Appendix E: Geotechnical Study

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CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Preliminary Geotechnical Investigation
PROJECT NAME	River Oaks Residential
LOCATION	211 River Oaks Parkway San Jose, California
CLIENT	Valley Oak Partners, LLC
PROJECT NUMBER	384-16-1
DATE	June 19, 2023

CORNERSTONE
EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**River Oaks Residential**
Location	**211 River Oaks Parkway San Jose, California**
Client	**Valley Oak Partners, LLC**
Client Address	**734 The Alameda San Jose, California**
Project Number	**384-16-1**
Date	**June 19, 2023**

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Type of Services	**Preliminary Geotechnical Investigation**
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	San Jose, California

SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of Valley Oak Partners, LLC for the River Oaks Residential project in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of this study was to evaluate the existing subsurface conditions and develop a preliminary 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.

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 9.8 acre-site for residential use. Based on our conversations with you and review of preliminary conceptual plans, we understand the new project will include multi-family townhomes on the northern half of the site and two wrap podium apartment buildings on the southern half of the site. Based on our review of the conceptual plans provided, we understand the apartment buildings will consist of 5 to 6 levels of apartments overlying one to two levels of parking and the structures will total 5 to 7 stories. We anticipate the townhomes will be at-grade on the order of 2 to 4 stories. Appurtenant utilities, landscaping, storm water management areas, and other improvements necessary for overall site development will also be constructed.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated April 6, 2023 and consisted of a limited field program to evaluate physical and engineering properties of the subsurface soils, limited engineering analysis to prepare preliminary recommendations for site work, grading, building foundations, 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 June 2, 2023. The CPTs were advance to depths of approximately 50 to 100 feet. Seismic shear wave velocity measurements were collected from CPT-1.

The CPTs were backfilled with cement grout in accordance with Valley Water 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 site area range from about 500 to 700 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)
Hayward (Southeast Extension)	4.0	6.5
Hayward (Total Length)	6.6	10.7
Calaveras	9.3	15.0
Monte Vista-Shannon	9.9	16.0
San Andreas (1906)	13.4	21.7

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

SECTION 3: SITE CONDITIONS

3.1 SITE HISTORY AND SURFACE DESCRIPTION

The site consists of two parcels and is bounded by Cisco Way to the east, River Oaks Parkway to the south, Iron Point Drive to the west, and Anza Road and Levee Road to the north. The site is currently occupied by three office buildings with surrounding at-grade parking lot pavements, flatwork and landscaping.

We reviewed historic aerials and topographic maps on www.historicaerials.com from 1948 to 2020. A summary of pertinent historic surface changes observed on historic aerial photographs within the site vicinity is as follows:

- 1948: The general site vicinity including the project site appears to be used for agriculture purposes. Educational structures, currently named as Dolores Huerta Middle School and Kathleen Macdonald High School, are observed in the adjacent property to the northwest.
- 1980: The project site and surrounding area appears to be graded for development. The agriculture fields on the project site and surrounding area have been demolished. Grading operations for River Oak Parkway are also observed.
- 1982: The three existing office structures appear on-site. Industrial structures also appear on the adjacent western and eastern properties. River Oaks Parkway is also established.
- 1998 - 1999: The northeastern adjacent property appears to be graded for the Cisco Systems development and is established by the 1999 photo.
- 1999 – 2020: No pertinent changes are observed at and near the site from 1999 to 2020, the last available aerial image of the project site.

The site is currently occupied by three two-story office buildings and surrounding asphalt concrete parking lot. An at-grade courtyard consisting of flatwork and landscaping is located in the center of the three buildings. 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 buildings.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered interbedded layers of medium stiff to very 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 100 feet.

3.3 GROUNDWATER

Groundwater was inferred from CPT pore pressure dissipation tests at depths ranging from about 10 to 15½ 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 is mapped by the California Geologic Survey (CGS, Milpitas 7.5-Minute Quadrangle, 2001) at depths of approximately 5 to 10 feet below the existing ground surface in the site vicinity.

Based on the above, on a preliminary basis, we recommend a design groundwater depth of 7 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. 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 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 liquefaction analysis we assumed an exception and used a PGA_M of 0.735g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Milpitas Quadrangle, 2004) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our limited field program addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet and evaluating CPT data.

The potential for liquefaction should also be further evaluated at 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 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. 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 and CPT-5) are presented on Figures 4A and 4E of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from about ⅓ to 2⅔ 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 up to 1¾ inches over a horizontal distance of 30 to 40 feet.

The potential for liquefaction should be further evaluated during 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 7-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.

This concern should be further evaluation during the design-level geotechnical investigation.

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 above the design groundwater depth based on the work by Robertson and Shao (2010). Based on our preliminary analyses, the potential for significant seismic dry sand settlement affecting the proposed improvements is low.

This concern should be further evaluation during the design-level geotechnical investigation.

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 3 miles inland from the San Francisco Bay shoreline and is approximately 21 to 27 feet above mean sea level. In addition, the site is mapped by the State of California as being outside a tsunami hazard area (CGS, 2021). 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 Coyote and Leroy Anderson Reservoirs.

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.

- Strong ground shaking
- Potential for significant static and seismic settlements
- Shallow groundwater
- Presence of moderately to highly expansive soils
- Redevelopment 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 and Seismic Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ⅓ to 2⅔ inches could occur, resulting in differential settlement up to about 1¾ inches. However, additional site-specific subsurface explorations and settlements estimates should be performed and evaluated during a design-level geotechnical investigation.

In addition, 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 building loads for the proposed 5- to 7-story podium apartment structures and anticipated subsurface conditions, we estimate large static and long-term consolidation settlements may occur over the design life of the structure. Therefore, on a preliminary basis, based on our engineering judgment, experience with similar projects in the vicinity, and the subsurface conditions, 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 10 to 15½ feet below the existing ground surface. As discussed above, on a preliminary basis we recommend a design groundwater depth of 7 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 highly 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 three two-story office buildings 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, Milpitas 7.5-Minute Quadrangle, 2001) indicate the high groundwater to be approximately 5 to 10 feet below the existing ground surface in the site vicinity. On a preliminary basis, we used a design groundwater depth of 7 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 understand 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 100 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-1 to a depth of 100 feet, resulting in a time-averaged shear wave velocity for the top 100 feet (V_S) of 228 meters per second (748 feet per second). Therefore, 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 web-based 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 may 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	37.404966°
Site Longitude	-121.930522°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.587g
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}	1.587g
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}	1.058g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.020g
Site Amplification Factor at PGA – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.735g

Because the potential for liquefaction and the potential for affects to the structure appear high, based on ASCE 7-16, on a preliminary basis, the site should be classified as Site Class F. ASCE 7-16 generally indicates that sites classified as Site Class F shall have a site response analysis performed in accordance with Section 21.1 of ASCE 7-16, unless the proposed structure meets the following exception.

> **EXCEPTION:** For structures having 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.

As discussed, on a preliminary basis, we have assumed an exception will be taken; therefore, the above Site Classification of D in Table 2 of this report, and the presented seismic coefficients, appear valid. The Project Structural Engineer should verify this assumption. If the structure will have a fundamental period of greater than 0.5 seconds, and meets the requirements for a Site Class designation of F, the requirement for a site response analysis will be triggered, and additional geotechnical analysis will need to be approved. However, if ground improvement is implemented as recommended in the “Foundations” section below to mitigate potential seismic settlements, the site may also be classified as Soil Classification D. We recommend the Soil Classification and Seismic Design Criteria be further evaluated and confirmed during the design-level investigation.

SECTION 8: FOUNDATIONS

On a preliminary basis, proposed structures may be supported on shallow mat foundations provided they can be designed to tolerate anticipated total and differential settlements (seismic and static). As an alternative, or if it is determined that the total and differential settlements exceed tolerable limits due to the anticipated significant total and differential settlement, the proposed structures 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.

8.1 SHALLOW MAT FOUNDATIONS

Provided the mat foundation can be designed to tolerate total and differential settlements, the proposed structures may be supported on a conventionally reinforced mat foundation. Mats should be designed in accordance with the current California Building Code.

On a preliminary basis, to reduce potential differential movement, mats should be designed for a maximum average areal bearing pressure of 750 psf for dead plus live loads; at column or wall loading, the maximum localized allowable bearing pressure should be limited to about 2,000 psf. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. Additional reinforcing steel may be required to help span irregularities and differential settlement.

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 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is 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), deep soil mixing, 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. Based on our CPT explorations, we expect ground improvement may extend about 30 to 35 feet below the ground surface.

8.3 DEEP FOUNDATIONS

On a preliminary basis, as an alternative to mat foundations or shallow foundations overlying ground improvement, the proposed structures 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 can be provided in a design-level report.

SECTION 9: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Valley Oak Partners, LLC specifically to support the design of the River Oaks Residential project in San Jose, 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.

