	Arrival 47.81mS	Velocity 671.38ft/S
Depth 35.01ft	Ref 30.02ft	Arrival 54.45mS	Velocity 739.14ft/S
Depth 40.03ft	Ref 35.01ft	Arrival 60.93mS	Velocity 764.94ft/S
Depth 45.01ft	Ref 40.03ft	Arrival 67.57mS	Velocity 744.02ft/S
Depth 50.03ft	Ref 45.01ft	Arrival 73.28mS	Velocity 873.65ft/S
Depth 55.02ft	Ref 50.03ft	Arrival 79.99mS	Velocity 737.73ft/S
Depth 60.04ft	Ref 55.02ft	Arrival 85.85mS	Velocity 852.37ft/S
Depth 65.03ft	Ref 60.04ft	Arrival 91.87mS	Velocity 825.45ft/S
Depth 70.05ft	Ref 65.03ft	Arrival 98.59mS	Velocity 744.39ft/S
Depth 75.03ft	Ref 70.05ft	Arrival 102.81mS	Velocity 1178.34ft/S

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

COMMENT:

Cornerstone Earth Group

Location	600 Cedar Street	**Operator**	AJ-GM-BH		
Job Number	1271-2-1	**Cone Number**	DDG1596	**GPS**	
Hole Number	CPT-02	**Date and Time**	4/11/2022 7:59:09 AM		
Equilized Pressure	2.4	**EST GW Depth During Test**	10.9		



Cornerstone Earth Group

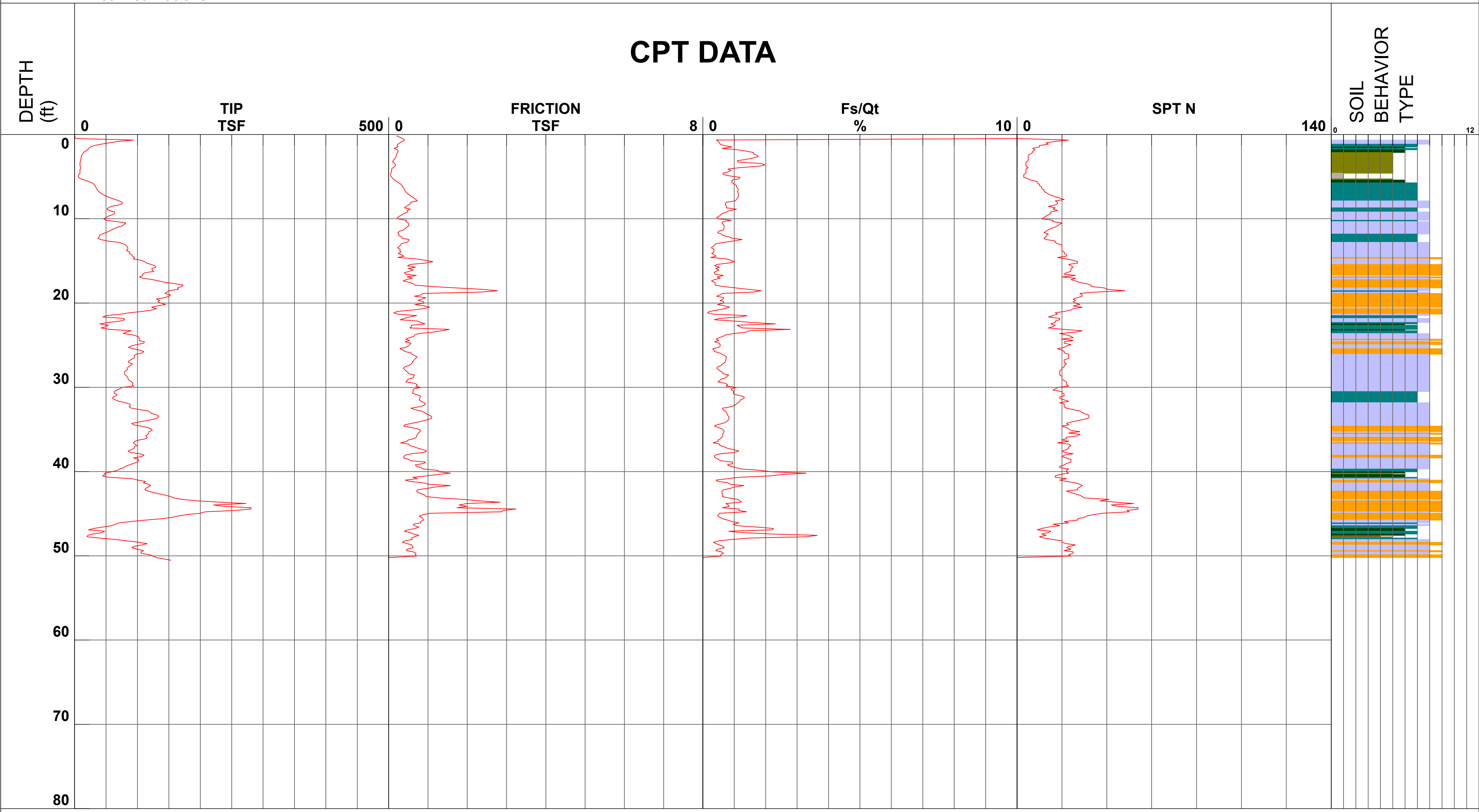
Middle Earth Geo Testing Inc.

