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SUPPLEMENTAL RESPONSES TO CEC STAFF DATA REQUEST SET 1 (6, 24-27, 37-40 AND 45)

Bowers Backup Generating Facility (22-SPPE-01)

SUBMITTED TO: CALIFORNIA ENERGY COMMISSION
SUBMITTED BY: **GI Partners**

March 2023

DAYZEN LLC

INTRODUCTION

Attached are GI Partners Supplemental Responses to California Energy Commission (CEC) Staff Data Request Set No. 1 (6, 24-27, 37-40 and 45) for the Bowers Backup Generating Facility (BBGF) Application for Small Power Plant Exemption (SPPE) (22-SPPE-01).

The Data Responses are grouped by individual discipline or topic area. Within each discipline area, the responses are presented in the same order as Staff presented them and are keyed to the Data Request numbers. Additional tables, figures, or documents submitted in response to a data request (e.g., supporting data, stand-alone documents such as plans, folding graphics, etc.) are found in Attachments at the end of the document and labeled with the Data Request Number for ease of reference.

For context, the text of the Background and Data Request precede each Data Response.

AIR QUALITY AND GREENHOUSE GAS EMISSIONS

BACKGROUND: Particulate Matter Emission Factor

Appendix AQ-1 in Appendix A of the SPPE application shows that the applicant assumed the emission factor for particulate matter (PM) of 10 micrometers or less in diameter (PM10) and particulate matter of 2.5 micrometers and smaller in diameter (PM2.5) to be 0.015 grams per brake horsepower-hour (g/bhp-hr). However, the MIRATECH performance warranty data (sent electronically from applicant to staff) shows that the target outlet PM10 emission factor would be 0.02 g/bhp-hr. Staff needs to confirm the PM emission factor to make sure the PM impacts were not underestimated.

DATA REQUESTS

6. Please confirm which PM emission factor is correct and provide documentation of the correct emission factor.

Supplemental Response To Data Request 6

The specification data relating to the PM 10 emission rate and ammonia slip was provided to Staff electronically.

GEOLOGICAL AND PALEONTOLOGICAL RESOURCES

BACKGROUND: Subsurface Geotechnical Soil Properties

Appendix D of the SPPE application includes a Soil Report generated from the Natural Resources Conservation Service's website. Natural Resources Conservation Soil Reports do not provide sufficient subsurface geotechnical soil properties to determine the potential of site-specific geologic hazards such as the potential for liquefaction, the presence of expansive materials, or the lateral and vertical extent of undocumented fill material at the site. This information is necessary for staff to complete their analysis.

DATA REQUESTS

24. Provide site-specific subsurface geotechnical soil information.

Response To Data Request 24

The preliminary geotechnical report is included in Attachment DR GEO-24.

25. Provide any adverse soil conditions present including, but not limited to, liquefaction potential, the presence of expansive soils, and the presence of existing fills at the site.

Response To Data Request 25

Please see the preliminary geotechnical report included in Attachment DR GEO-24.

26. If such adverse soil conditions are present, provide the maximum depths of disturbance for each of the possible foundation solutions noted in Section 3.7.2.1 (mat slab, soil-mixed columns, and drilled displaced piers).

Response To Data Request 26

Foundations are likely to be drilled displace piers, likely to exceed depths of 80 feet. However, alternative foundation designs could be viable based on the results of future geotechnical investigations and more detailed foundation designs.

BACKGROUND: Potential Fossil Yield Classification Ranking

In the SPPE application, Section 3.7.1.2, Paleontological Resources, the applicant referenced the City of Santa Clara Draft General Plan, dated January 2011, page 328, and noted,

"The site is situated on alluvial fan deposits of the Holocene age. These sediments have low potential to yield fossil resources or to contain significant nonrenewable paleontological resources. However, these recent sediments overlie sediments of older Pleistocene age sediments with high potential to contain paleontological resources. These older sediments, often found at depths of ten feet or more below the ground surface, have yielded the fossil remains of plants and extinct terrestrial Pleistocene vertebrates. Ground disturbing activities of ten feet or more have the potential to impact undiscovered paleontological resources in older Pleistocene sediments."

In addition to the information provided, the potential for paleontological resources to occur in the project area should also be evaluated using the federal Potential Fossil Yield Classification (PFYC) system developed by the Bureau of Land Management (BLM 2016). Because of its demonstrated usefulness as a resource management tool, the PFYC has been utilized for many years for projects across the country, regardless of land ownership. It is a predictive resource management tool that classifies geologic units based on their likelihood to contain paleontological resources on a scale of 1 (very low potential) to 5 (very high potential) or Unknown. This system is intended to aid in predicting, assessing, and mitigating impacts to paleontological resources.

DATA REQUEST

27. Provide the PFYC ranking for the project site.

Response To Data Request 27

GI Partner's consultants attempted to access the BLM PFYC ranking tool including downloading the source ARCgis files and it does not appear to include California properties. GI Partners recommends that since the soil conditions ate the site are similar to the soil conditions for the adjacent CA3DC project, Staff consider the analysis contained in the CA3BGF and CA3DC Final EIR (TN242453 at pages 4.7-13 and 14) for adaption to the BBGF site.

> There are no known paleontological resources within the project site. A search of the University of California Museum of Paleontology database failed to identify any paleontological resources in the vicinity of the site (UCMP 2021). However, ground disturbing activities of ten feet or more have the potential to impact undiscovered paleontological resources. The CA3 Data Center would require excavation trenching of depths of up to 15 feet. Foundations could be augered cast piles or driven piles, likely to exceed depths of 80 feet. However, alternative foundation designs could be viable based on the results of future geotechnical investigations

(DayZenLLC 2021b). Although unlikely, paleontological resources could be encountered during construction of the CA3 Data Center.

The applicant has proposed a measure to reduce impacts to a unique paleontological resource. The measure includes protocols for training, identification of paleontological resources and salvage plan, including treatment and reporting. Staff evaluated this measure in the context of impacts to paleontological resources and considers the measure sufficient to reduce impacts. Staff proposes GEO-1 to address the potential for discovery of paleontological resources during excavation in native materials.

Although the CA3 Data Center site will be graded and any excavation for deep foundations would be completed prior to installation of any of the CA3 Backup Generating Facilities, construction of the CA3 Backup Generating Facilities would include trenching to install the underground cabling for the electrical interconnection between each generator yard and the facilities they serve. This trenching is most likely to occur in previously disturbed soils shallower than 10 feet. It is unlikely that trenching activities will encounter potential paleontological resources. However, any potential impacts from the trenching activities would be reduced to less than significant levels significant with GEO-1.

PROJECT DESCRIPTION

BACKGROUND: Project Interconnection and System Reliability

The SPPE application Section 2.3 indicated that the BDC includes an onsite new substation with three electrical supply lines that would connect to the Silicon Valley Power (SVP) Uranium Substation. Staff requires a complete description of the BDC interconnection to the SVP system to understand how the interconnection would affect the potential operation of the back-up generators.

DATA REQUESTS

Supplemental Responses To Data Requests 37 through 40

GI Partners had requested this information from SVP. SVP responded that it has not performed any detailed design information that would satisfy these requests. Although prior applicants have provided this information when it was available, the CEC does not require this detailed information in order to perform a CEQA analysis of potential environmental impacts of the BBGF and therefore, the Draft EIR should not be delayed as a result of the inability of GI Partners to provide this information.

LAND USE

BACKGROUND: Thermal Plume Analysis

According to the SPPE application, the project would have emergency generators and air-cooled chillers and the project site is located 1.87 miles west of the Norman Y. Mineta San Jose International Airport. Therefore, staff will require the following information in order to complete its evaluation of thermal plumes from the proposed BBGF emergency generators and the BDC building and server chilling units to ensure air traffic safety and analyze any potentially significant impacts from such plumes.

DATA REQUESTS

45. Please perform a thermal plume modeling of the project's emergency generators for the BBGF and provide modeling files with all calculations embedded in.

Response To Data Request 45

The thermal plume analysis was docketed on February 7, 2023 TN 248688.

ATTACHMENT GEO DR-24

Preliminary Geotechnical Investigation dated 02/24/2023
Prepared by Cornerstone Earth Group

CORNERSTONE EARTH GROUP

TYPE OF SERVICES	Preliminary Geotechnical Investigation
PROJECT NAME	2805 Bowers Data Center
LOCATION	2805 Bowers Avenue Santa Clara, California
CLIENT	GI Partners
PROJECT NUMBER	1413-1-1
DATE	February 24, 2023

CORNERSTONE EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**2805 Bowers Data Center**
Location	**2805 Bowers Avenue** **Santa Clara, California**
Client	**GI Partners**
Client Address	**188 The Embarcadero, Suite 700** **San Francisco, California**
Project Number	**1413-1-1**
Date	**February 24, 2023**

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CORNERSTONE EARTH GROUP

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Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**2805 Bowers Data Center**
Location	**2805 Bowers Avenue**
	Santa Clara, California

SECTION 1: INTRODUCTION

This preliminary geotechnical investigation was prepared for the sole use of GI Partners for the 2805 Bowers Data Center project in Santa Clara, California. The purpose of this study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. For our use, we were provided with the following documents:

- A set of architectural plans titled "2805 Walsh Bowers, Issued for PCC Review," prepared by Sheehan, Nagle, Hartray Architects, dated September 30, 2022.

1.1 PROJECT DESCRIPTION

We understand the project is in the early planning stages and final development plans are not currently available. The project will consist of redeveloping the approximately 5-acre site for a new data center facility. Based on our review of the plans provided, we understand the new development will likely include a 4-story data center building with an exterior generator yard and substation. Based on our review of the plans provided, it appears the building will be about 60,735 square feet and located in the approximate center of the site with the generator yard to the east and substation to the south. At-grade auto and trailer parking and drive aisles will cover the remainder of the site. Appurtenant utilities, landscaping, storm water management areas, and other improvements necessary for overall site development will also be constructed.

Based on our experience with similar developments, column loads are anticipated to be high and typical of this type of construction. Site grading is anticipated to be minor with cuts and fills on the order of 1 to 3 feet.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated January 10, 2023 and consisted of a field program to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration program is presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of five Cone Penetration Tests (CPTs) advanced on February 1, 2023. The CPTs were advance to depths of 50 to 121½ feet. Seismic shear wave velocity measurements were collected from CPT-2.

The 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 explorations are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thicknesses in the area of the site ranges from about 400 to 500 feet (Rogers & Williams, 1974).

2.2 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)
Monte Vista-Shannon	6.5	10.5
Hayward (Southeast Extension)	7.5	12.1
Hayward (Total Length)	10.0	16.1
San Andreas (1906)	10.1	16.3
Calaveras	10.9	17.5

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

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

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

- 1948: The general site vicinity appears to be used for agriculture purposes. The project site appears to be a vacant lot. A drainage canal is also observed to run along the northeastern section of site, running generally north-south.

- 1980: A commercial structure and surrounding asphalt parking lot appear at the project site. Neighboring commercial structures also appear. The surrounding roadways are established including Bowers Avenue and Walsh Avenue.

- 1980-2020: No pertinent changes are observed at and near the site from 1980 to 2020, the last available aerial image of the project site.

3.2 SURFACE DESCRIPTION

The site is currently occupied by a two-story office building and surrounding asphalt concrete parking lot. The site appears relatively level, but graded to drain to storm drain facilities.

Various mature trees and landscaping islands are present within the parking lot and adjacent to the existing building.

Based on visual observations, areas of the existing pavement appear to have been recently re-paved or had a surface treatment applied. However, in general, the pavements appear to be in highly variable conditions ranging from fair to poor with areas of alligator cracking and longitudinal cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered interbedded layers of medium stiff to stiff clay with variable amounts of silt and sand, and medium dense to very dense sand with varying amounts of silt and gravels to the maximum depth explored of 121½ feet, where practical refusal was encountered.

3.4 GROUNDWATER

Groundwater was inferred from CPT pore pressure measurements at depths ranging from about 4 to 12 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. Historic high groundwater maps prepared by the California Geologic Survey (CGS, San Jose West 7.5-Minute Quadrangle, 2002) indicate the high groundwater to be approximately 5 to 10 feet below the existing ground surface.

Based on the above, on a preliminary basis, we recommend a design groundwater depth of 6 feet be used for preliminary planning. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. Seasonal and high groundwater elevations should be further evaluated during the design-level geotechnical investigation. Groundwater depth should be further evaluated as part of the design-level geotechnical investigation.

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, or a Santa Clara County Fault Hazard 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 estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2022 edition of the

California Building Code when an exception has been taken per ASCE 7-16, Section 11.4.8. For our preliminary analysis we have assumed an exception will be taken per ASCE 7-16 Section 11.4.8. If an exception is not taken, we should be notified so that we can perform a Ground Motion Hazard Analysis per Section 21.2 of ASCE 7-16. For our preliminary liquefaction analyses we used a PGA_M of 0.551g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, San Jose West Quadrangle, 2002) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our preliminary field program addressed this issue by testing potentially liquefiable layers to depths of at least 50 feet, and evaluating CPT data.