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

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

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

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

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

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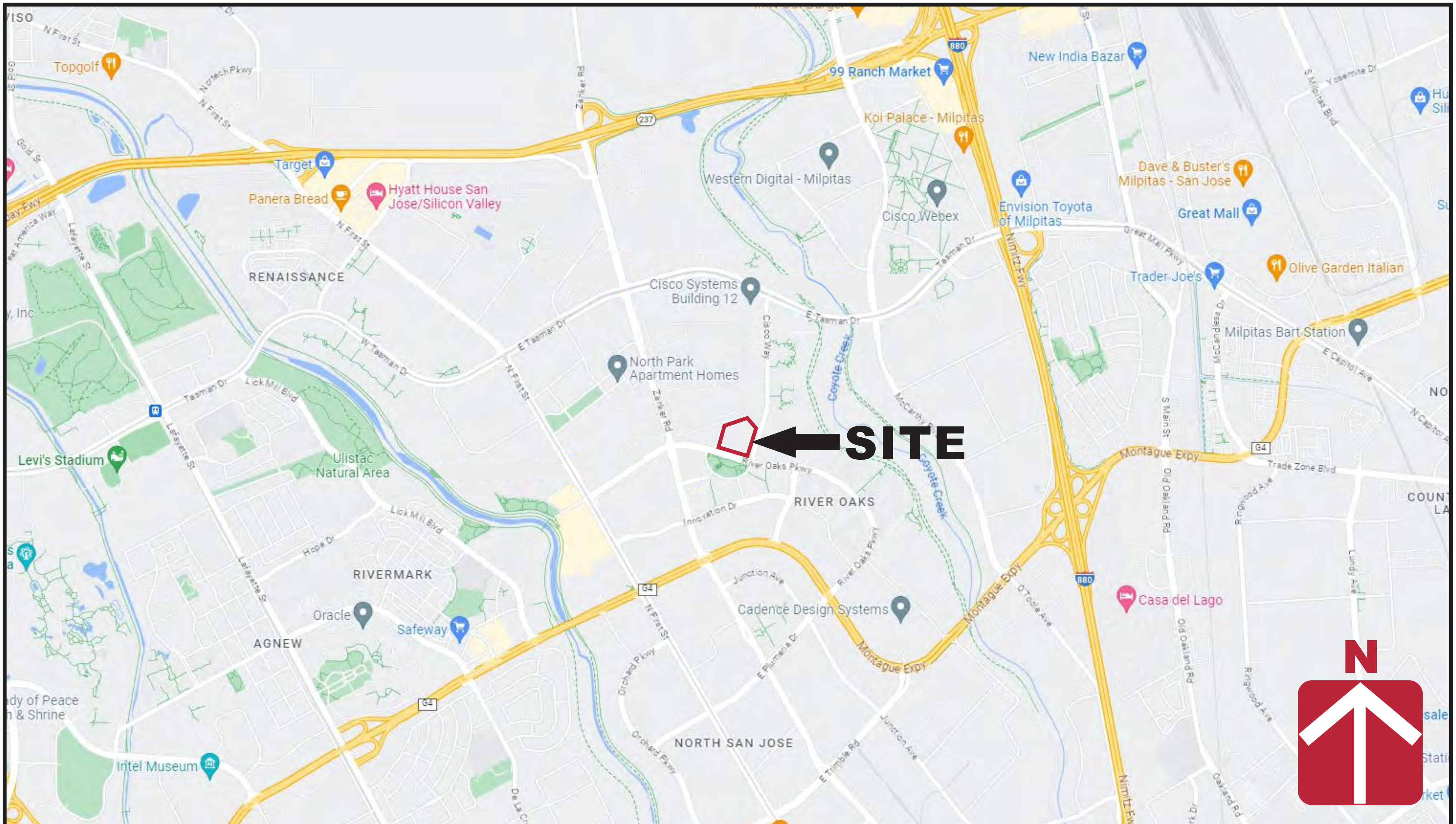

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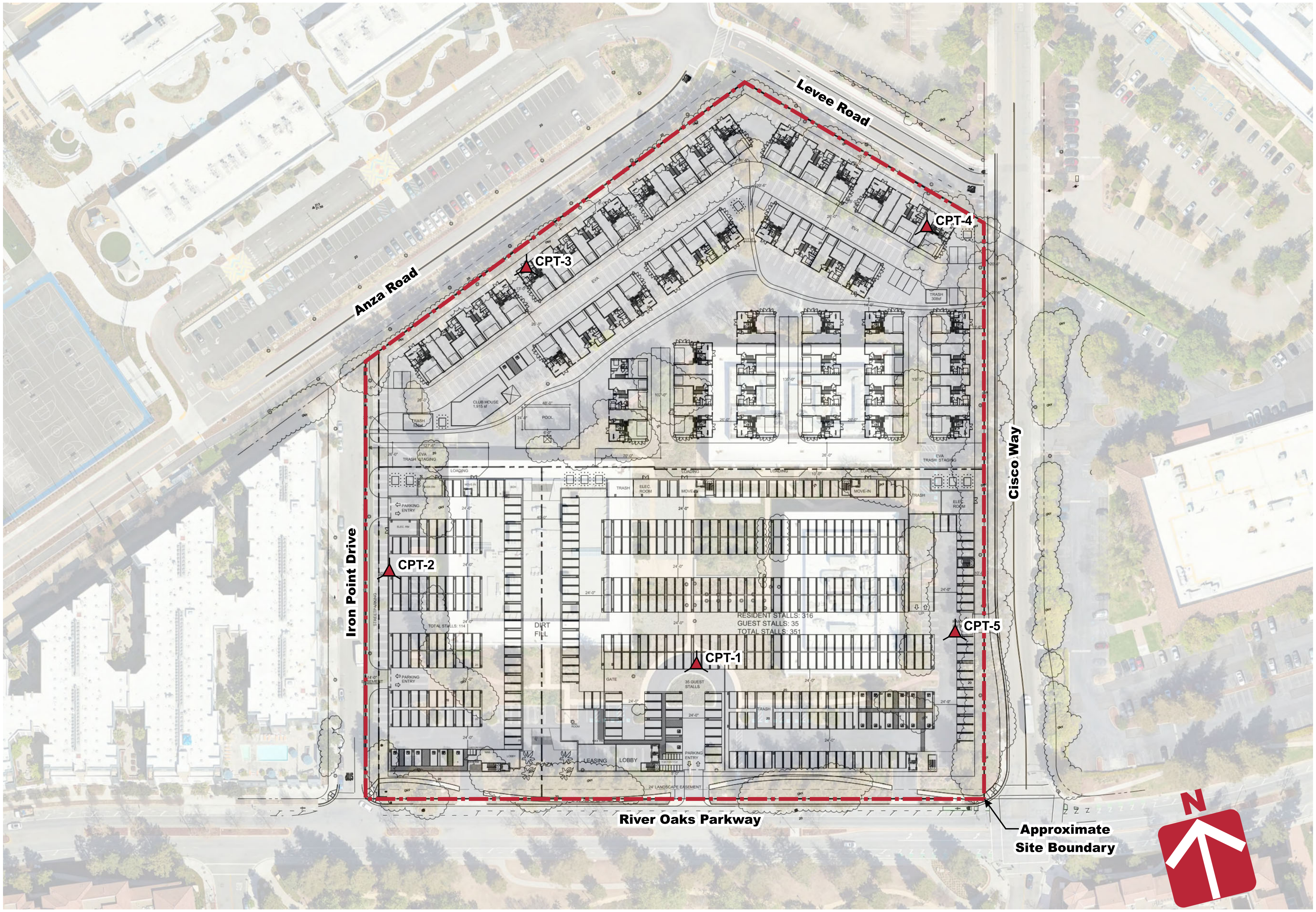
Vicinity Map

River Oaks Residential
211-251 River Oaks Parkway
San Jose, CA

Project Number	384-16-1
Figure Number	Figure 1
Date	June 2023
Drawn By	RRN



Legend

▲ Approximate location of cone penetration test (CPT)