Project 600 Cedar Street
Job Number 1271-2-1
Hole Number CPT-03
EST GW Depth During Test 10.00 ft

Operator AJ-GM-BH
Cone Number DDG1596
Date and Time 4/11/2022 1:52:55 PM

Filename SDF(704).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8



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

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

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

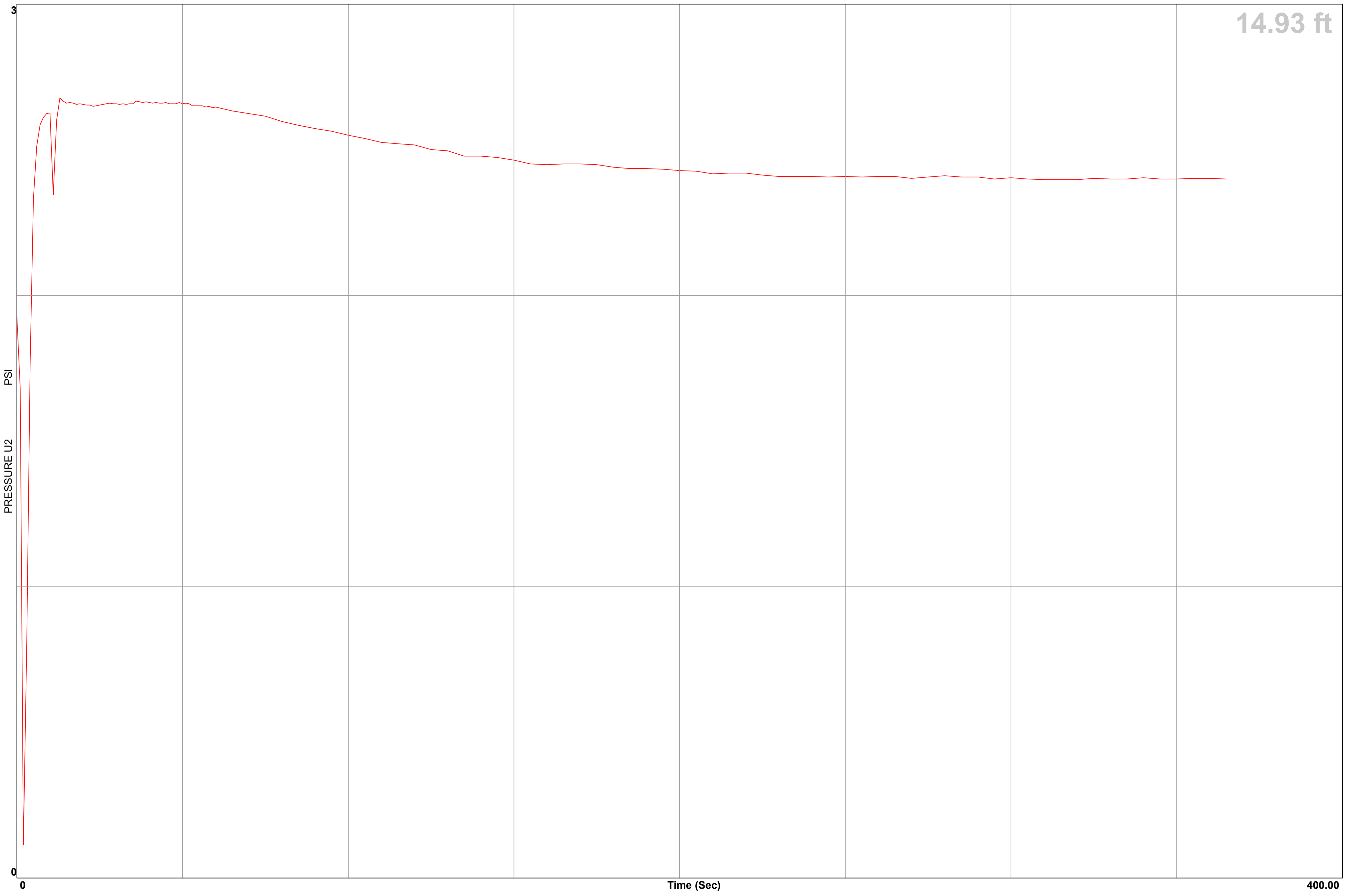
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	600 Cedar Street	Operator	AJ-GM-BH		
Job Number	1271-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-03	Date and Time	4/11/2022 1:52:55 PM		
Equilized Pressure	2.4	EST GW Depth During Test	9.3		