The potential for liquefaction should be further evaluated as part of the design-level geotechnical investigation.

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

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

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. 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-5) are presented on Figures 4A through 4E of this report.

4.3.3 Summary

Our preliminary analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from about ¼ to ¾ inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ½-inch or less over a horizontal distance of 30 to 40 feet. The potential for liquefaction induced settlement should be further evaluated as part of the design-level geotechnical investigation.

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 6-foot thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

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

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

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose to medium dense sands based on the work by Robertson and Shao (2010). Based on our preliminary analyses, the potential for significant seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the 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 6½ miles inland from the San Francisco Bay shoreline and is approximately 43 to 48 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as “0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile.” 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 partially located within a dam failure inundation area for the James J. Lenihan Reservoir. We recommend the project civil engineer be retained to confirm this information.

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. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once site development plans are finalized confirming where proposed structures are planned and once building loads are available. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Strong ground shaking
- Potential for significant static settlements
- Shallow groundwater
- Presence of moderately to highly expansive soils
- Re-development considerations

5.1.1 Strong Ground Shaking

Strong ground shaking is expected at this site, as with most sites in the Bay Area, during a major earthquake in the area. To mitigate the effects of strong ground shaking, all planned structures should be designed in accordance with the recommendations in a final design-level geotechnical report, and the most recent California Building Code.

5.1.2 Potential for Significant Static Settlement

The compressibility and stiffness of clays, the groundwater conditions beneath the site, and the building loads will all dictate the total estimated static settlements building foundations may experience. Due to the anticipated high building loads for the proposed four-story data center and anticipated subsurface conditions, we estimate large static and long-term consolidation settlements may occur over the design life of the structure. Based on our engineering judgment, experience with similar projects in the vicinity, and the subsurface conditions, on a preliminary basis, the proposed building may need to be supported on shallow foundations over ground improvement or a deep foundation system. However, additional site-specific subsurface explorations and settlements estimates should be performed and evaluated during a design-level geotechnical investigation.

5.1.3 Shallow Groundwater

Shallow groundwater was inferred from pore pressure dissipation tests in our CPTs at depths ranging from approximately 4 to 12 feet below the existing ground surface. As discussed above, on a preliminary basis we recommend a design groundwater depth of 6 feet. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Preliminary recommendations addressing this concern are presented in the "Anticipated Earthwork" section of this report and should be further evaluated during the design-level geotechnical investigation.

5.1.4 Presence of Moderately to Highly Expansive Soils

Based on our experience in the area and nearby sites, we anticipate moderately to high expansive soils may be present across the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and hard when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. We recommend the expansive potential of the surficial soils be further evaluated during our design-level geotechnical investigation.

5.1.5 Re-Development Considerations

As discussed, the site is currently occupied by a two-story office building and at-grade asphalt pavement parking lots and site improvements. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fill. Preliminary recommendations addressing these issues are presented in the "Anticipated Earthwork" section of this report. We recommend the presence of existing fills and improvements be further evaluated during our design-level geotechnical investigation.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this preliminary investigation were based on limited site development information, limited exploration, and review of available subsurface information and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) 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; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend that any existing foundations, debris, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, any native soils that are disturbed during demolition of the existing improvements should also be removed and replaced as engineered fill. We anticipate undocumented fill associated with prior site development may be present at the site. If ground improvement is implemented and designed to mitigate potential settlement due to the presence of undocumented fill, the undocumented fill may potentially be left in place.

Historic high groundwater maps prepared by the California Geologic Survey (CGS, San Jose West 7.5-Minute Quadrangle, 2002) indicate the high groundwater to be approximately 5 to 10 feet below the existing ground surface. On a preliminary basis, we used a design groundwater depth of 6 feet. Dewatering of deeper excavations should be anticipated along with the need to stabilize the excavation bottoms with material such as crushed rock. High moisture content soils should be expected and will require drying back to be re-used as engineered fill.

Surface water runoff should not be allowed to pond 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 away from buildings. Bio-treatment basins should be kept at least 10 feet away from buildings and, where possible, at least 3 feet away from pavements and flatwork.

SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We anticipate that the project structural design will be based on the 2022 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30}) and the anticipated soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our CPT explorations generally encountered medium stiff to hard clay and medium dense to very dense sand deposits to a depth of 121½ feet, where practical refusal was encountered. Shear wave velocity (V_S) measurements were performed while advancing CPT-2 to a depth of 121½ feet, resulting in a time-averaged shear wave velocity for the top 100 feet (V_S) of 250 meters per second (818 feet per second). Based on the shear wave velocity measures and available geologic data, on a preliminary basis, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_s and S_1 were calculated using the webbased program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. ***Recommended values in Table 2 may only be used for design if in the judgement of the project structural engineer the exception for Site Class D can be taken per ASCE 7-17 Section 11.4.8.*** Based on our current project understanding and experience with similar projects, we anticipate a site-specific analysis in accordance with ASCE 7-16 Chapter 21 will be required. On a preliminary basis, we recommend a site-specific analysis be planned for and performed during the design-level investigation.

The table below lists the various factors used to determine the seismic coefficients and other parameters. We recommend the site classification and be confirmed during the design-level geotechnical investigation.

Table 2: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.372356°
Site Longitude	-121.976514°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.500g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.7
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.530g*
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.000g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.020g*

*Per Site Class D exception, ASCE 7-16 Section 11.4.8 and Supplement 3.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, due to the anticipated significant total and differential settlement, the proposed structure may need to be supported on shallow foundations overlying ground improvement or a deep foundation system. Additional preliminary ground improvement recommendations are provided below. Foundation recommendations and ground improvement alternatives should be evaluated further during the design-level investigation.

8.2 SHALLOW FOUNDATIONS OVERLYING GROUND IMPROVEMENT

If determined during the design-level geotechnical investigation that estimated total and differential settlements are still of concern, shallow foundations would likely not be feasible unless they are supported on ground improvement. Ground improvement, such as vibro replacement (i.e. stone columns), granular compaction piles (i.e. rammed aggregate), grouted displacement columns (i.e. CLSM), deep dynamic compaction (DDC), or similar densification techniques, should be designed to provide vertical support through the existing soils.

8.2.1 Conventional Shallow Foundations

On a preliminary basis, the planned structures may be supported on conventional shallow footings overlying ground improvement. Footings should bear on engineered fill overlying ground improvement, and extend at least 30 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is recommended due to the potential presence of moderately to highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Bearing pressures will be dependent on the final ground improvement technique and spacing; however, substantial improvement in bearing capacity would be expected. On a preliminary basis, we expect allowable bearing pressures on the order of 4,000 to 5,000 psf for combined dead plus live loads would be feasible.

Ground improvement should be designed to reduce total settlement due to potential static and seismic conditions to tolerable levels. The feasibility of conventional shallow foundations with ground improvement should be evaluated during the design-level geotechnical investigation.

8.2.2 Ground Improvement

Ground Improvement, such as vibro replacement (i.e. stone columns), granular compaction piles (i.e. rammed aggregate), grouted displacement columns (i.e. CLSM), deep dynamic compaction (DDC), or similar densification techniques, should be designed to provide vertical support through the existing soils, as well as partial mitigation of the liquefaction potential. If implemented, we anticipate that the ground improvement construction will be a design-build process where Cornerstone Earth Group will review preliminary design-build submittals, including proposed spacing and layout relative to the foundation plans and installation lengths, and anticipated densification improvement of the surrounding soils prepared by prospective contractors, provided comments, and come to a general agreement with the contractor on the intended design approach.

On a preliminary basis, the ground improvement design should be such that the total foundation settlement (static and seismic) are reduced to about 1 to 1½ inch or less, with no more than 1 inch for either the static or seismic component.

8.3 DEEP FOUNDATIONS

On a preliminary basis, the proposed structure may be supported by a deep foundation system, such as conventional drilled, cast-in-place augercast (APG) piles. APG piles have been successfully used for projects throughout the Bay Area and California in similar soil conditions. APG piles are constructed by augering and removing the soil column as a hollow-stem auger is advanced, prior to pumping sand-cement grout (4,000 to 6,000 psi) through the hollow-stem as the drill stem is extracted. A benefit of the augercast pile installation process is that augercast piles are a low noise and vibration installation compared to driven piles. If this option is desired, additional information, including vertical and lateral pile capacities ca be provided in a design-level report.

On a preliminary basis, we recommend that a pile load test program be developed. On a preliminary basis, we anticipate the test program may include one (1) compression test and one (1) tension test for every 150 to 250 piles installed. Additional discussion on pile testing will be included in a design-level investigation.

SECTION 9: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of GI Partners specifically to support the design of the 2805 Bowers Data Center project in Santa Clara, California. The opinions, conclusions, and preliminary 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.

Preliminary recommendations in this report are based upon the soil and ground water conditions encountered during our limited subsurface exploration. Preparation of a design-level investigation is anticipated to provide additional information and refine the preliminary recommendations presented herein. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

GI Partners may have provided Cornerstone with plans, reports and other documents prepared by others. GI Partners 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 owner's 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 10: REFERENCES