0 100 200

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 03/10/2022

Overlay by Studio T Square, Floor Plan: Level 1, dated 06/30/2023

Project Number: 384-16-1

Figure Number: Figure 2

Date: June 2023

Drawn By: RRN

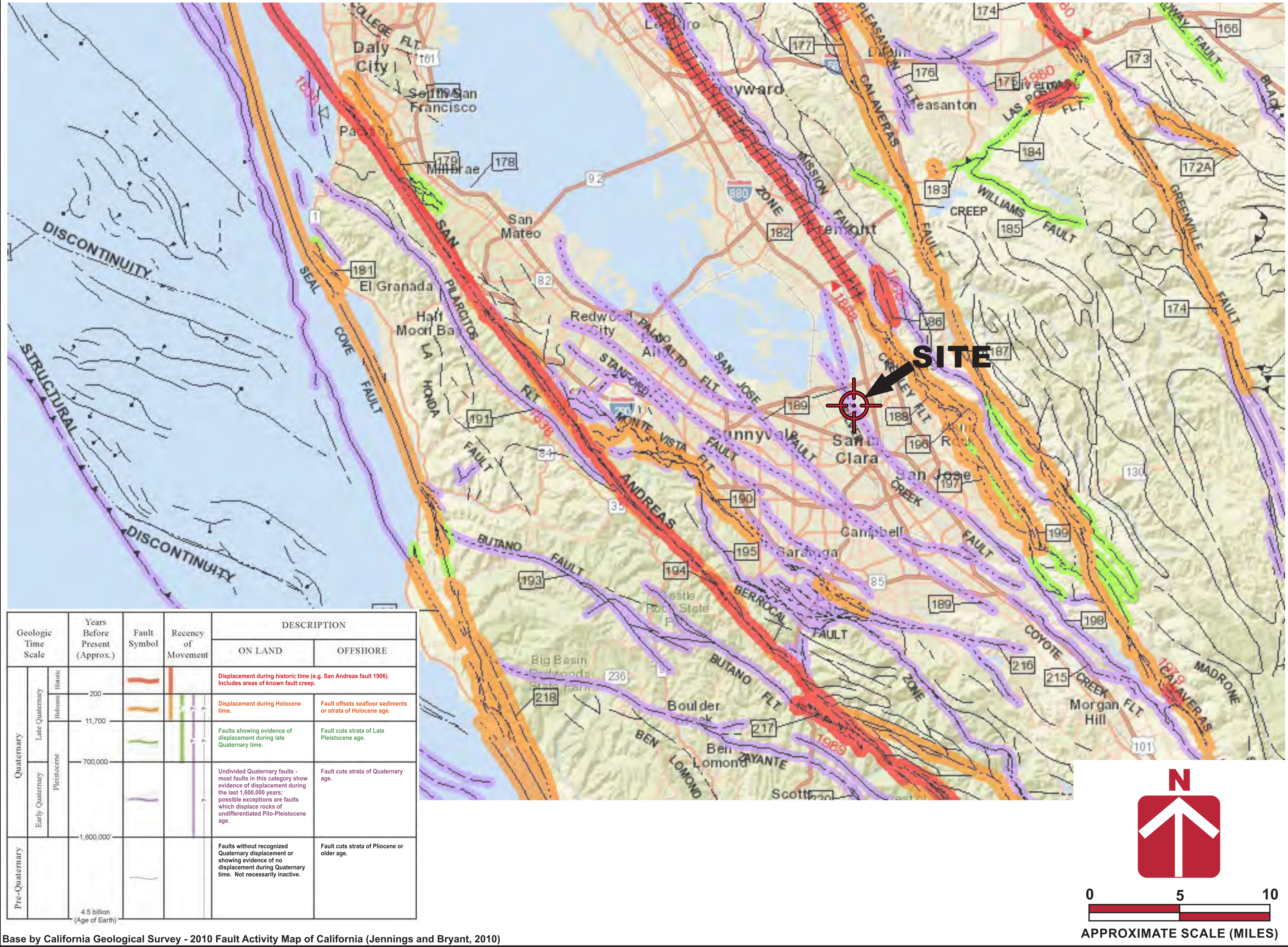
Site Plan

River Oaks Residential

211-251 River Oaks Parkway

San Jose, 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 Plio-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 384-16-1

Figure Number Figure 3

Date June 2023

Drawn By RRN

Regional Fault Map

River Oaks Residential
211-251 River Oaks Parkway
San Jose, CA

CORNERSTONE EARTH GROUP

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FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	River Oaks Residential
Project No.	384-16-1
Project Manager	MFR

SEISMIC PARAMETERS

Controlling Fault	Hayward	
Earthquake Magnitude (Mw)	7.08	
PGA (Amax)	0.735	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	12.3
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

0.02 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.36 (Inches)

TOTAL SEISMIC SETTLEMENT 1.4 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 280.0

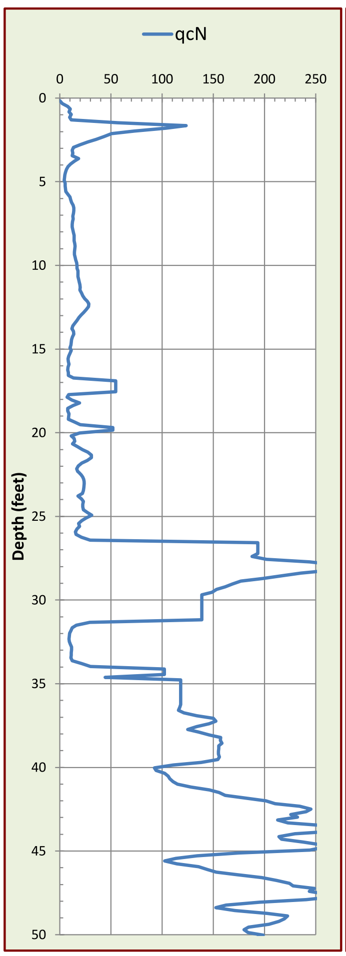
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

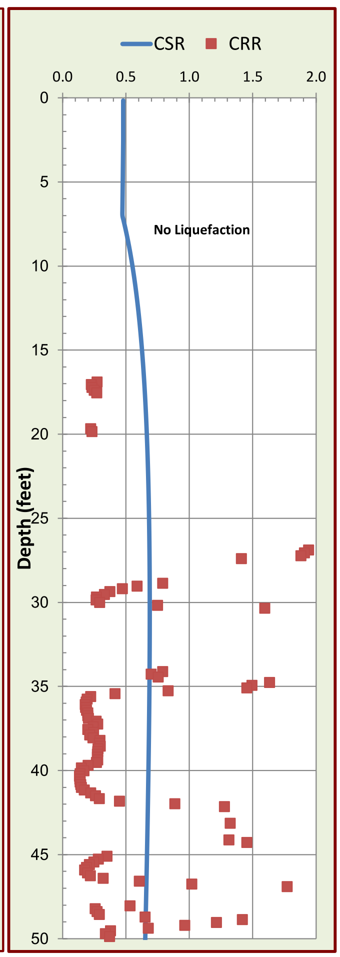
EXPECTED RANGE OF DISPLACEMENT

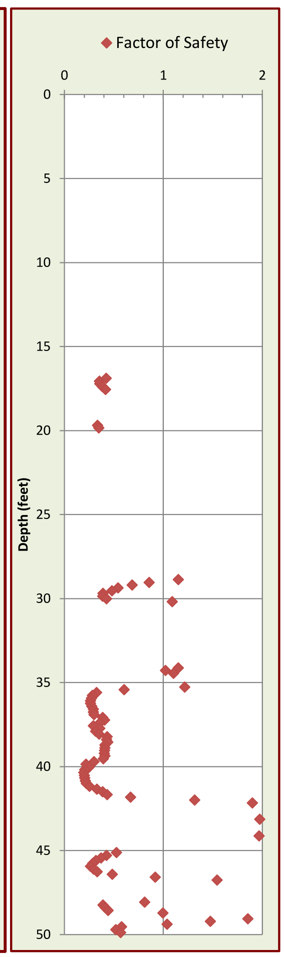
0.0 to 0.0 feet

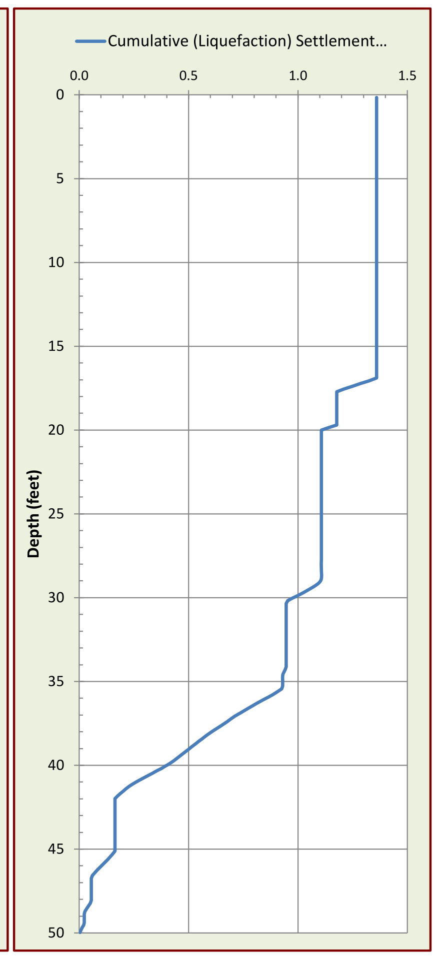
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title	**River Oaks Residential**
Project No.	**384-16-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.735**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.7**
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**
	0.00	**(Inches)**
LIQUEFACTION SETTLEMENT FROM	50	**FEET**
	2.64	**(Inches)**
TOTAL SEISMIC SETTLEMENT	2.6	**INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 280.0

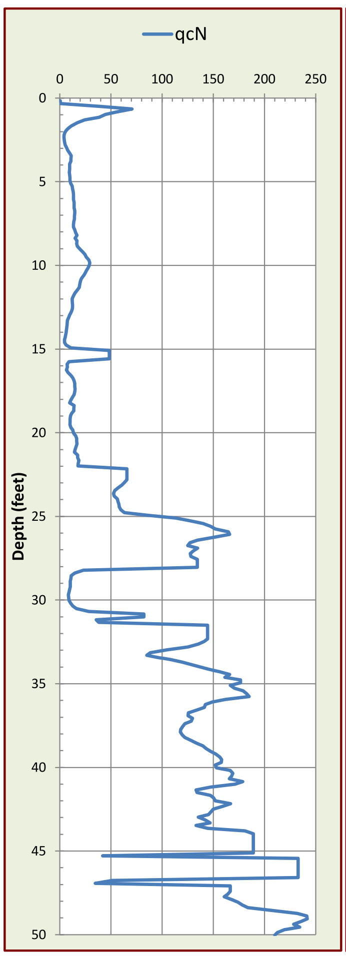
LDI[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