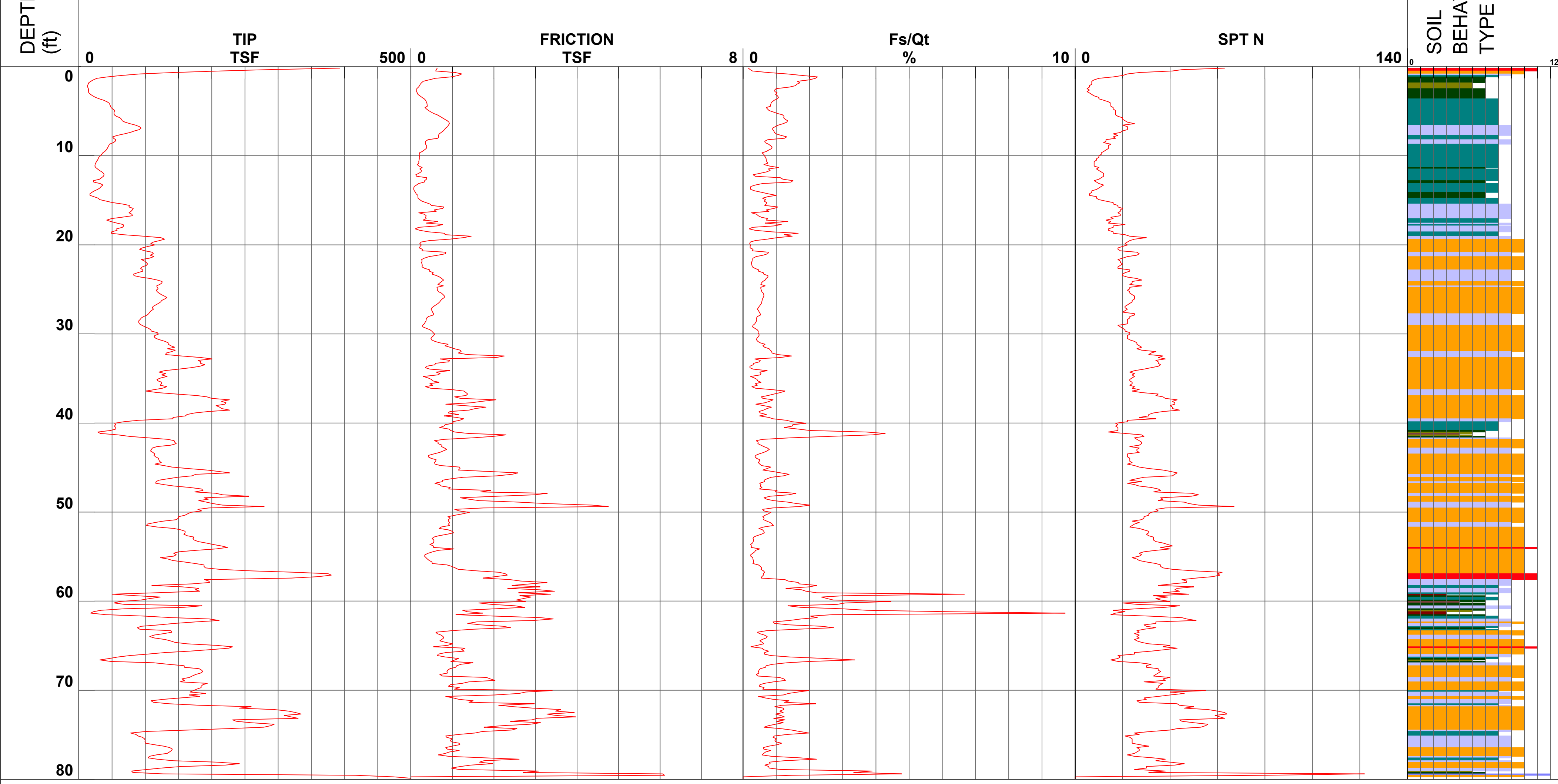
Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