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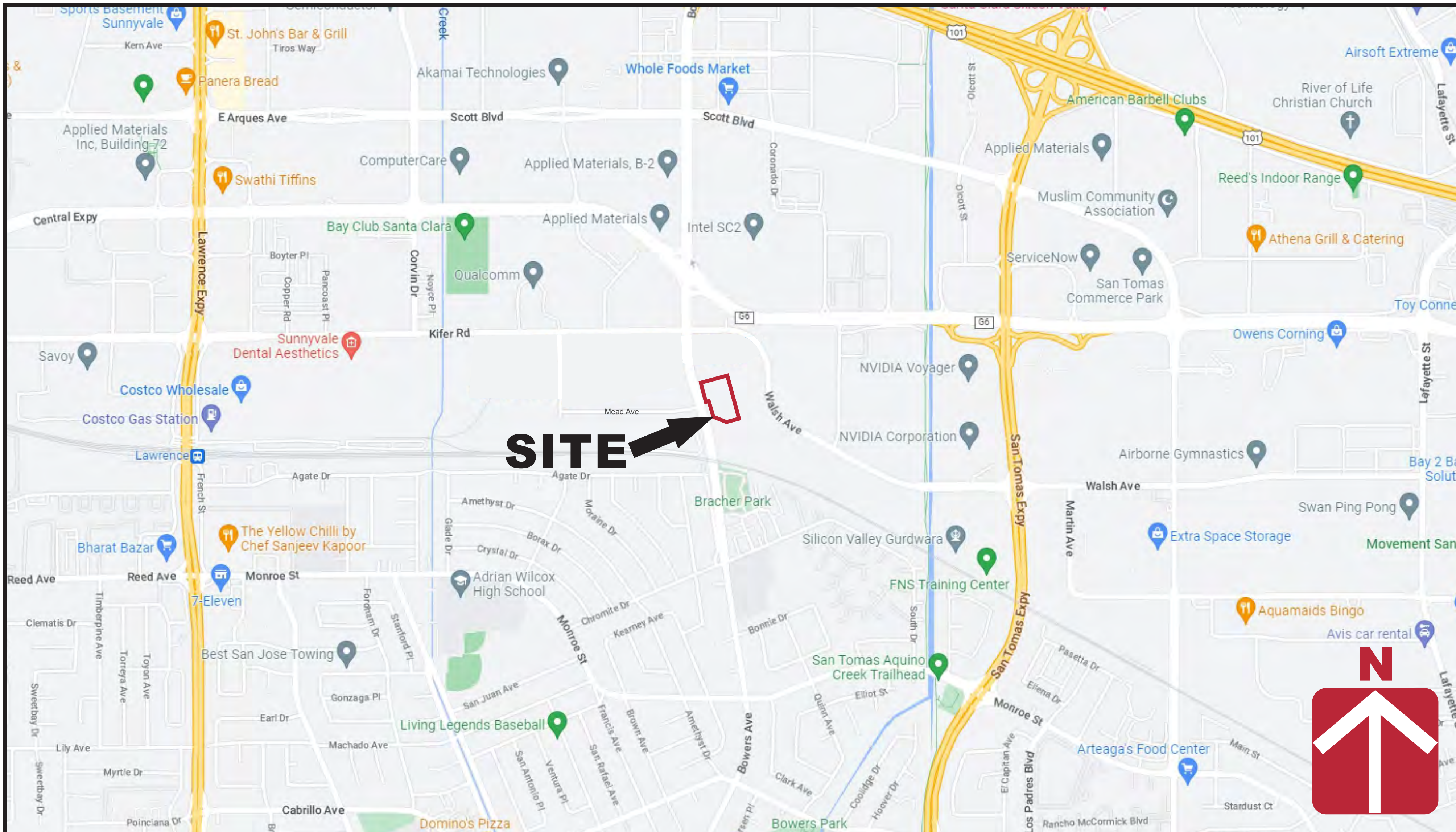
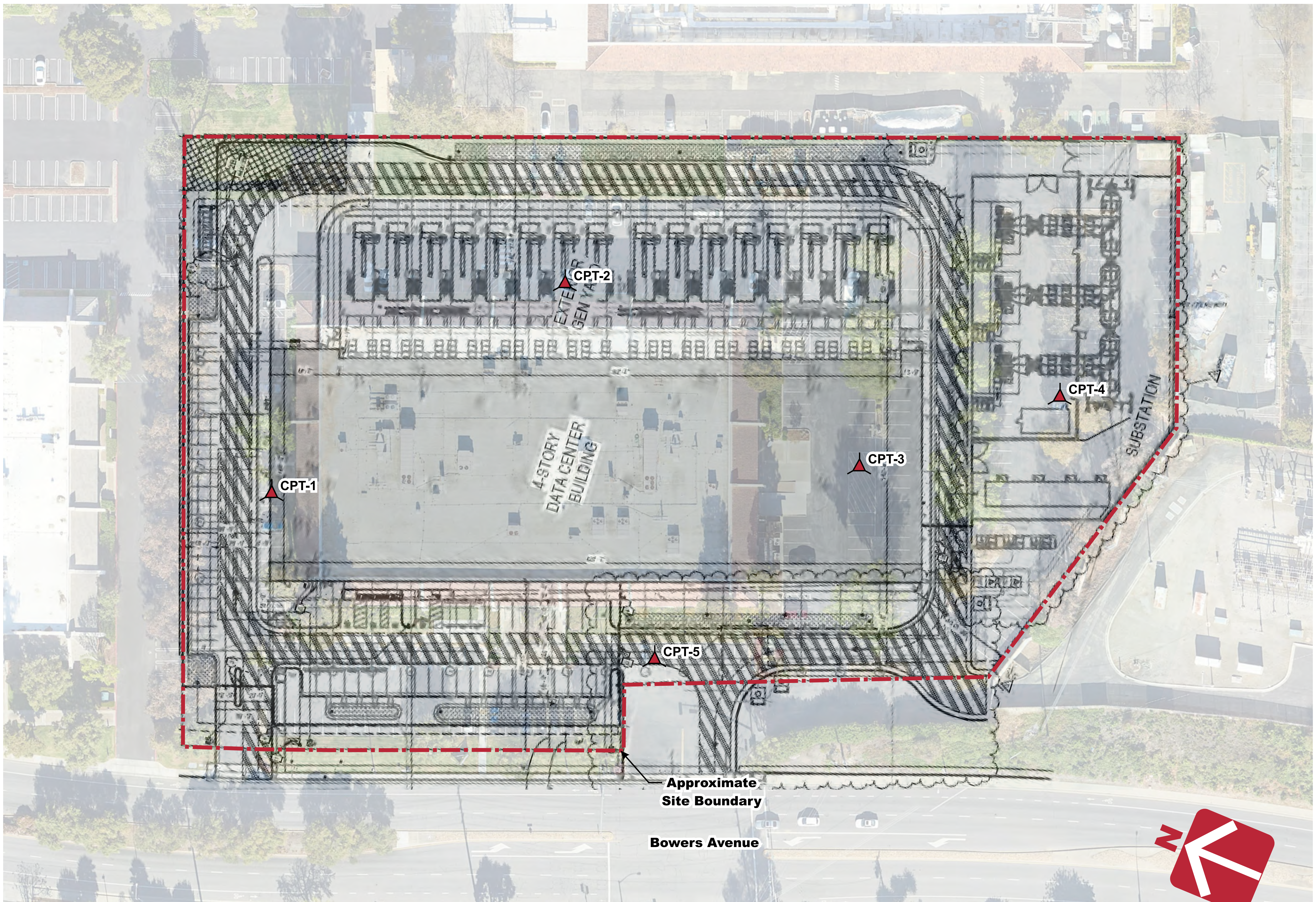


CORNERSTONE EARTH GROUP	**Vicinity Map**	Project Number 1413-1-1	
	2805 Bowers Data Center **Santa Clara, CA**	Figure Number Figure 1	
		Date February 2023	Drawn By RRN



Legend

Approximate location of cone penetration test (CPT)

Base by Google Earth, dated 03/10/2022
Overlay: Proposed Site Plan, undated

Project Number 1413-1-1

Figure Number Figure 2

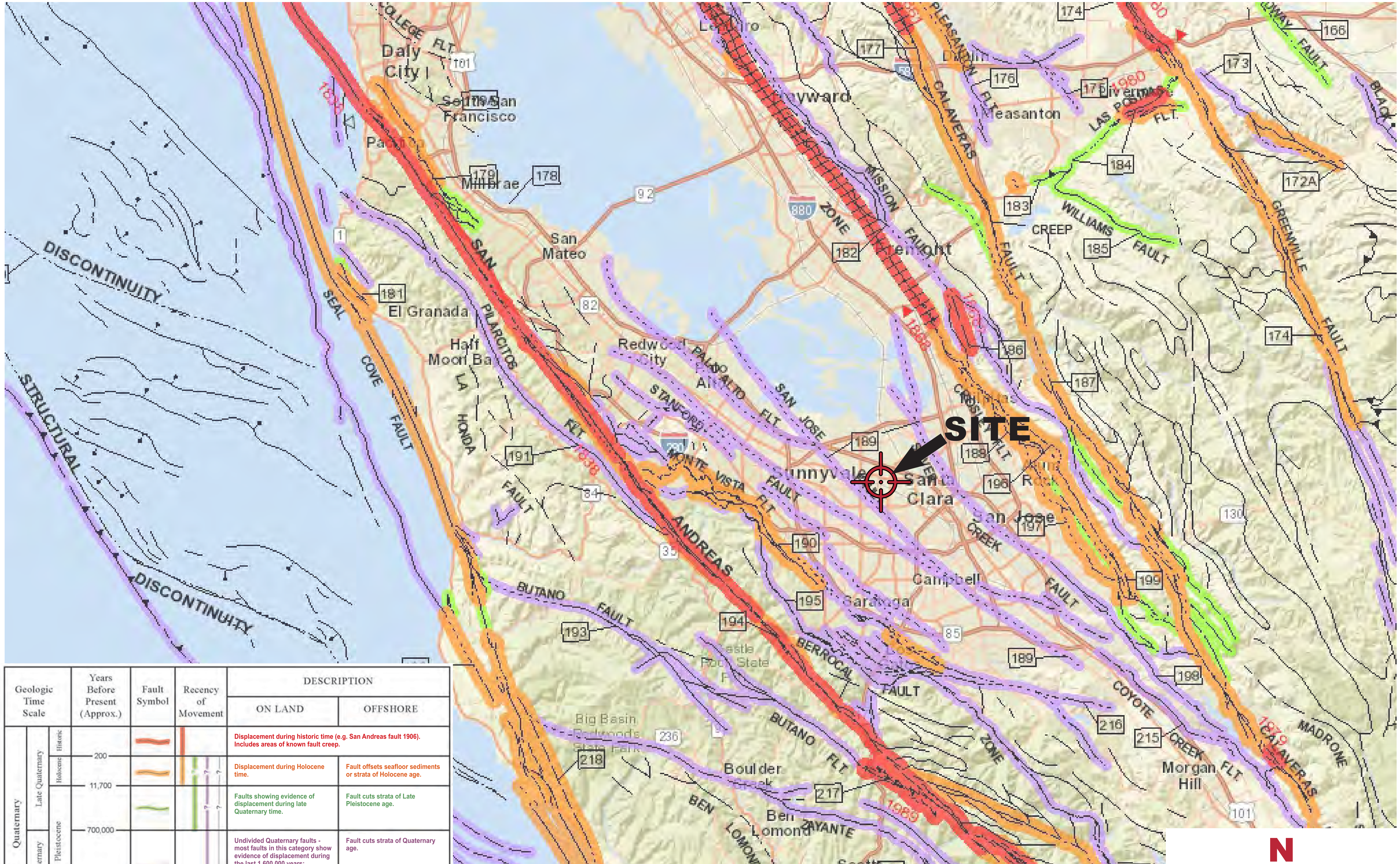
Date February 2023

Drawn By RRN

Site Plan

2805 Bowers Data Center
Santa Clara, 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)

N

0 5 10

APPROXIMATE SCALE (MILES)

Project Number 1413-1-1

Figure Number Figure 3

Drawn By RRN

Date February 2023

Regional Fault Map

2805 Bowers Data Center
Santa Clara, CA

CORNERSTONE EARTH GROUP

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FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	**2805 Bowers Data Center**
Project No.	**1413-1-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.551**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**3.7**
Design Water Depth (feet)	**6**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.64 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.6 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.42 **L/H** 119.0

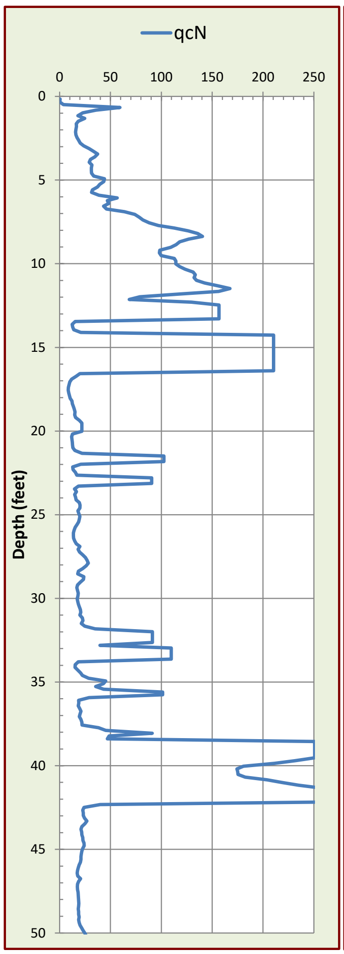
LDI[1]**Corrected for Distance** 0.05 (4 < L/H < 40)

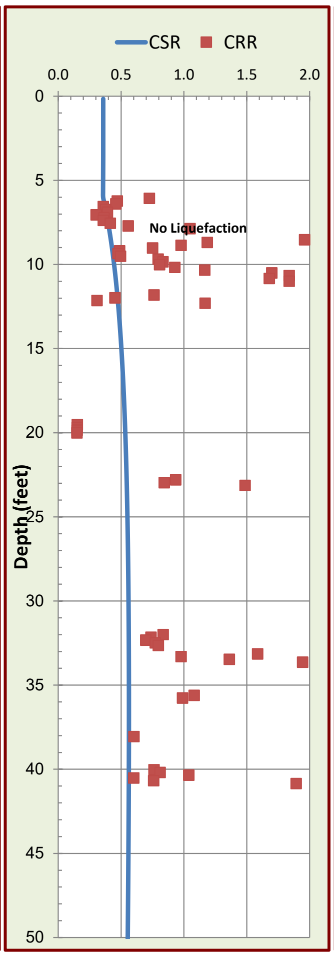
EXPECTED RANGE OF DISPLACEMENT

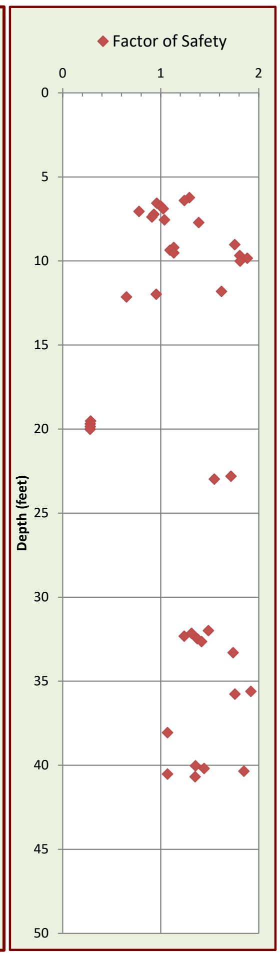
0.0 **to** **0.1** **feet**

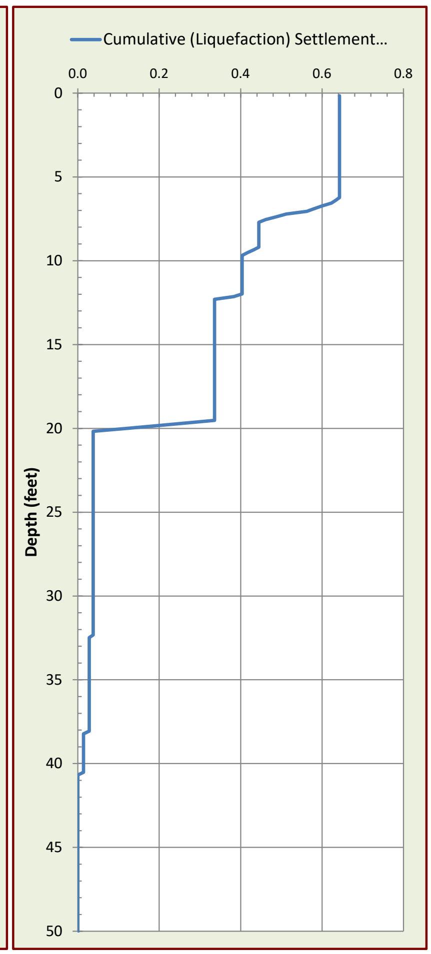
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE	4B
CPT NO.	2