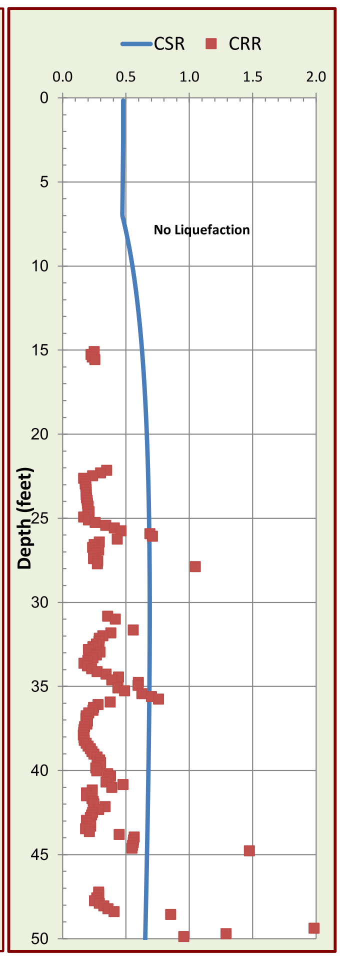
EXPECTED RANGE OF DISPLACEMENT

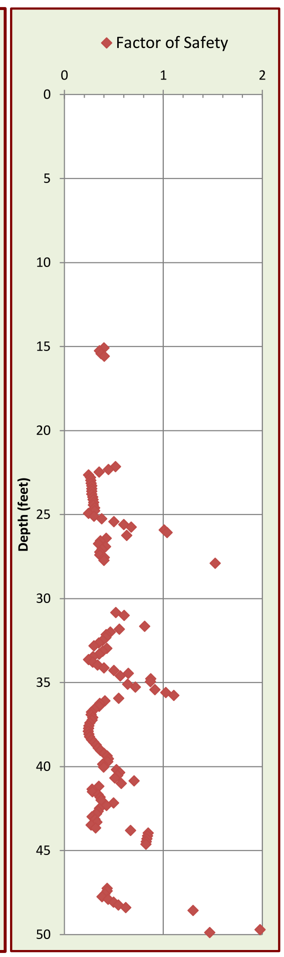
0.0 **to** **0.0** **feet**

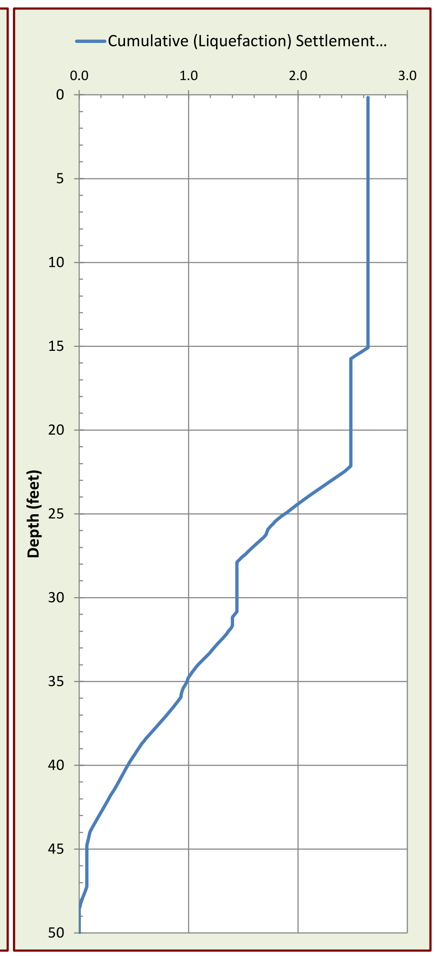
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title	**River Oaks Residential**
Project No.	**384-16-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**
Earthquake Magnitude (Mw)	**7.08**
PGA (Amax)	**0.735** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.6**
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**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.52 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 280.0

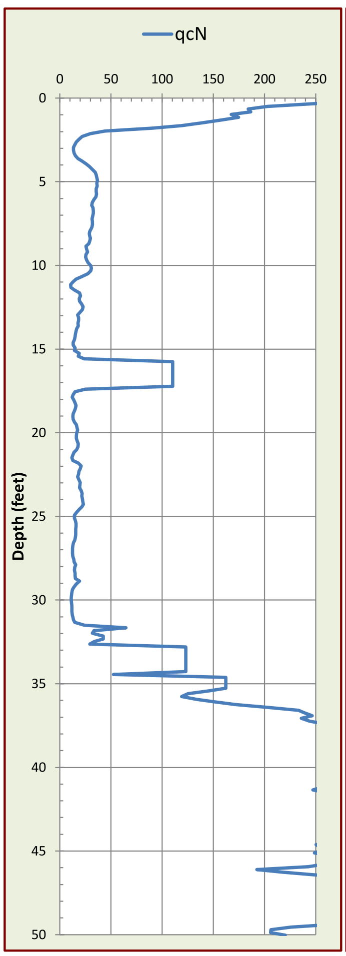
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

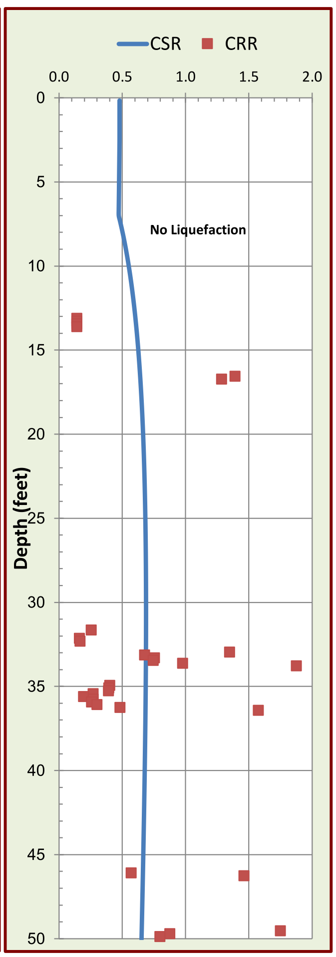
EXPECTED RANGE OF DISPLACEMENT

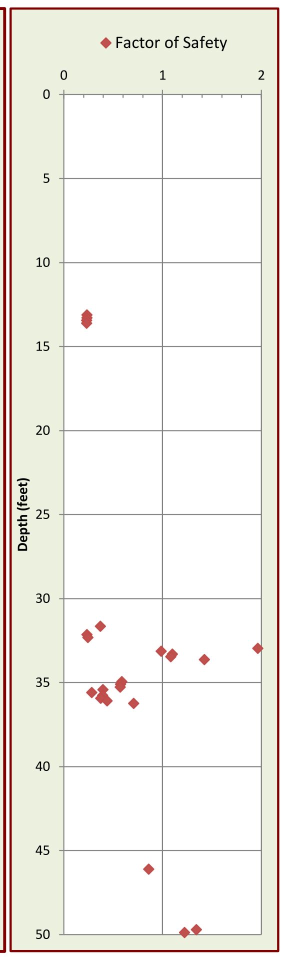
0.0 **to** **0.0** **feet**

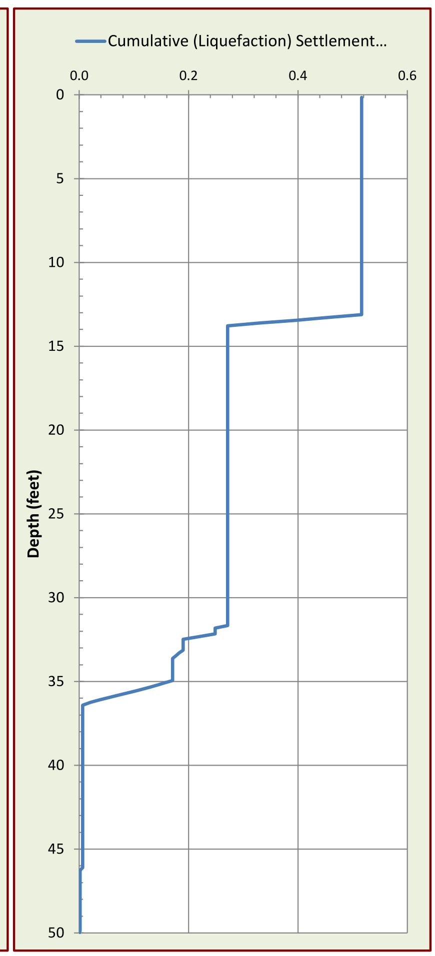
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title: **River Oaks Residential**

Project No.: **384-16-1**

Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **Hayward**

Earthquake Magnitude (Mw): **7.08**

PGA (Amax): **0.735** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **15.4**

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**

0.06 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.38 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 280.0