Project 600 Cedar Street
Job Number 1271-2-1
Hole Number CPT-04
EST GW Depth During Test 10.00 ft
Operator AJ-GM-BH
Cone Number DDG1596
Date and Time 4/11/2022 10:10:39 AM
Filename SDF(701).cpt
GPS
Maximum Depth 80.05 ft

Net Area Ratio .8

CPT DATA



◼ 1 - sensitive fine grained
◼ 2 - organic material
🟥 3 - clay

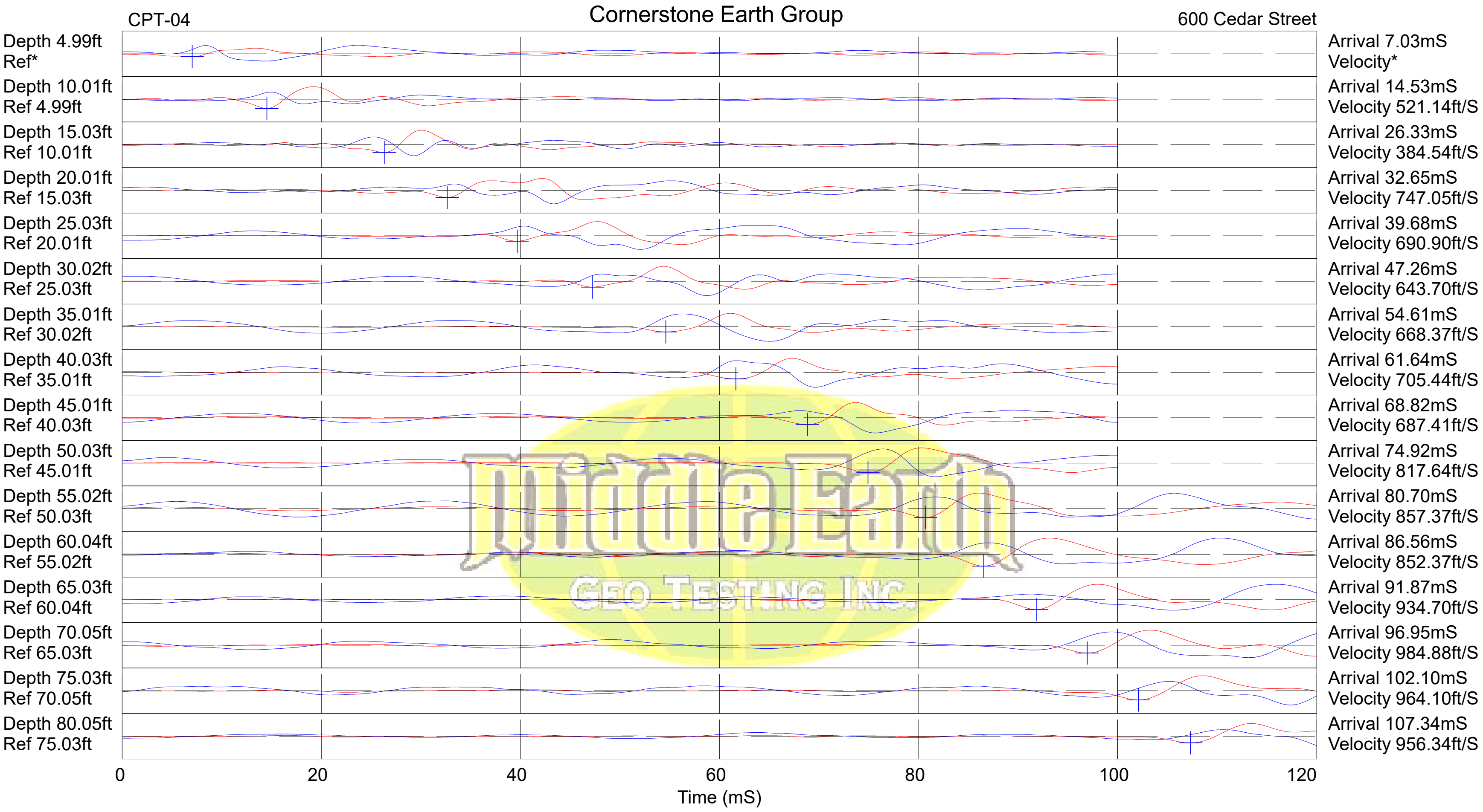
🟫 4 - silty clay to clay
🟫 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt

🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand

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

Cone Size 15cm squared

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

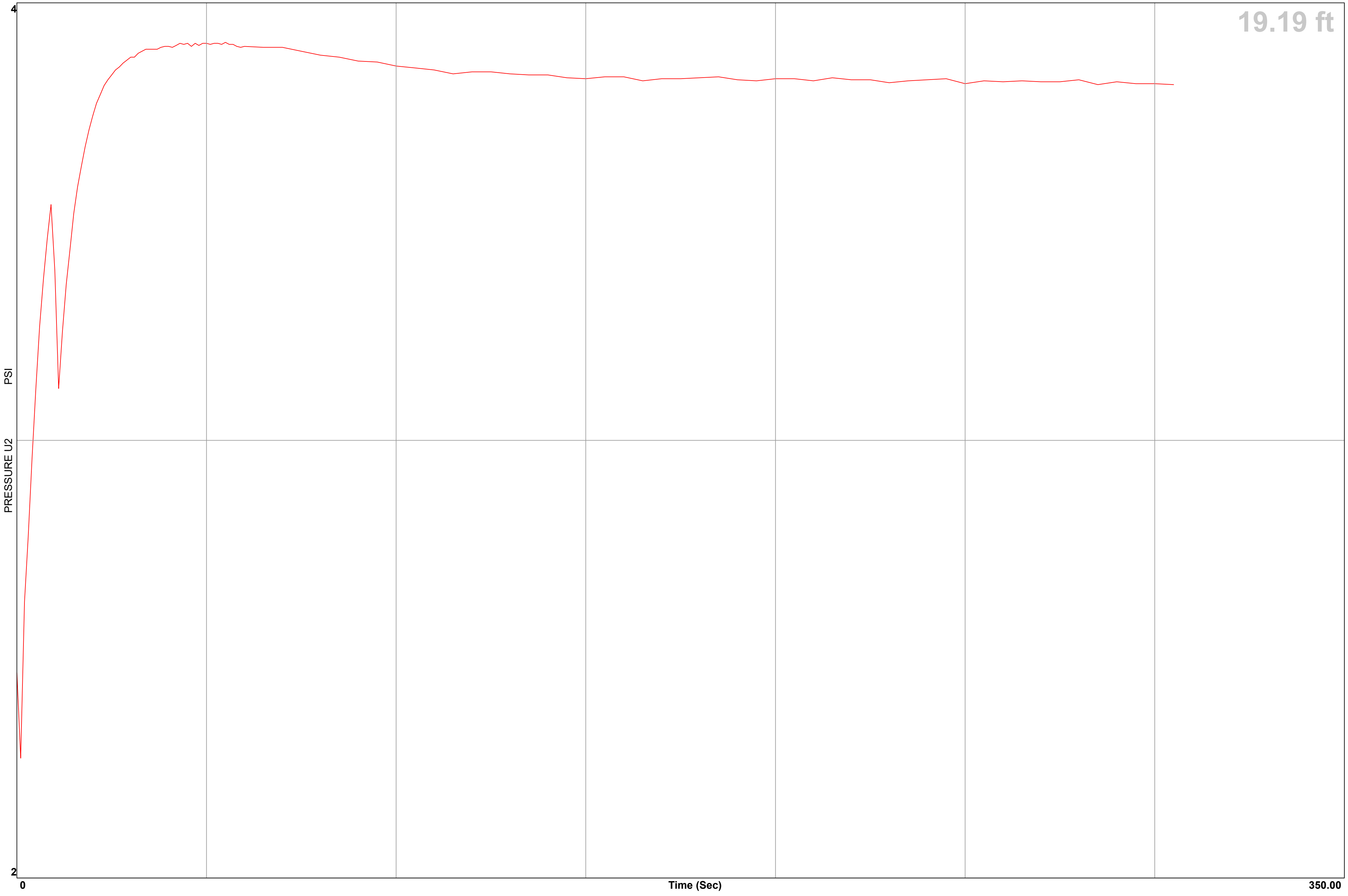


Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Cornerstone Earth Group

Location 600 Cedar Street
Job Number 1271-2-1
Hole Number CPT-04
Equilized Pressure 3.8
Operator AJ-GM-BH
Cone Number DDG1596
Date and Time 4/11/2022 10:10:39 AM
EST GW Depth During Test 10.3
GPS



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.

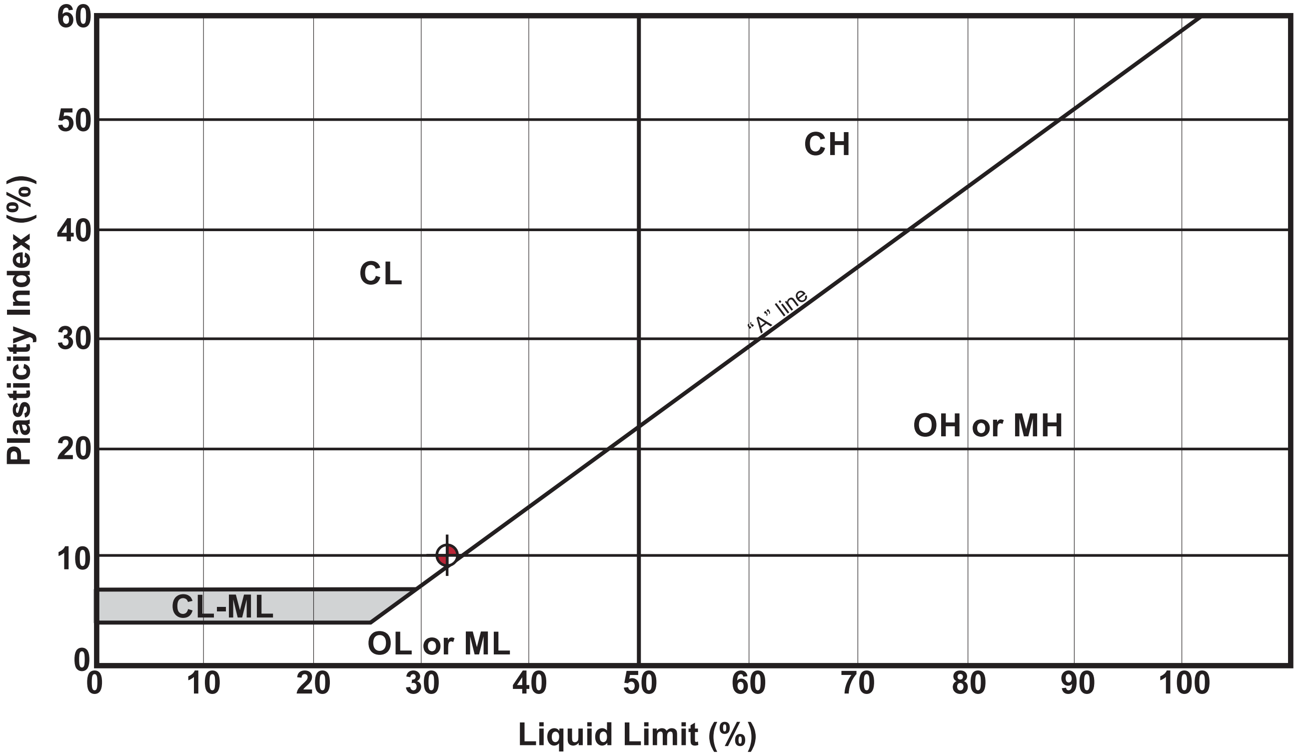
Moisture Content: The natural water content was determined (ASTM D2216) on 38 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 13 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 6 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: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil 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 this test are shown on the boring log at the appropriate sample depth.

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	20	33	23	10	75	Lean Clay with Sand (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Downtown Library Mixed-Use
600 Cedar Street
Santa Cruz, CA

Project Number
1271-2-1

Figure Number
Figure B1

Date
May 2022

Drawn By
FLL