PROJECT/CPT DATA

Project Title	**2805 Bowers Data Center**
Project No.	**1413-1-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.551**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**9**
Design Water Depth (feet)	**6**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM	6	**FEET**
	0.00	**(Inches)**
LIQUEFACTION SETTLEMENT FROM	50	**FEET**
	0.75	**(Inches)**
TOTAL SEISMIC SETTLEMENT	0.7	**INCHES**

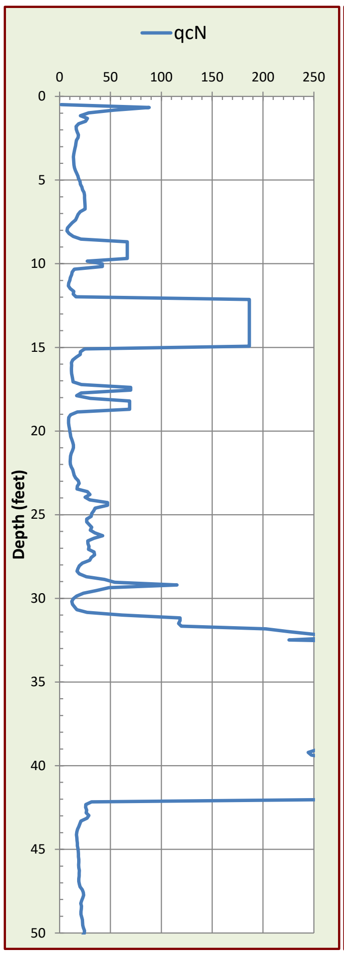
POTENTIAL LATERAL DISPLACEMENT

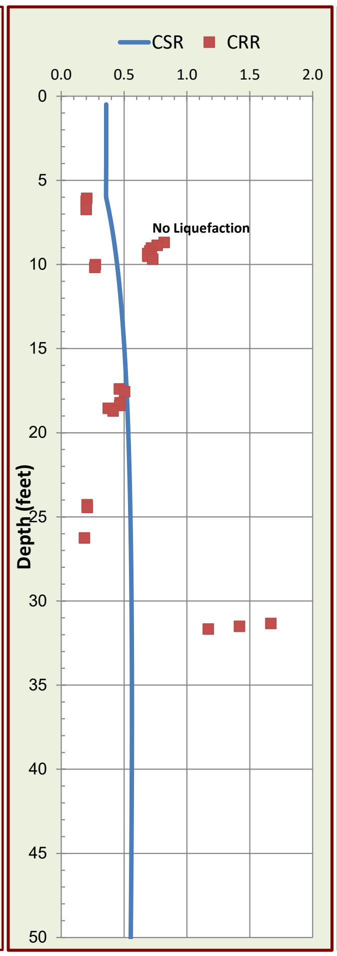
LDI[2]	0.48	**L/H**	108.8
LDI[1]$_{\text{Corrected for Distance}}$	0.07	(4 < L/H < 40)	

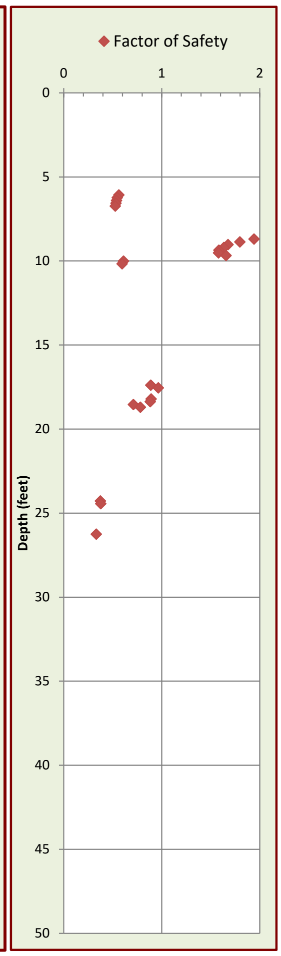
EXPECTED RANGE OF DISPLACEMENT

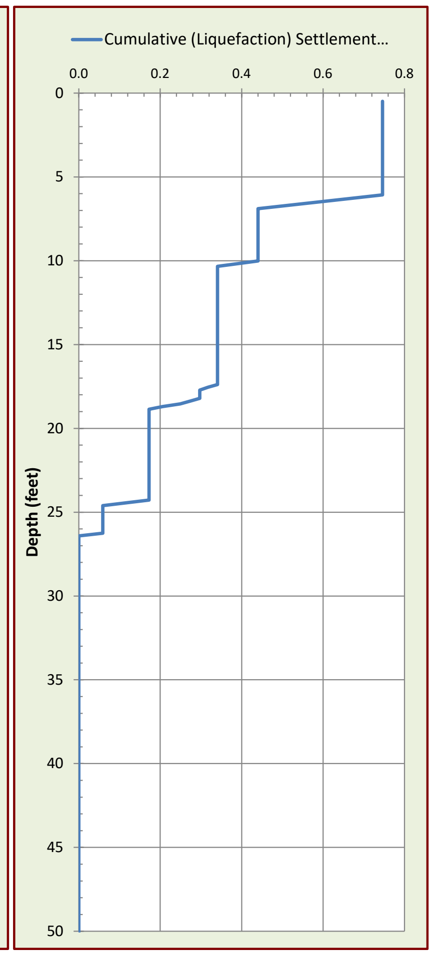
0.0	**to**	**0.1**	**feet**

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title	**2805 Bowers Data Center**
Project No.	**1413-1-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.551**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**11.8**
Design Water Depth (feet)	**6**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.23 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.15 **L/H** 110.0

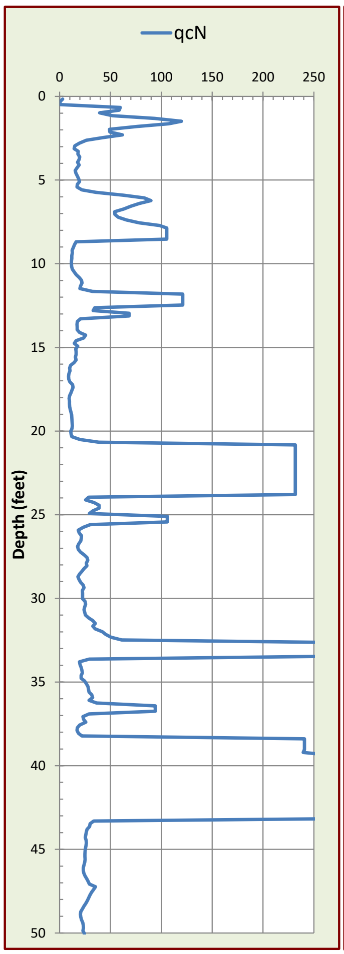
LDI[1]**Corrected for Distance** 0.02 (4 < L/H < 40)

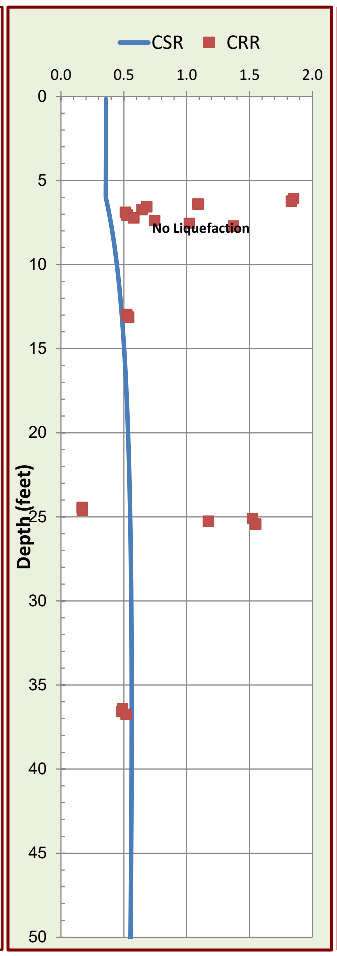
EXPECTED RANGE OF DISPLACEMENT

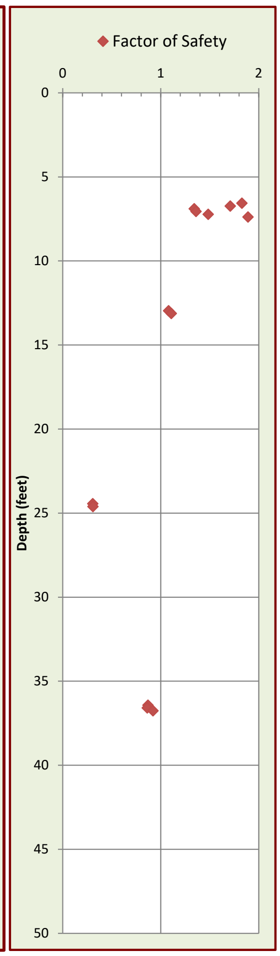
0.0 **to** 0.0 **feet**

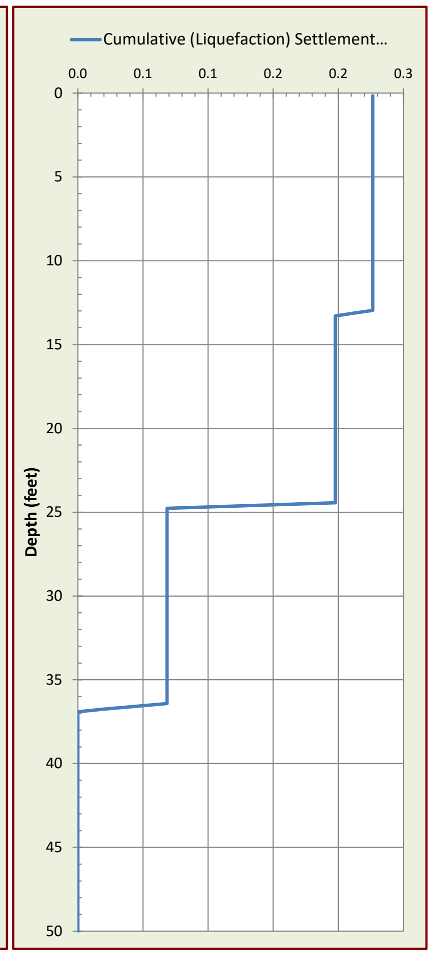
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title: **2805 Bowers Data Center**

Project No.: **1413-1-1**

Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **San Andreas**

Earthquake Magnitude (Mw): **7.9**

PGA (Amax): **0.551** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **8.6**

Design Water Depth (feet): **6**

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

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

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.36 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.24 **L/H** 109.5

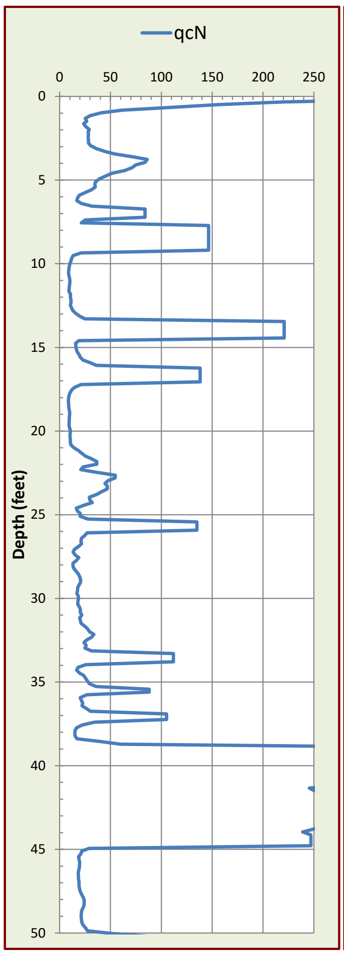
LDI[1] Corrected for Distance 0.03 (4 < L/H < 40)