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

EXPECTED RANGE OF DISPLACEMENT

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

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

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

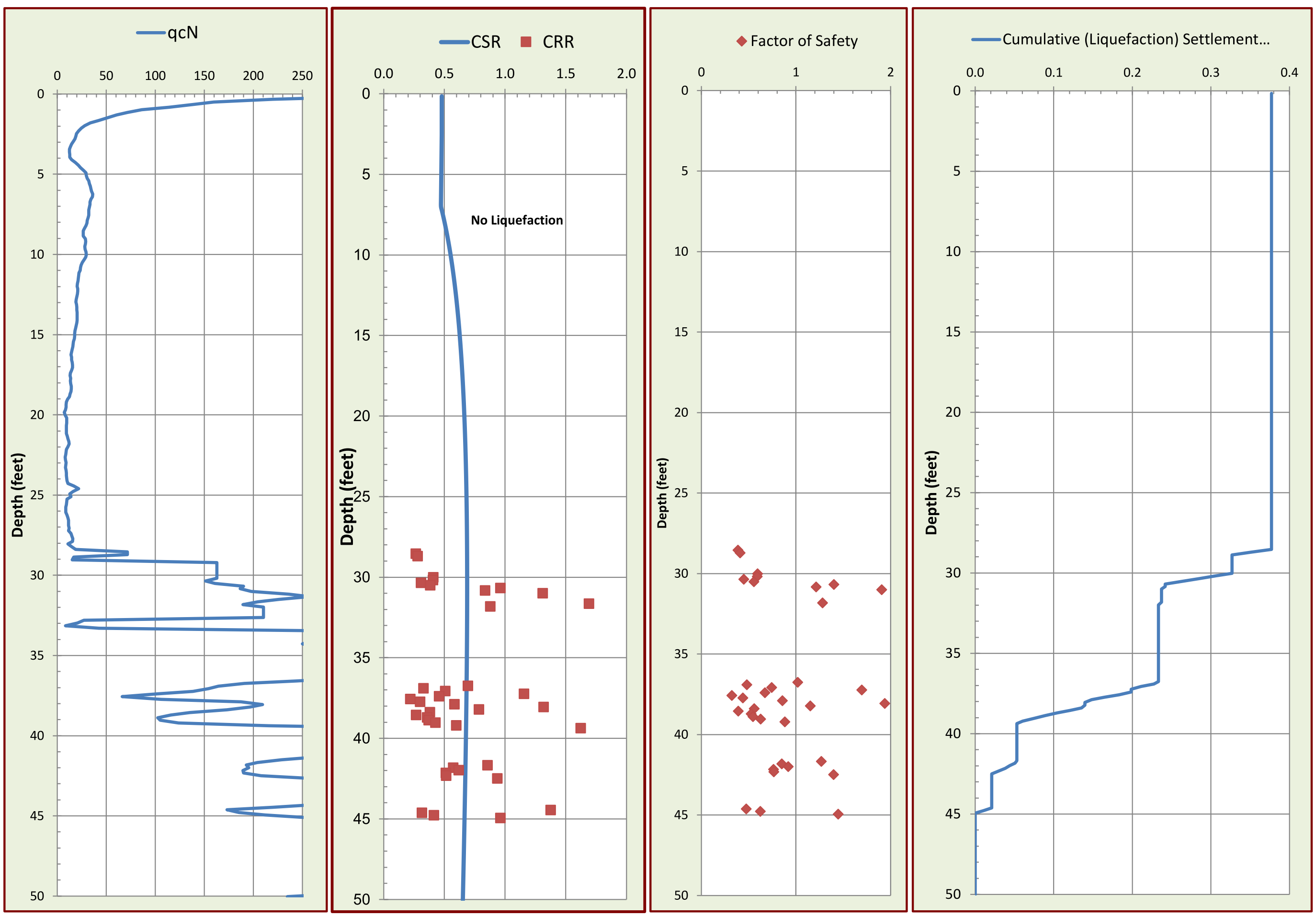


FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title	**River Oaks Residential**
Project No.	**384-16-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**
Earthquake Magnitude (Mw)	**7.08**
PGA (Amax)	**0.735** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.2**
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**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

1.46 **(Inches)**

TOTAL SEISMIC SETTLEMENT 1.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

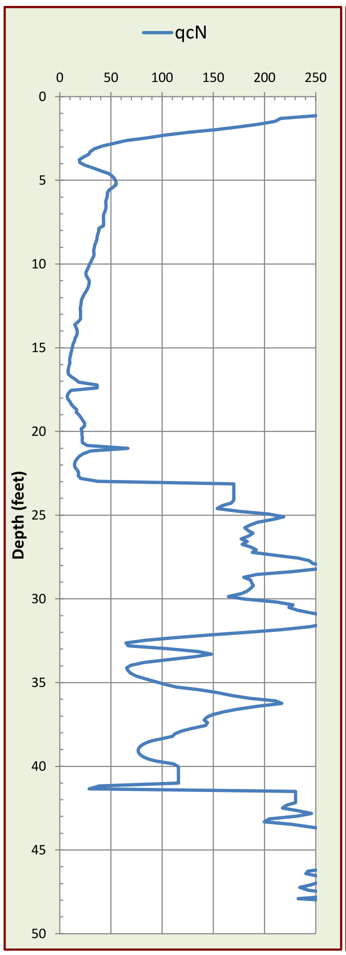
LDI[2] 0.09 **L/H** 280.0

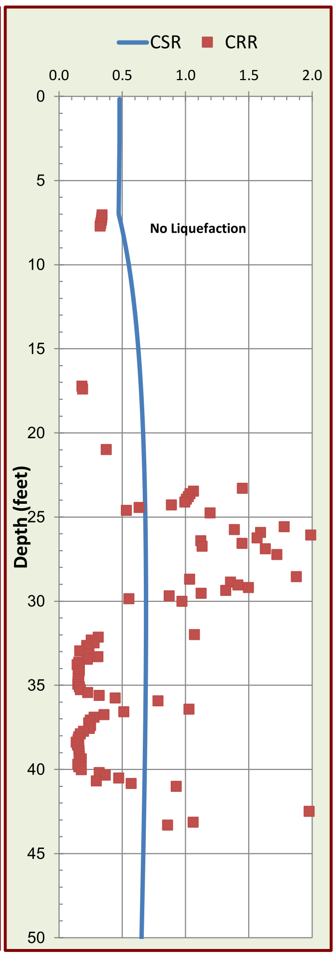
LDI[1]**Corrected for Distance** 0.01 (4 < L/H < 40)

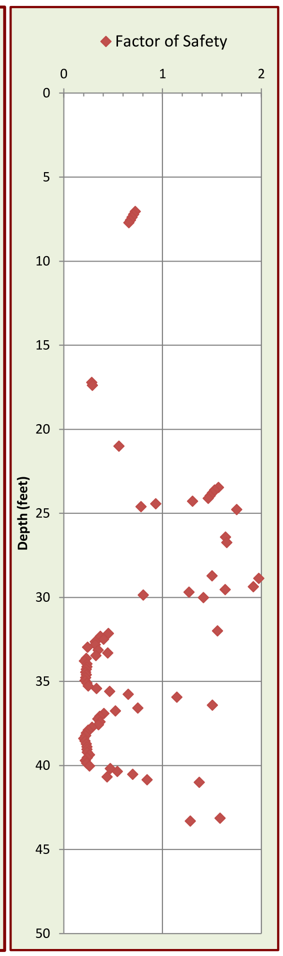
EXPECTED RANGE OF DISPLACEMENT

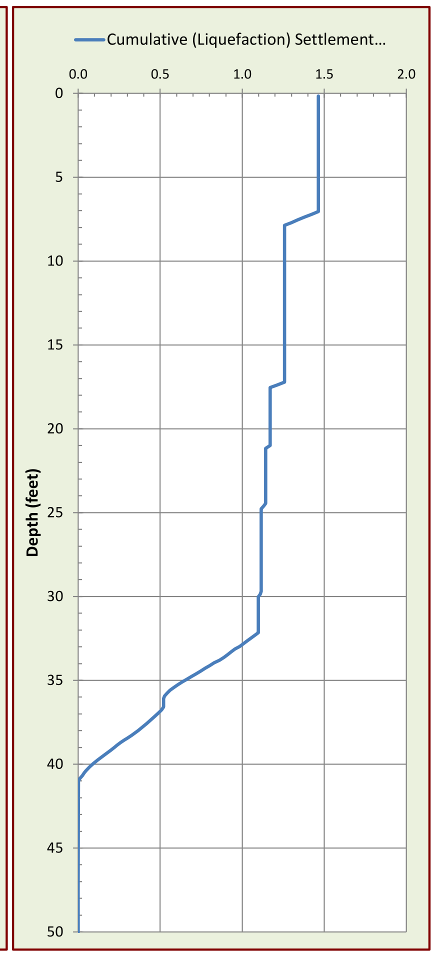
0.0 **to** 0.0 **feet**

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









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using 25-ton truck-mounted Cone Penetration Test equipment. Five CPT soundings were performed in accordance with ASTM D 5778-95 (revised, 2002) on June 2, 2023, to depths ranging from approximately 50 to 100 feet. The approximate locations of the CPTs are shown on the Site Plan, Figure 2.

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.

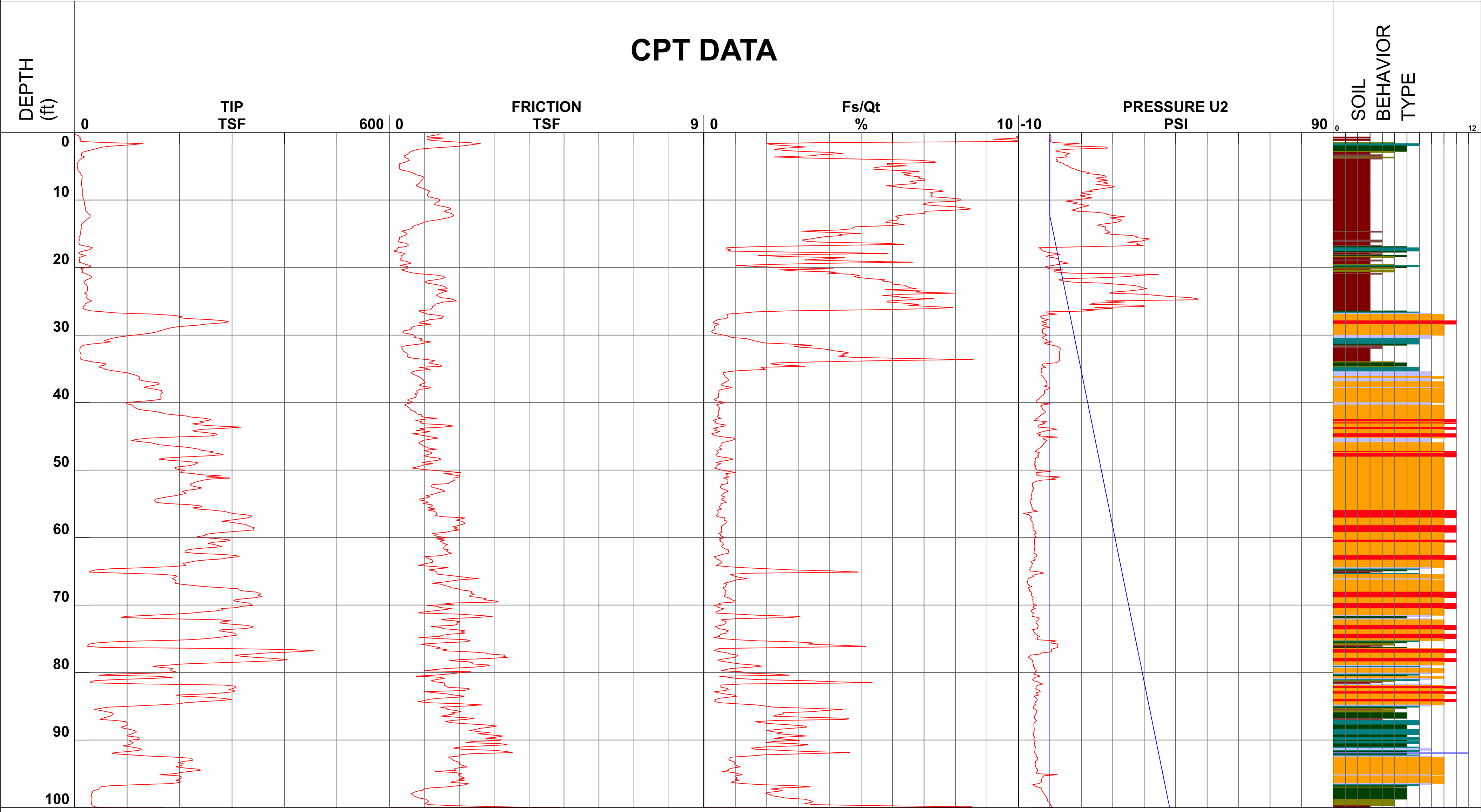
Attached CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these CPT locations. The passage of time may result in altered subsurface conditions 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	211-215 River Oaks Parkway GI	Operator	AJ-ER	Filename	SDF(736).cpt
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	6/2/2023 8:43:01 AM	Maximum Depth	100.72 ft
EST GW Depth During Test		12.30 ft			