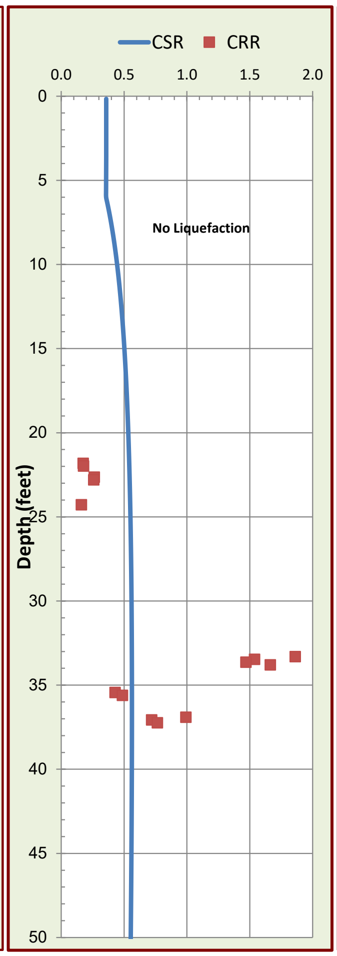
EXPECTED RANGE OF DISPLACEMENT

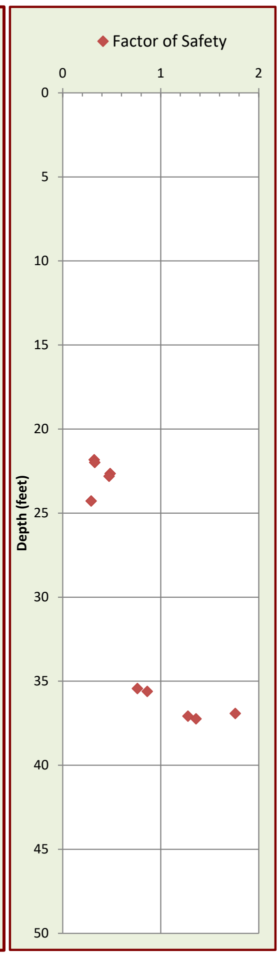
0.0 **to** 0.1 **feet**

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

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







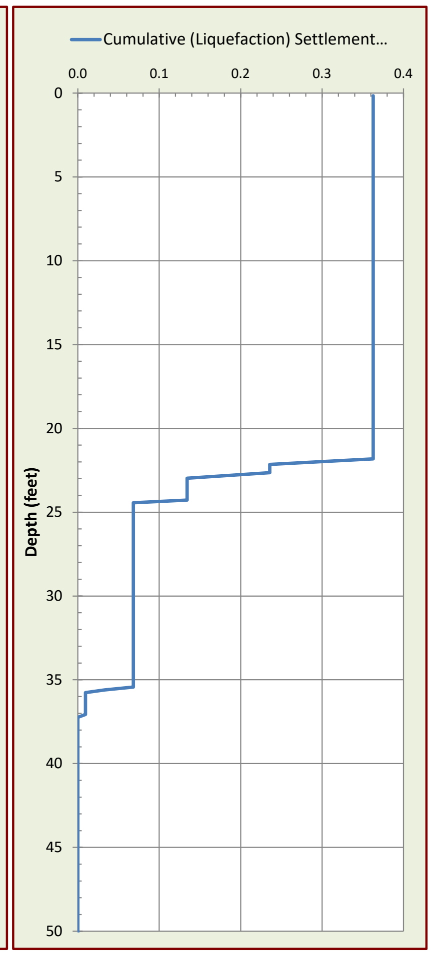


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FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title	2805 Bowers Data Center
Project No.	1413-1-1
Project Manager	MFR

SEISMIC PARAMETERS

Controlling Fault	San Andreas	
Earthquake Magnitude (Mw)	7.9	
PGA (Amax)	0.551	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	6.2
Design Water Depth (feet)	6
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 6 FEET

0.01 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.76 (Inches)

TOTAL SEISMIC SETTLEMENT 0.8 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.68 L/H 120.0

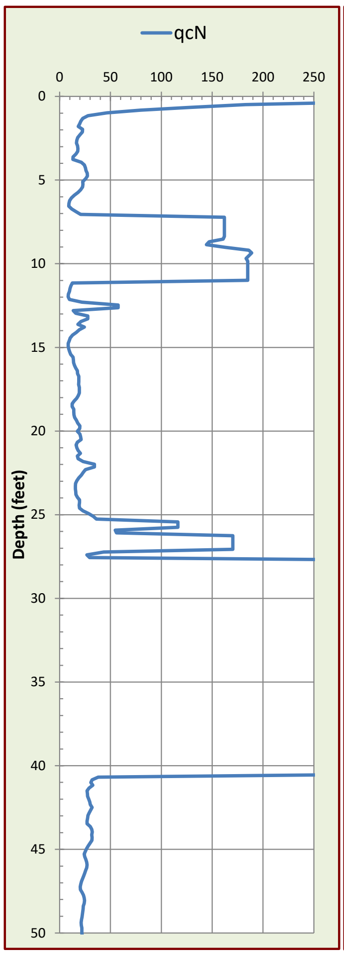
LDI[1] Corrected for Distance 0.09 (4 < L/H < 40)

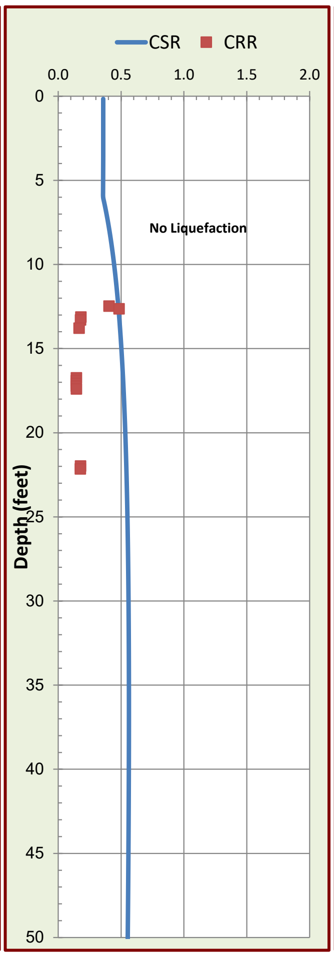
EXPECTED RANGE OF DISPLACEMENT

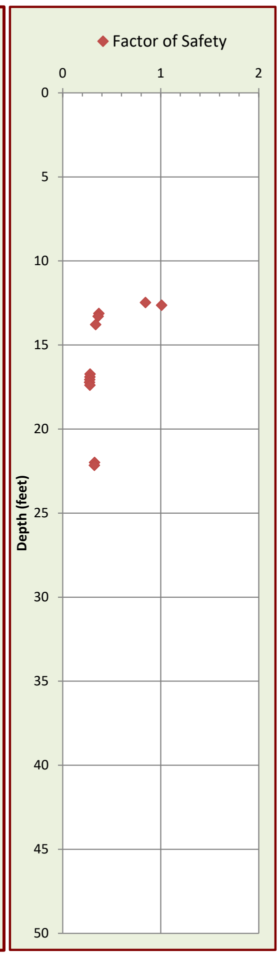
0.0 to 0.2 feet

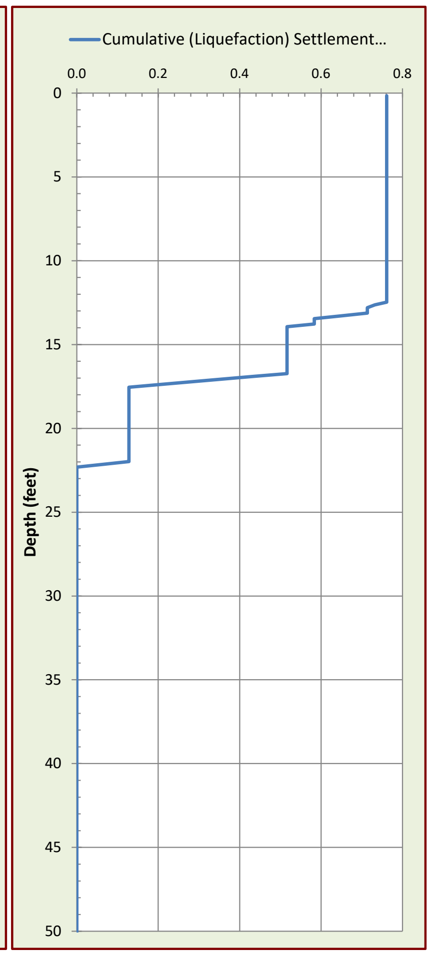
[1]Not Valid for L/H Values < 4 and > 40.

[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 25-ton truck-mounted Cone Penetration Test equipment. Five CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on February 1, 2023, to depths ranging from 50 to 121½ feet, or practical refusal. The approximate locations of the CPTs are shown on the Site Plan, Figure 2. CPT logs are included as part of this appendix.

CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. CPT elevations were not determined. The locations of the CPTs should be considered accurate only to the degree implied by the method used.

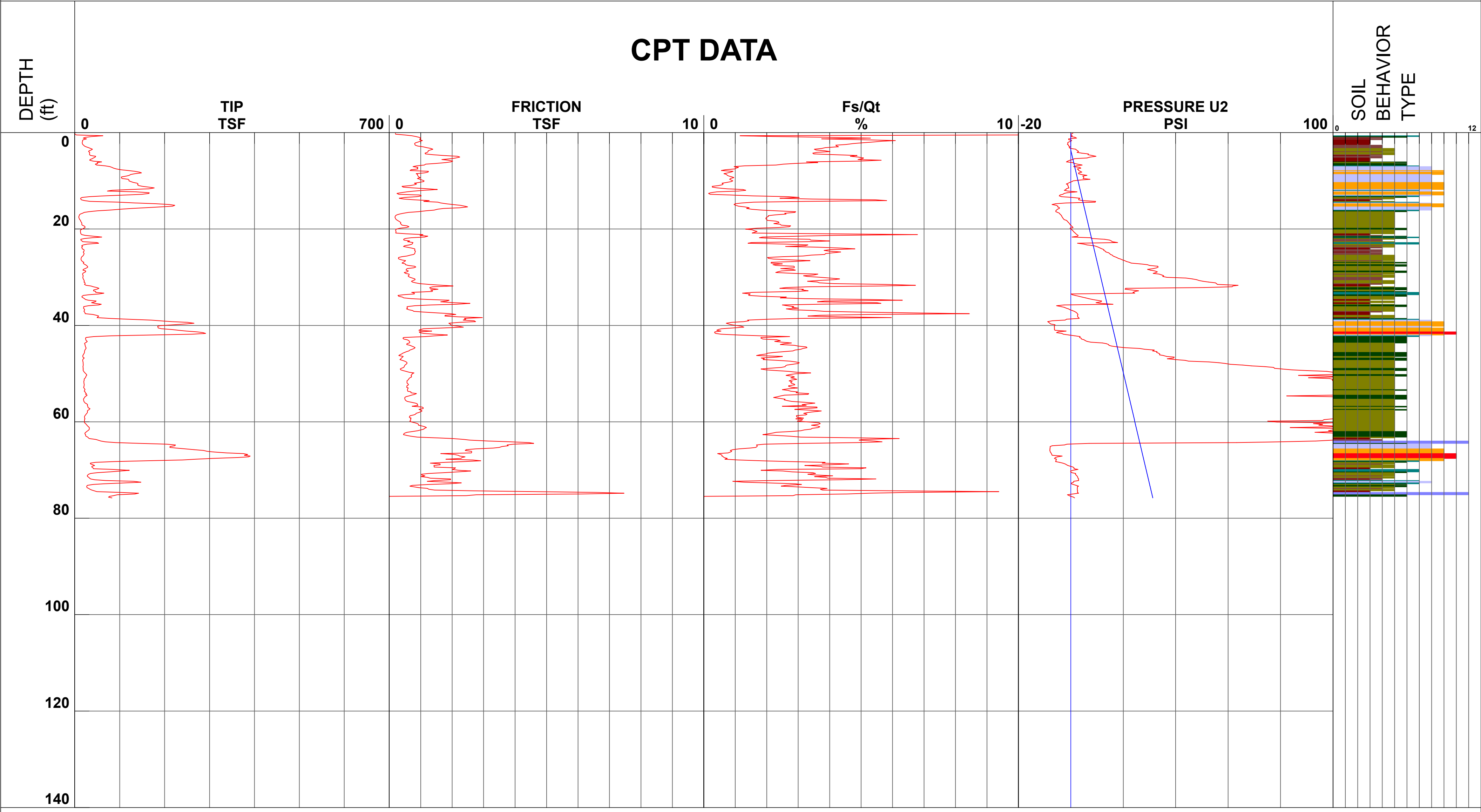
The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the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

Attached 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 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.