Net Area Ratio .8

CPT DATA



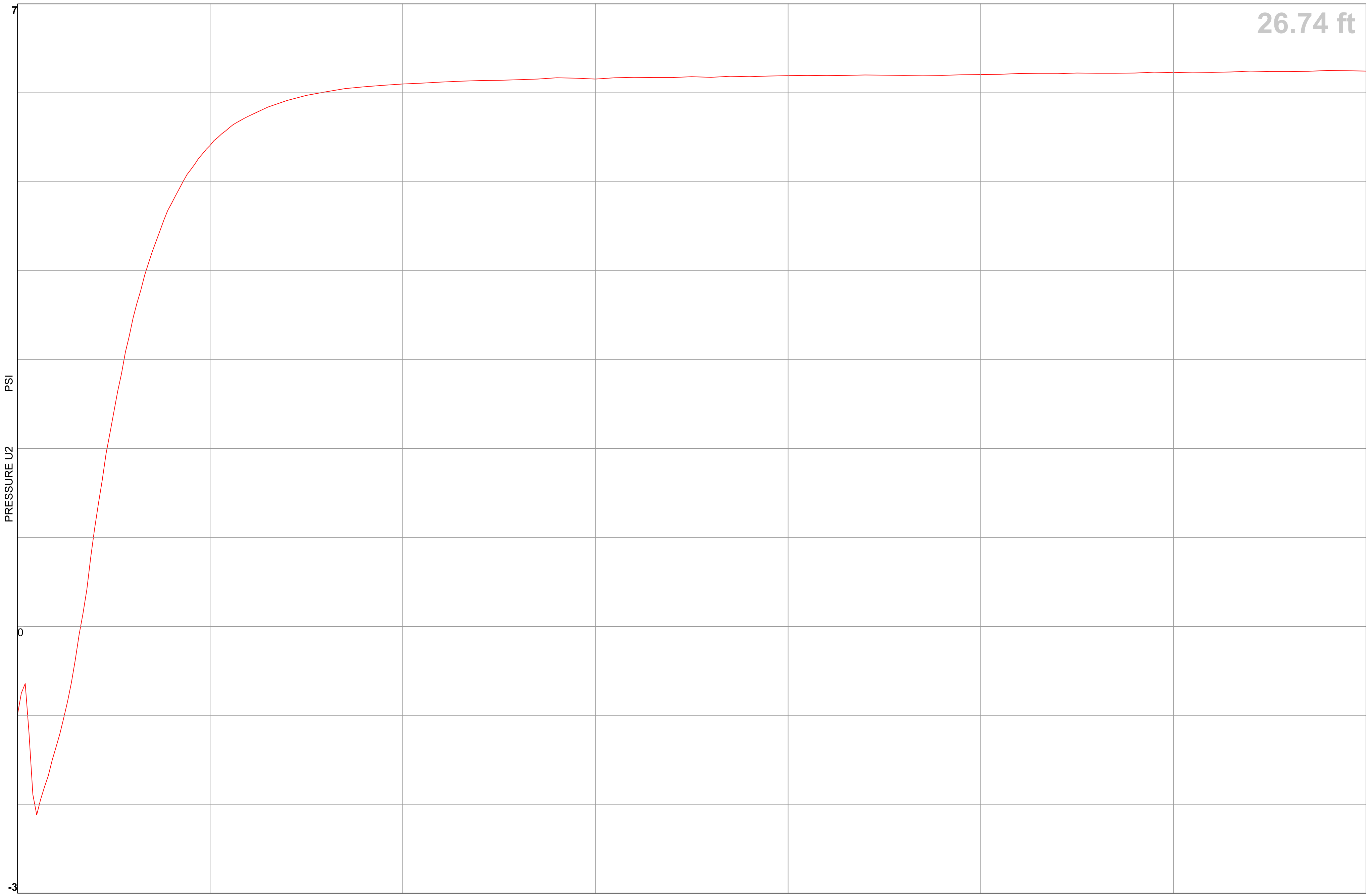
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

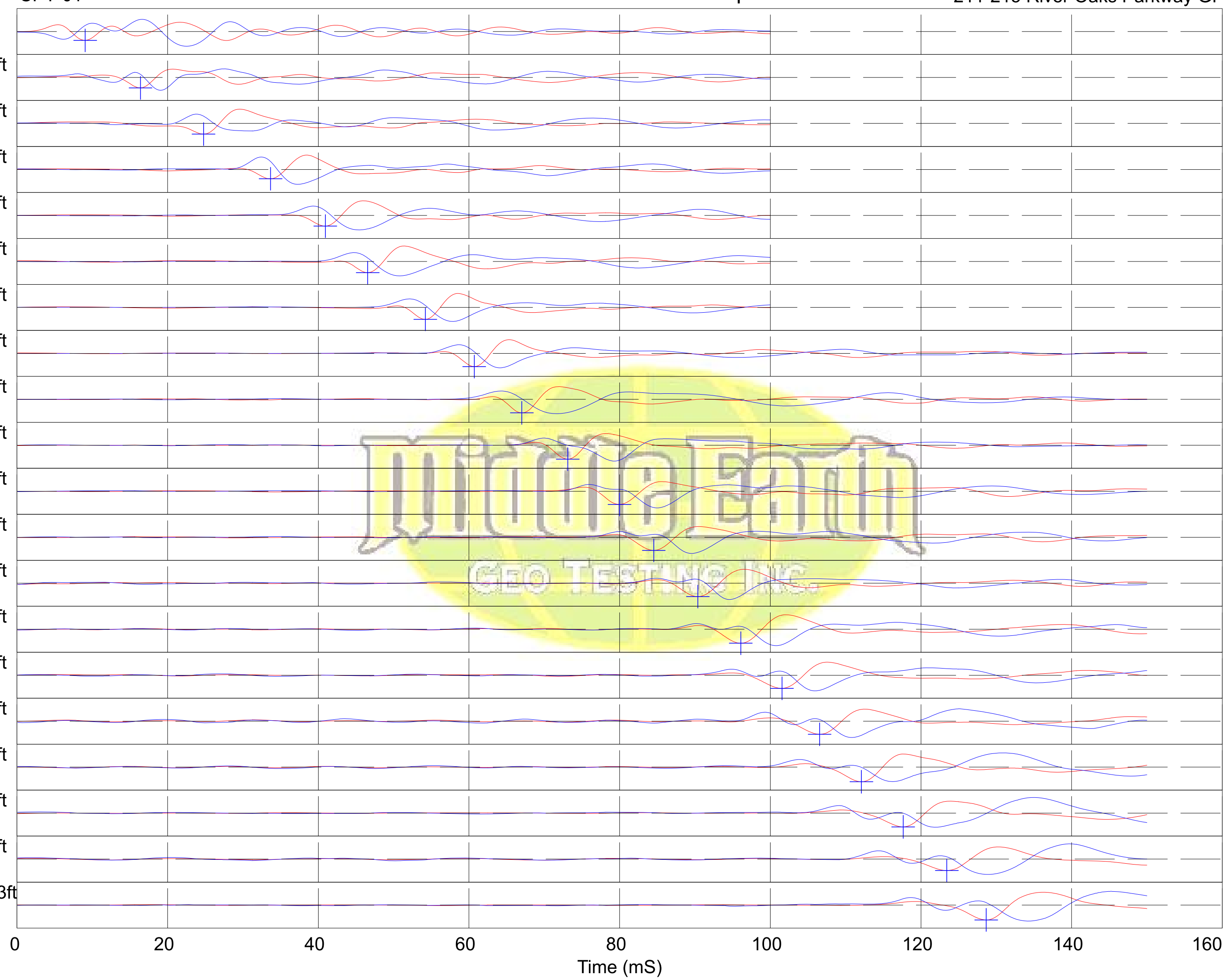
Location	211-215 River Oaks Parkway GI	Operator	AJ-ER		
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	6/2/2023 8:43:01 AM		
Equilized Pressure	6.2	EST GW Depth During Test	12.3		



CPT-01

Cornerstone Earth Group

211-215 River Oaks Parkway GI



Depth 4.99ft	Ref*	Arrival 9.06mS	Velocity*
Depth 10.10ft	Ref 4.99ft	Arrival 16.41mS	Velocity 543.82ft/S
Depth 15.03ft	Ref 10.10ft	Arrival 24.76mS	Velocity 532.49ft/S
Depth 20.18ft	Ref 15.03ft	Arrival 33.67mS	Velocity 548.49ft/S
Depth 25.43ft	Ref 20.18ft	Arrival 40.93mS	Velocity 699.72ft/S
Depth 30.02ft	Ref 25.43ft	Arrival 46.56mS	Velocity 799.01ft/S
Depth 35.17ft	Ref 30.02ft	Arrival 54.22mS	Velocity 662.23ft/S
Depth 40.03ft	Ref 35.17ft	Arrival 60.70mS	Velocity 739.98ft/S
Depth 45.01ft	Ref 40.03ft	Arrival 67.03mS	Velocity 780.76ft/S
Depth 50.03ft	Ref 45.01ft	Arrival 73.12mS	Velocity 817.64ft/S
Depth 55.94ft	Ref 50.03ft	Arrival 79.99mS	Velocity 853.86ft/S
Depth 60.04ft	Ref 55.94ft	Arrival 84.53mS	Velocity 900.57ft/S
Depth 65.03ft	Ref 60.04ft	Arrival 90.38mS	Velocity 847.46ft/S
Depth 70.05ft	Ref 65.03ft	Arrival 96.09mS	Velocity 876.95ft/S
Depth 75.20ft	Ref 70.05ft	Arrival 101.56mS	Velocity 938.91ft/S
Depth 80.05ft	Ref 75.20ft	Arrival 106.56mS	Velocity 968.46ft/S
Depth 85.04ft	Ref 80.05ft	Arrival 112.10mS	Velocity 896.86ft/S
Depth 90.06ft	Ref 85.04ft	Arrival 117.65mS	Velocity 903.01ft/S
Depth 95.05ft	Ref 90.06ft	Arrival 123.43mS	Velocity 860.94ft/S
Depth 100.13ft	Ref 95.05ft	Arrival 128.66mS	Velocity 969.85ft/S

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

COMMENT:

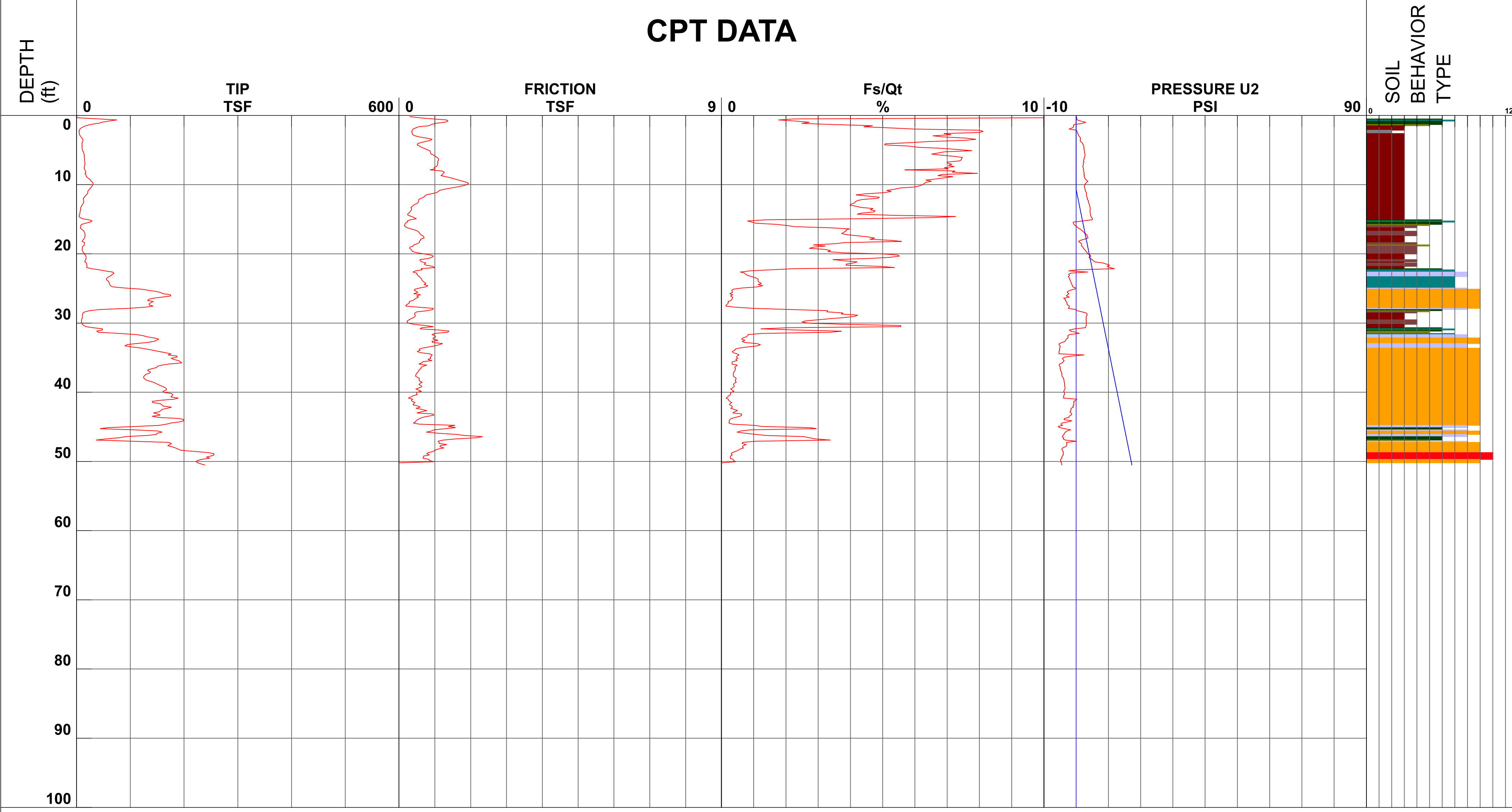
Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project	211-215 River Oaks Parkway GI	Operator	AJ-ER	Filename	SDF(737).cpt
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	6/2/2023 10:32:28 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		10.70 ft			

Net Area Ratio .8

CPT DATA



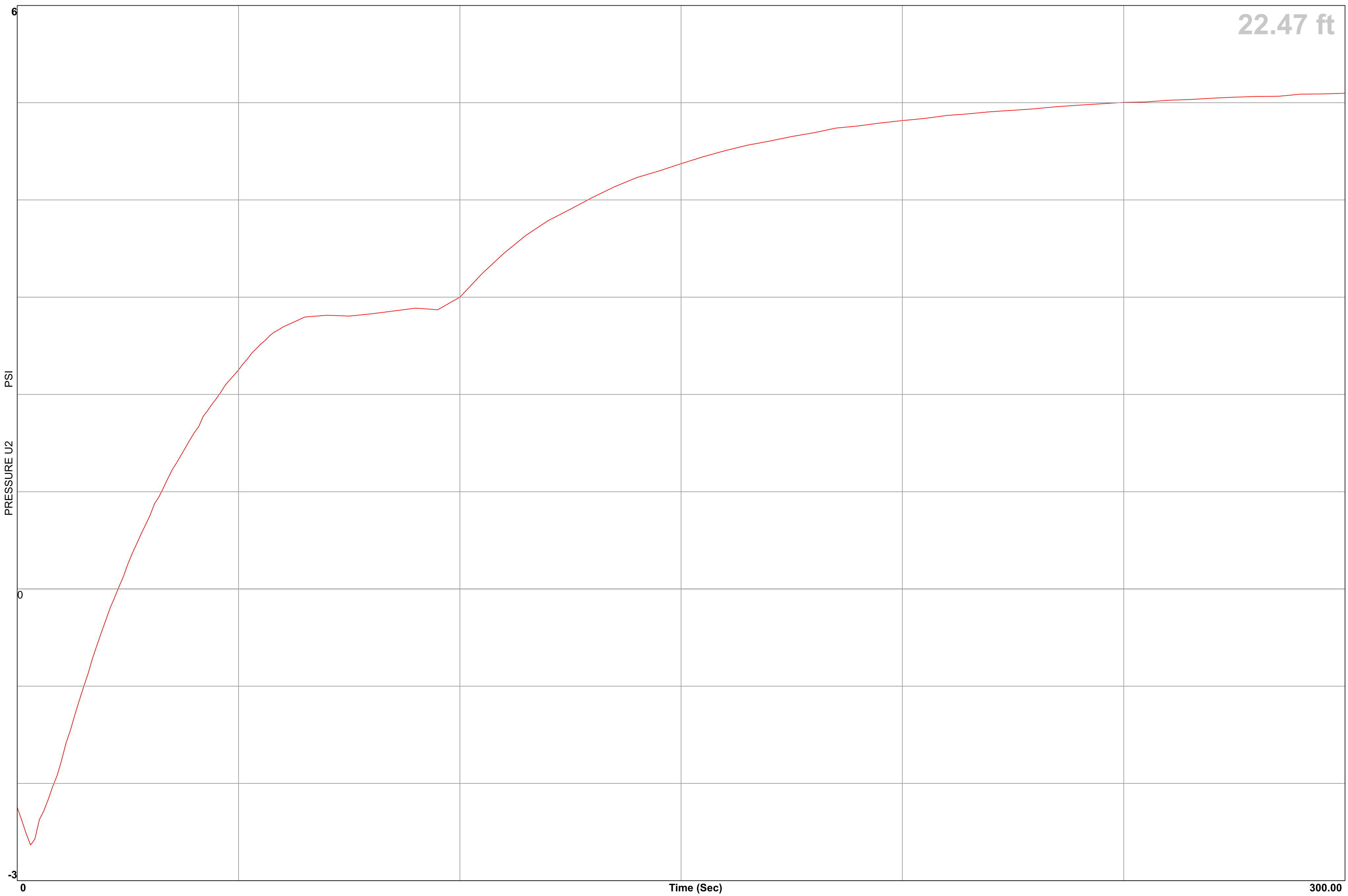
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	211-215 River Oaks Parkway GI	Operator	AJ-ER		
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	6/2/2023 10:32:28 AM		
Equilized Pressure	5.0	EST GW Depth During Test	10.7		