Cornerstone Earth Group

Project	2805 Bowers Ave	Operator	JM-GM	Filename	SDF(842).cpt
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-01	Date and Time	1413-1-1	Maximum Depth	75.79 ft
EST GW Depth During Test		3.70 ft			

Net Area Ratio .8



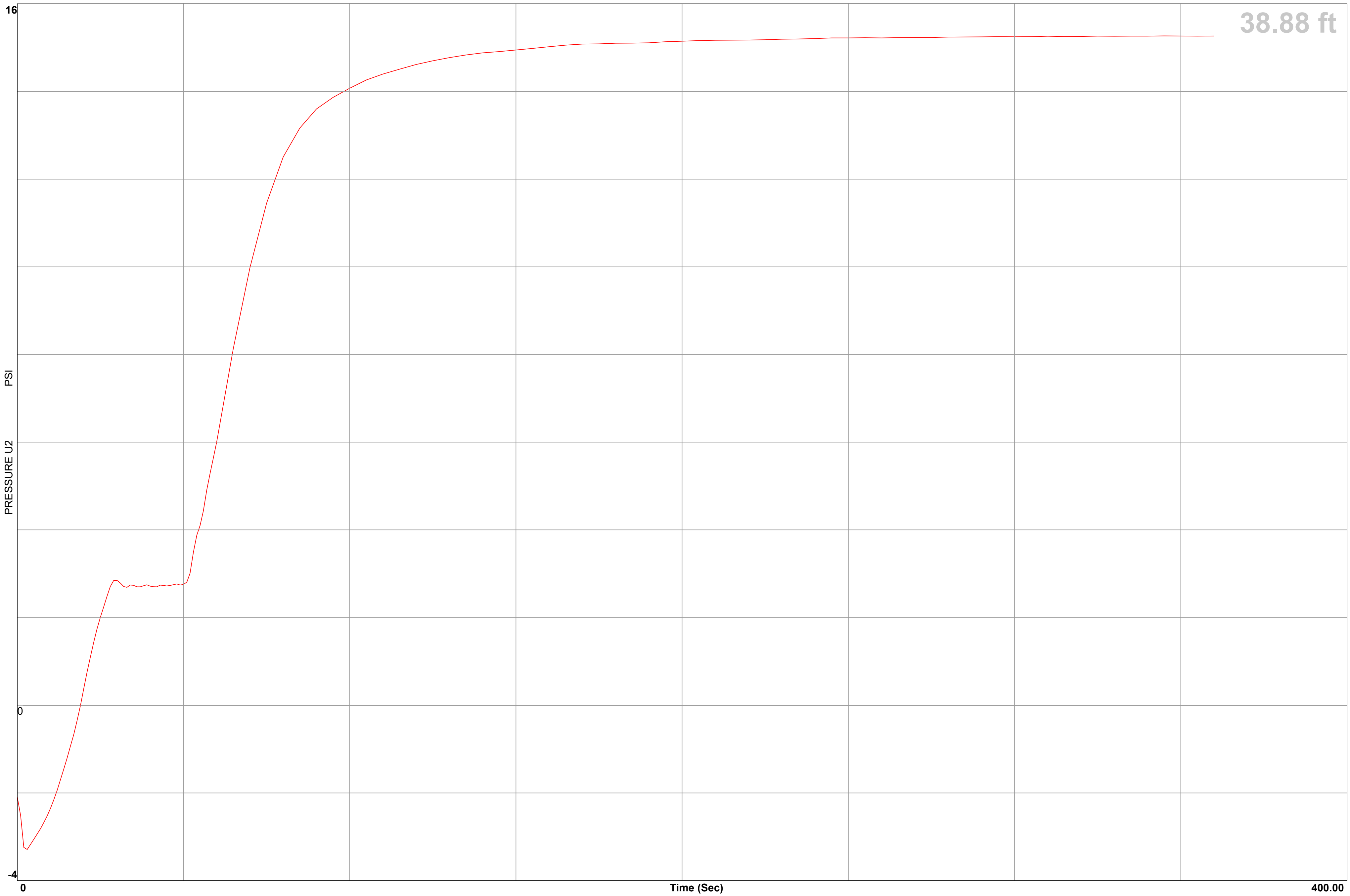
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	2805 Bowers Ave	Operator	JM-GM		
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-01	Date and Time	2/1/2023 1:27:57 PM		
Equilized Pressure	15.2	EST GW Depth During Test	3.7		

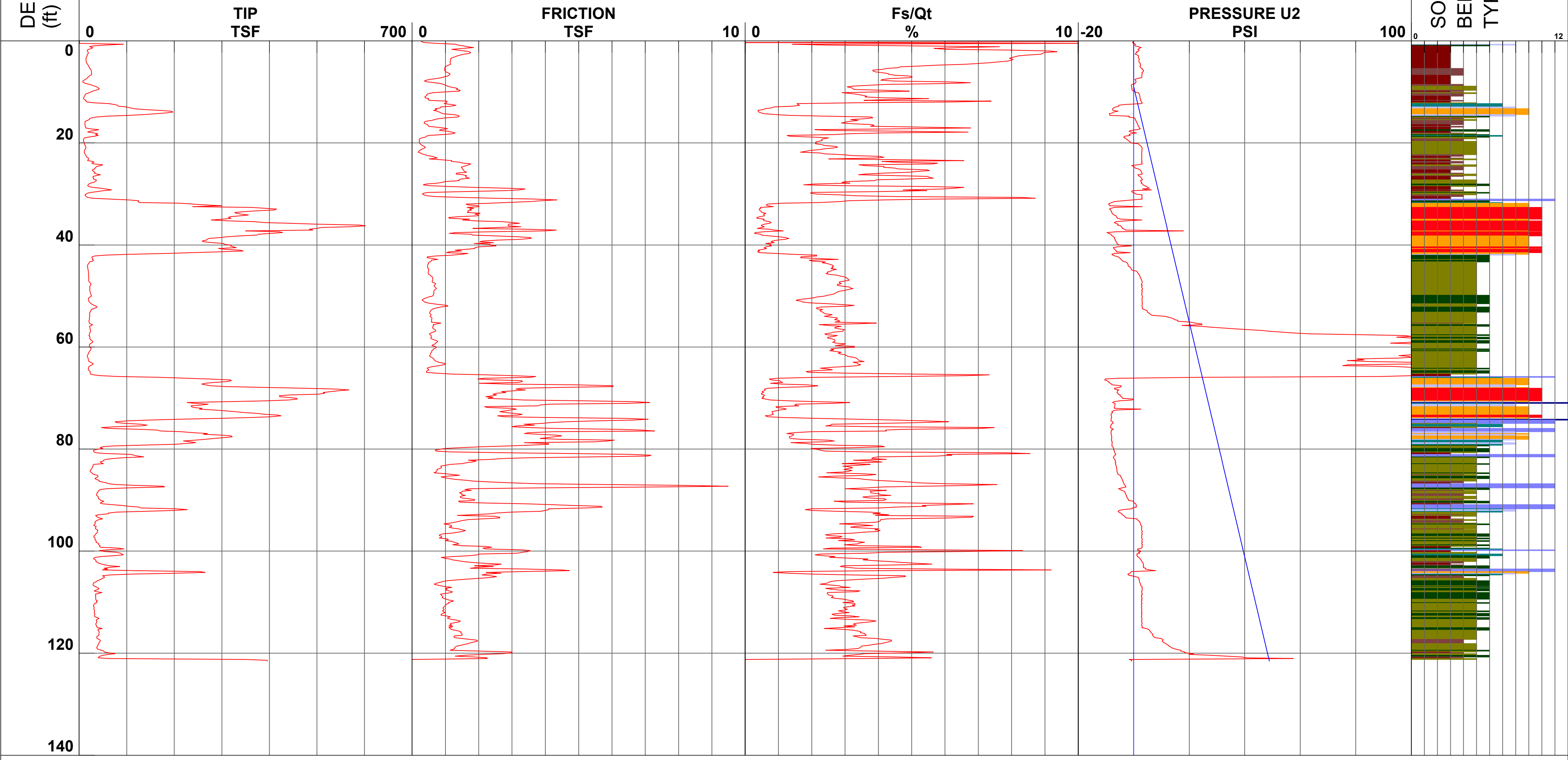


Cornerstone Earth Group

Project	2805 Bowers Ave	Operator	JM-GM	Filename	SDF(841).cpt
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-02	Date and Time	1413-1-1	Maximum Depth	121.55 ft
EST GW Depth During Test		9.00 ft			

Net Area Ratio .8

CPT DATA



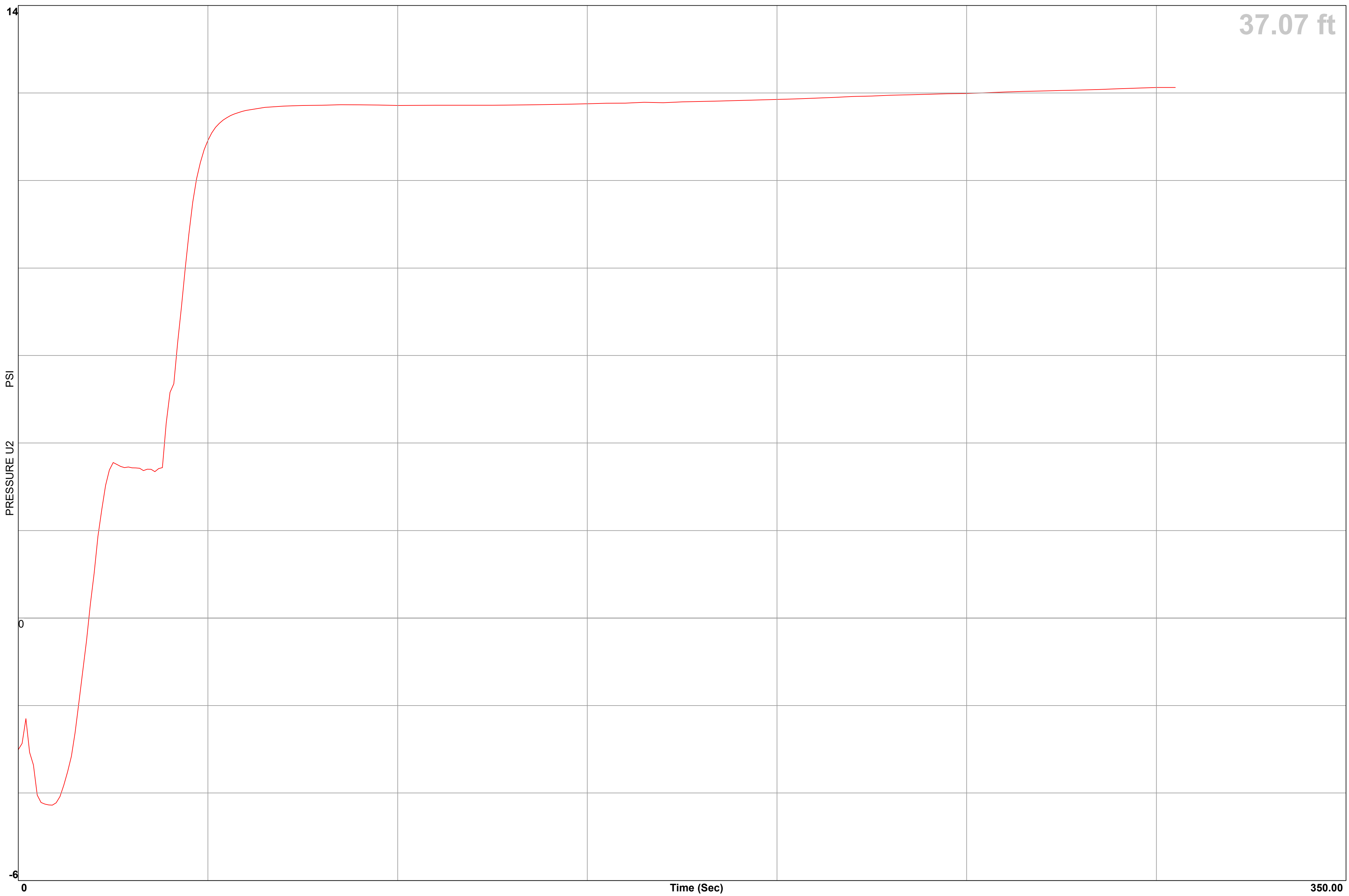
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

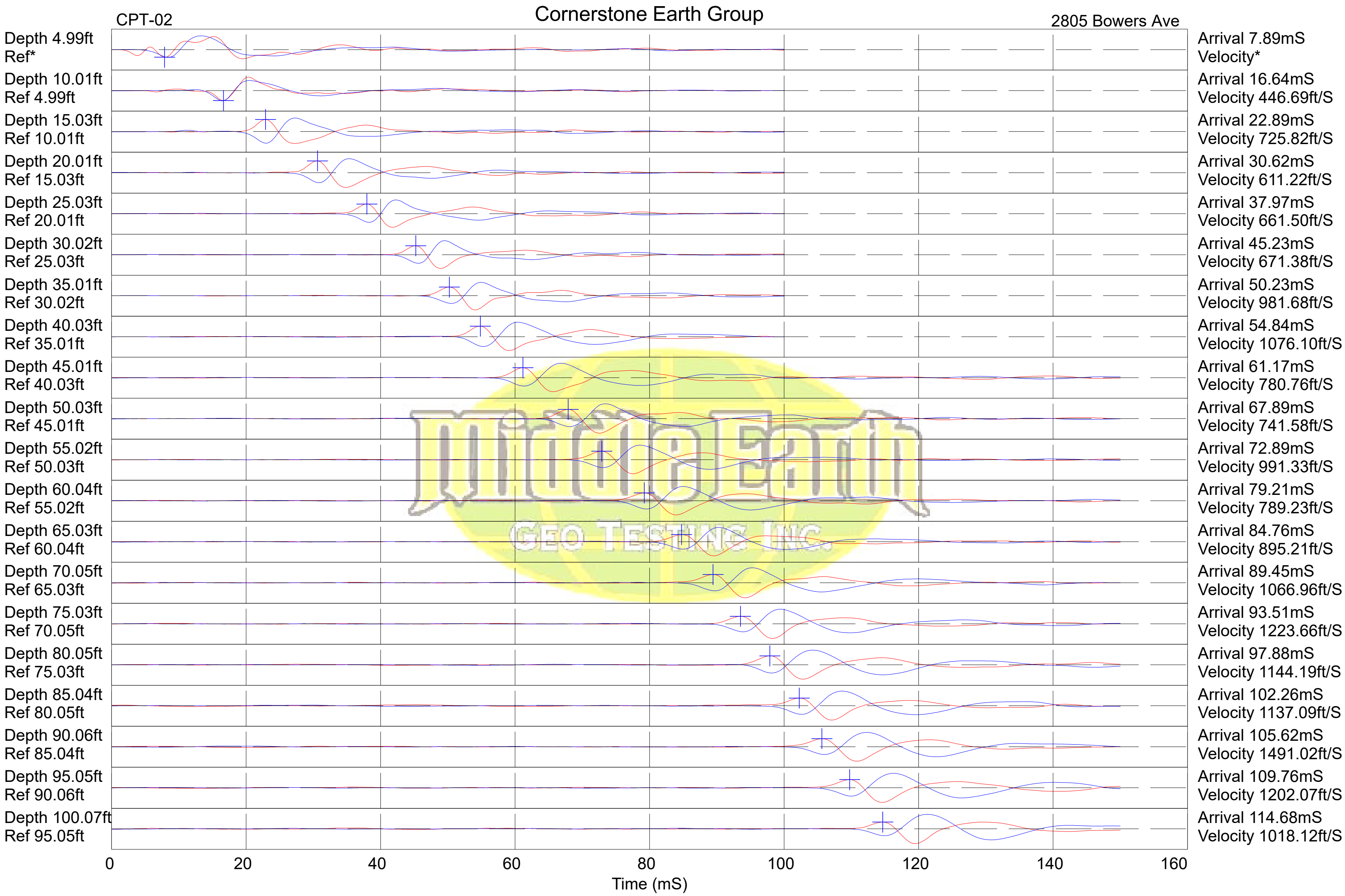
Cone Size 15cm squared

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

Cornerstone Earth Group

Location	2805 Bowers Ave	Operator	JM-GM		
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-02	Date and Time	2/1/2023 10:28:44 AM		
Equilized Pressure	12.1	EST GW Depth During Test	9.0		





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

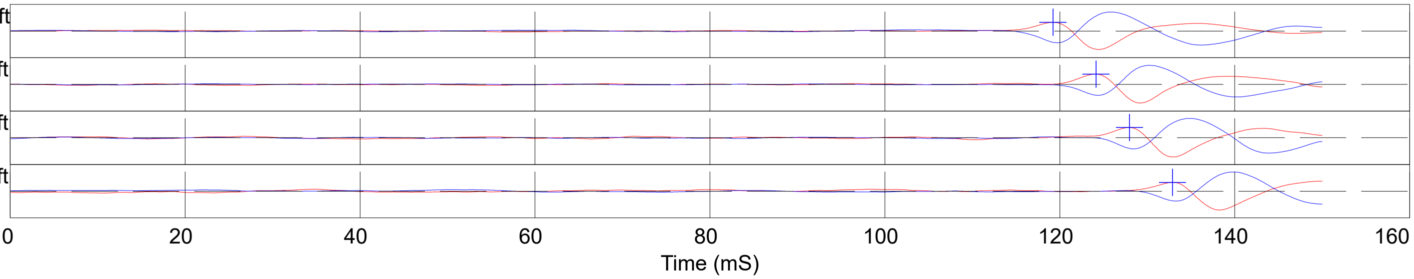
COMMENT:

CPT-02

SEISMIC TEST

2805 Bowers Ave

Depth 105.05ft Ref 100.07ft	Arrival 119.21mS Velocity 1098.85ft/S
Depth 110.63ft Ref 105.05ft	Arrival 124.13mS Velocity 1131.61ft/S
Depth 114.99ft Ref 110.63ft	Arrival 127.96mS Velocity 1138.41ft/S
Depth 119.98ft Ref 114.99ft	Arrival 132.88mS Velocity 1012.02ft/S



Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:

Middle Earth Geo Testing Inc.

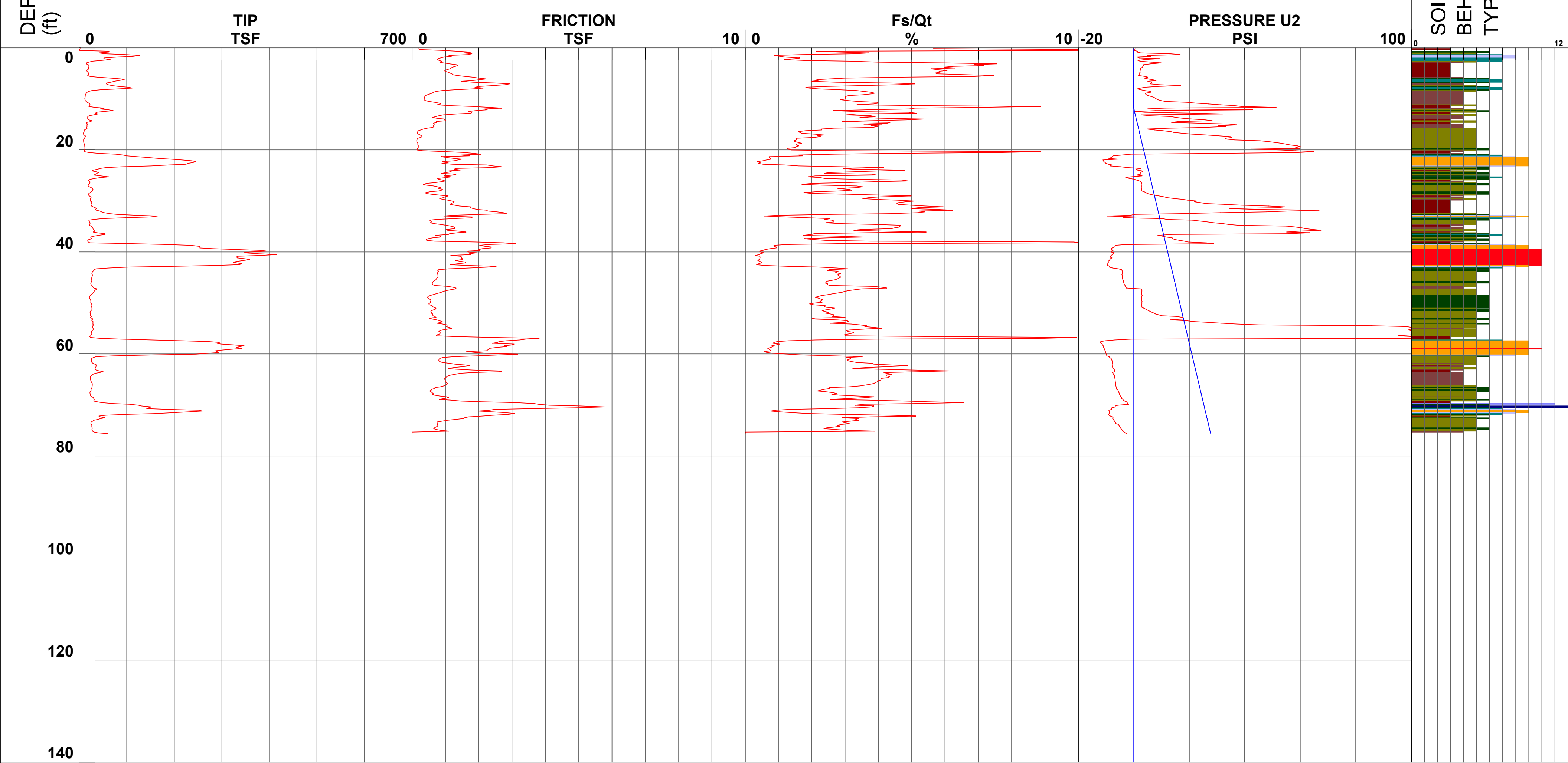
Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project	2805 Bowers Ave	Operator	JM-GM	Filename	SDF(840).cpt
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-03	Date and Time	1413-1-1	Maximum Depth	75.62 ft
EST GW Depth During Test		11.80 ft			

Net Area Ratio .8

CPT DATA



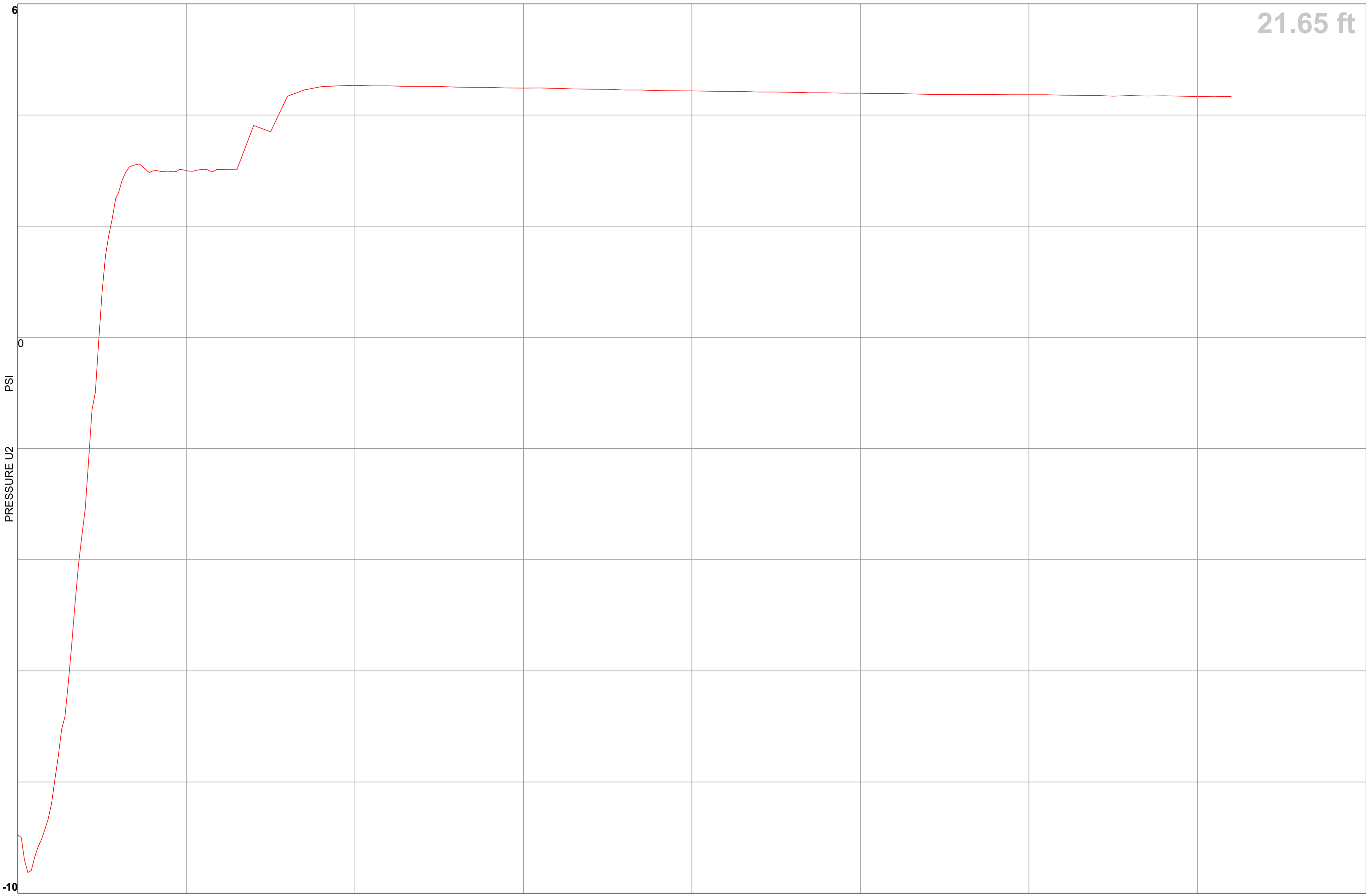
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	2805 Bowers Ave	Operator	JM-GM		
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-03	Date and Time	2/1/2023 7:39:46 AM		
Equilized Pressure	4.2	EST GW Depth During Test	11.8		

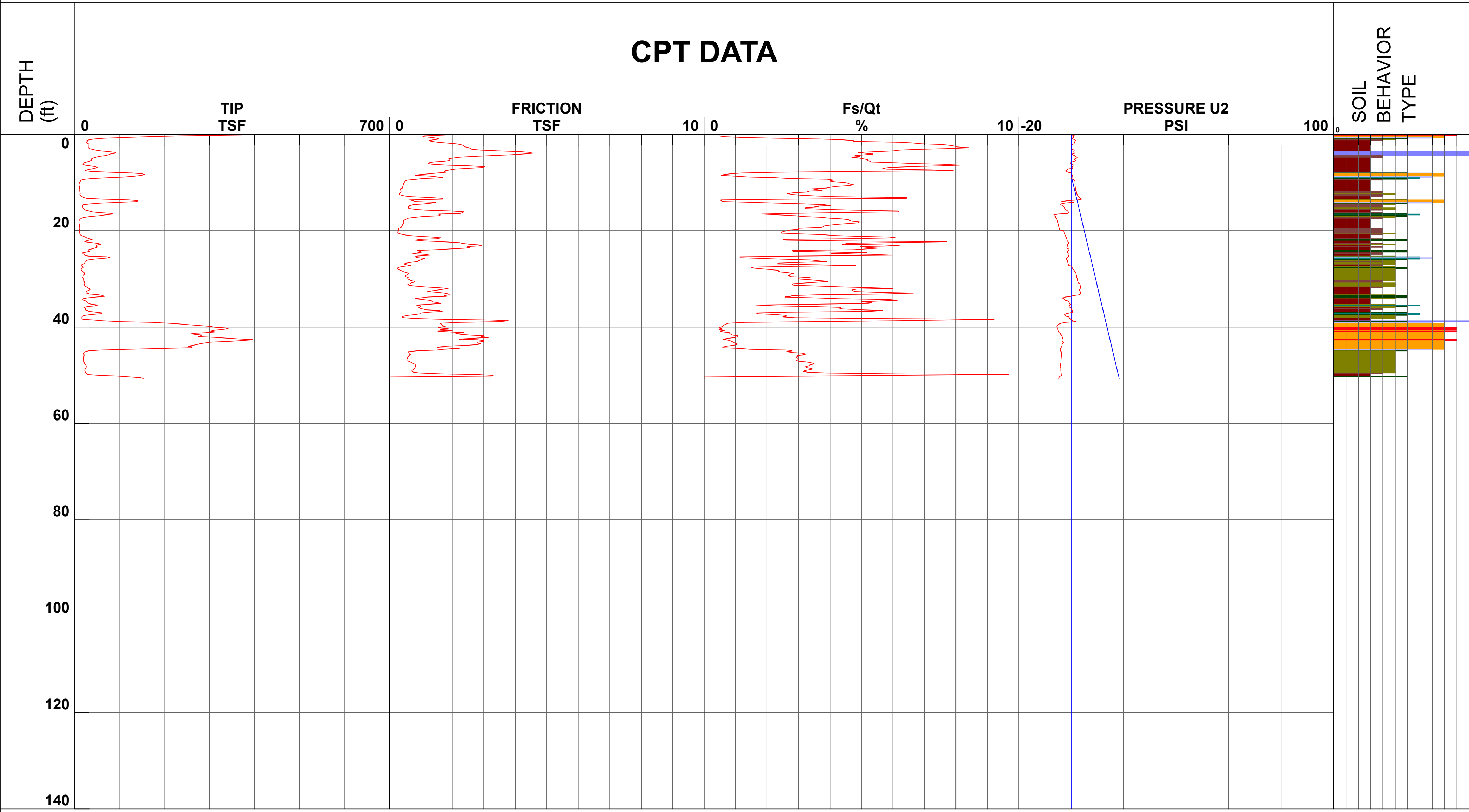


Cornerstone Earth Group

Project	2805 Bowers Ave	Operator	JM-GM	Filename	SDF(844).cpt
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-04	Date and Time	1413-1-1	Maximum Depth	50.69 ft
EST GW Depth During Test		8.60 ft			

Net Area Ratio .8

CPT DATA



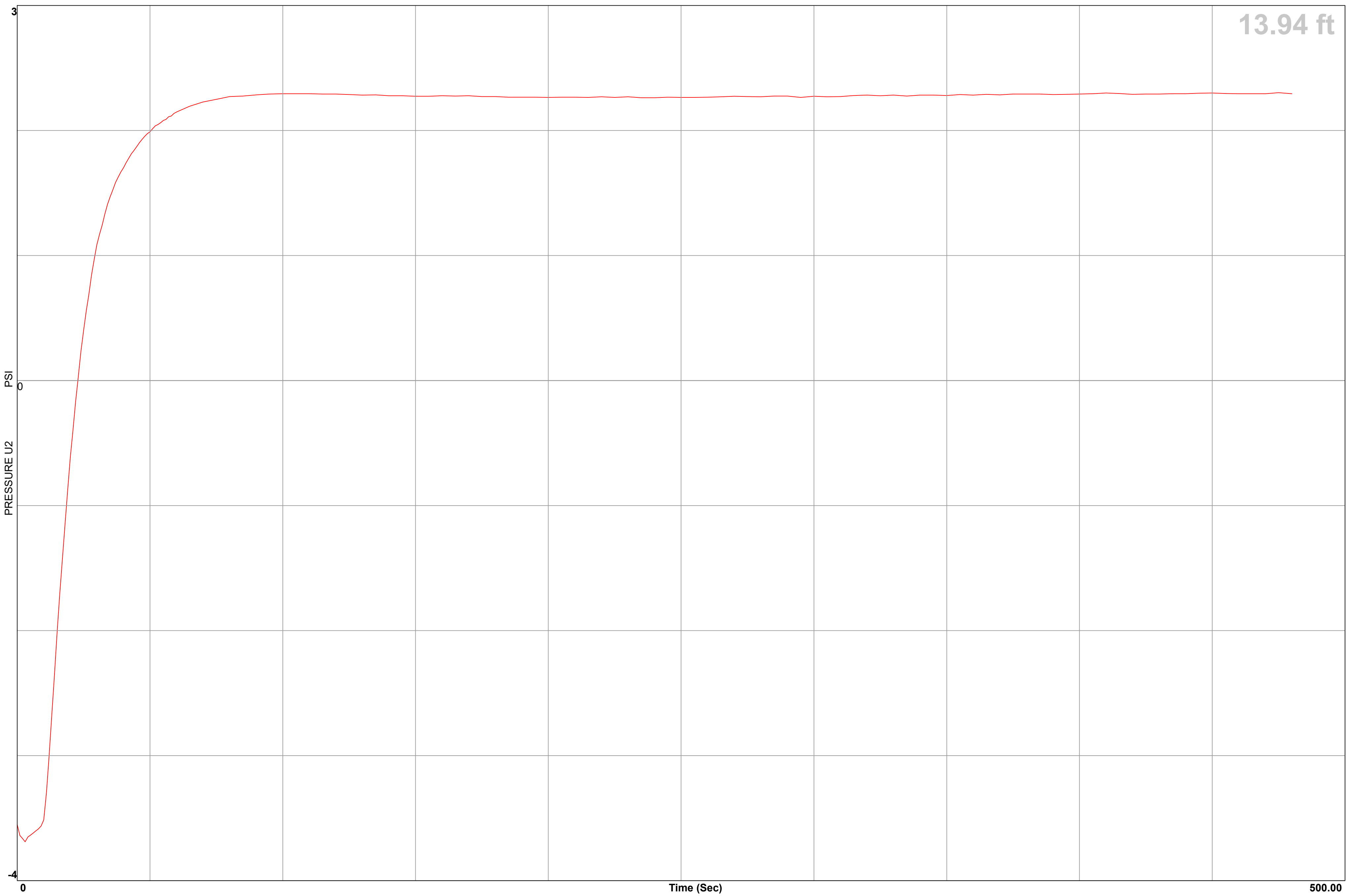
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

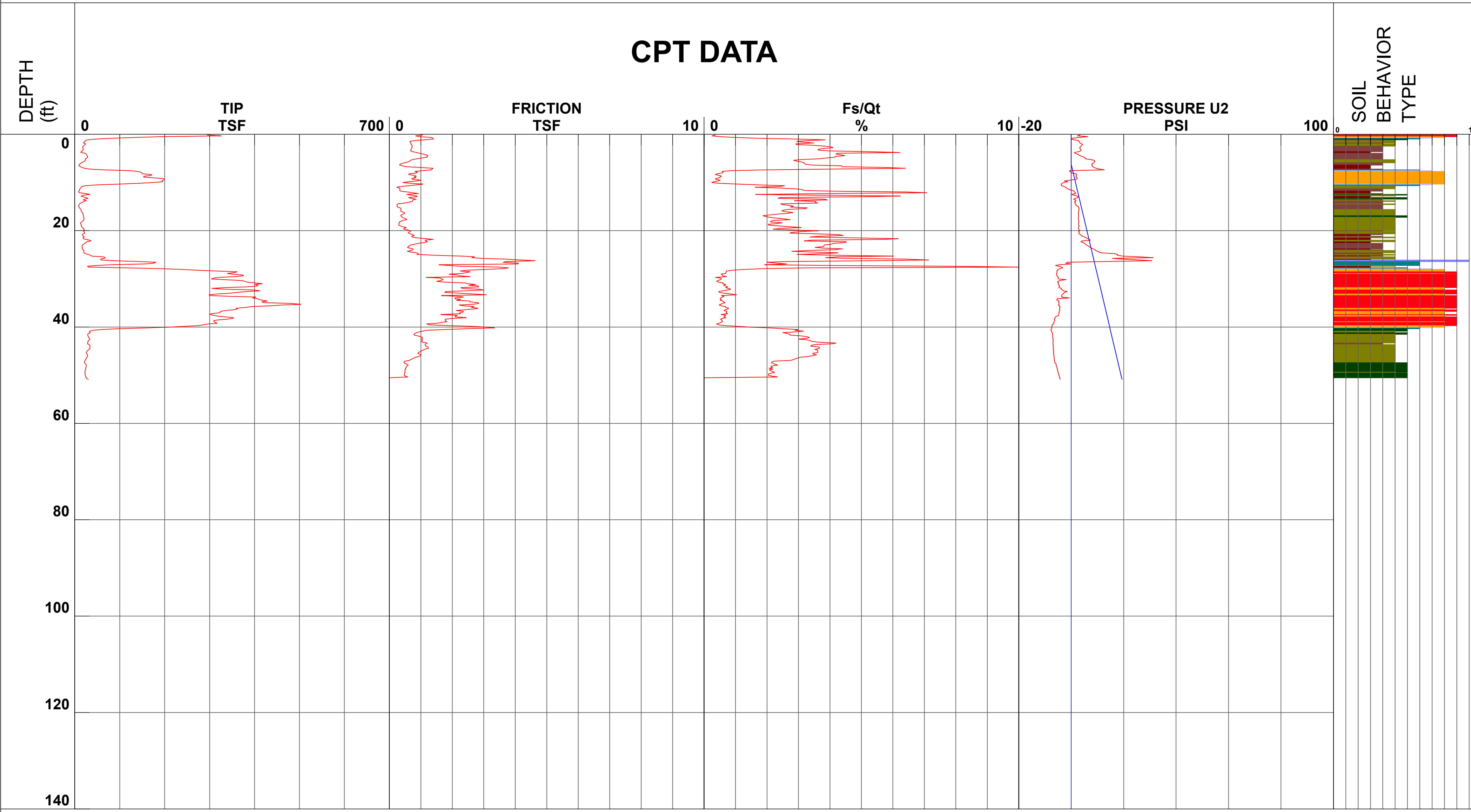
Location	2805 Bowers Ave	Operator	JM-GM		
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-04	Date and Time	2/1/2023 3:28:01 PM		
Equilized Pressure	2.2	EST GW Depth During Test	8.6		



Cornerstone Earth Group

Project	2805 Bowers Ave	Operator	JM-GM	Filename	SDF(843).cpt
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-05	Date and Time	1413-1-1	Maximum Depth	50.85 ft
EST GW Depth During Test		6.20 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	2805 Bowers Ave	Operator	JM-GM		
Job Number	1413-1-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-05	Date and Time	2/1/2023 2:34:20 PM		
Equilized Pressure	.7	EST GW Depth During Test	6.2