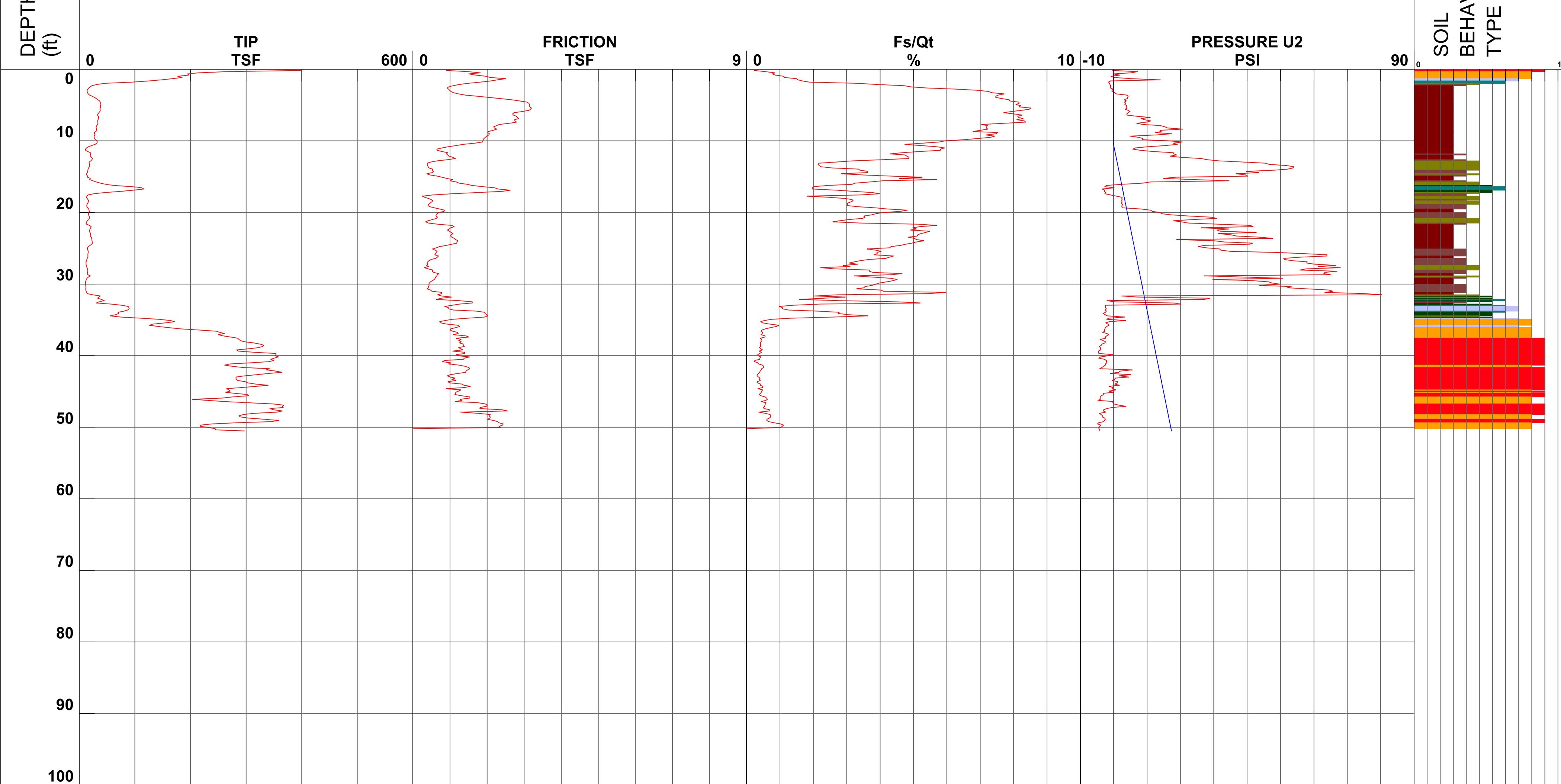


Cornerstone Earth Group

Project	211-215 River Oaks Parkway GI	Operator	AJ-ER	Filename	SDF(738).cpt
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	6/2/2023 12:08:20 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		10.60 ft			

Net Area Ratio .8

CPT DATA



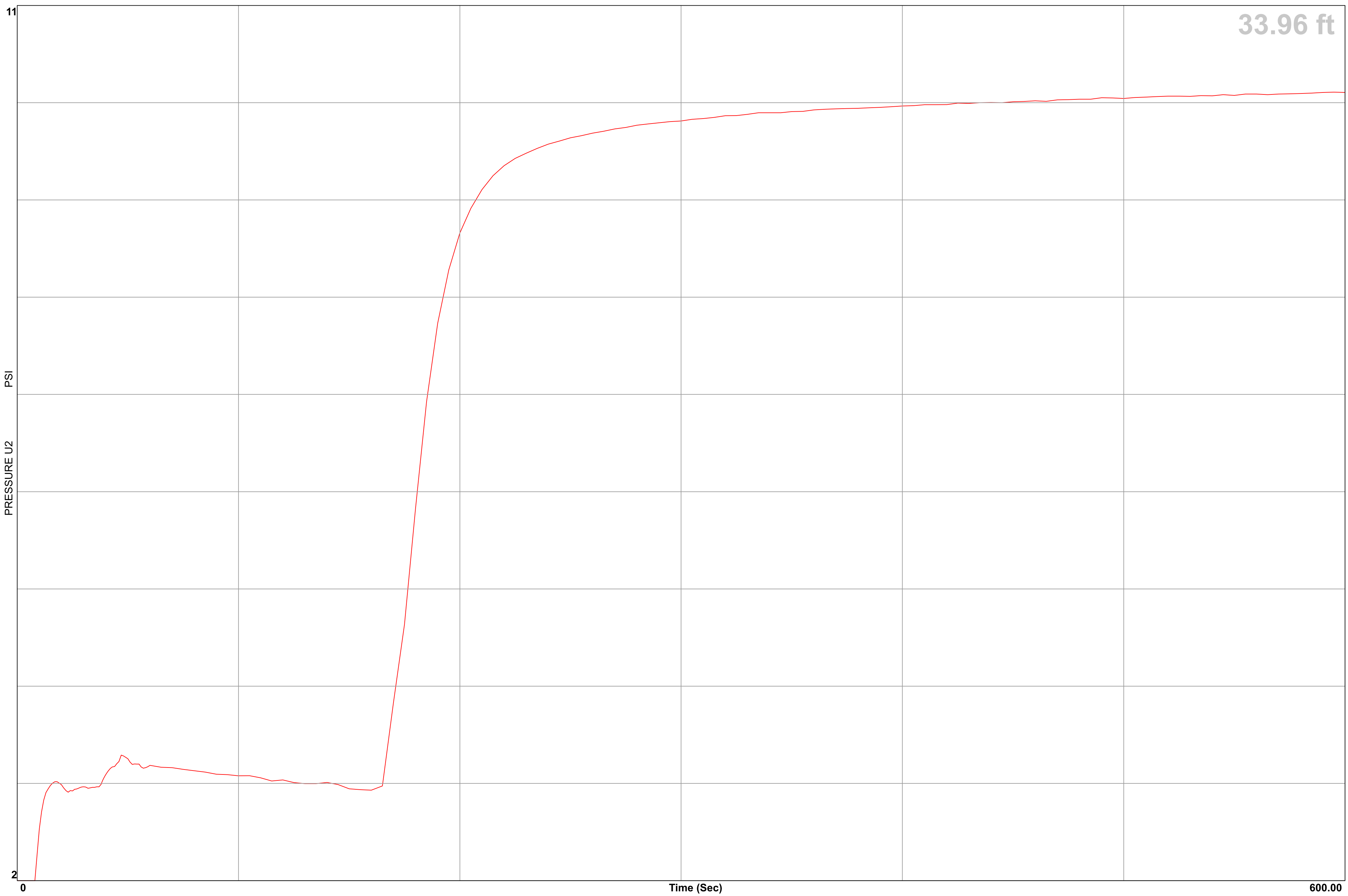
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	211-215 River Oaks Parkway Gl	Operator	AJ-ER		
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	6/2/2023 12:08:20 PM		
Equilized Pressure	10.0	EST GW Depth During Test	10.6		

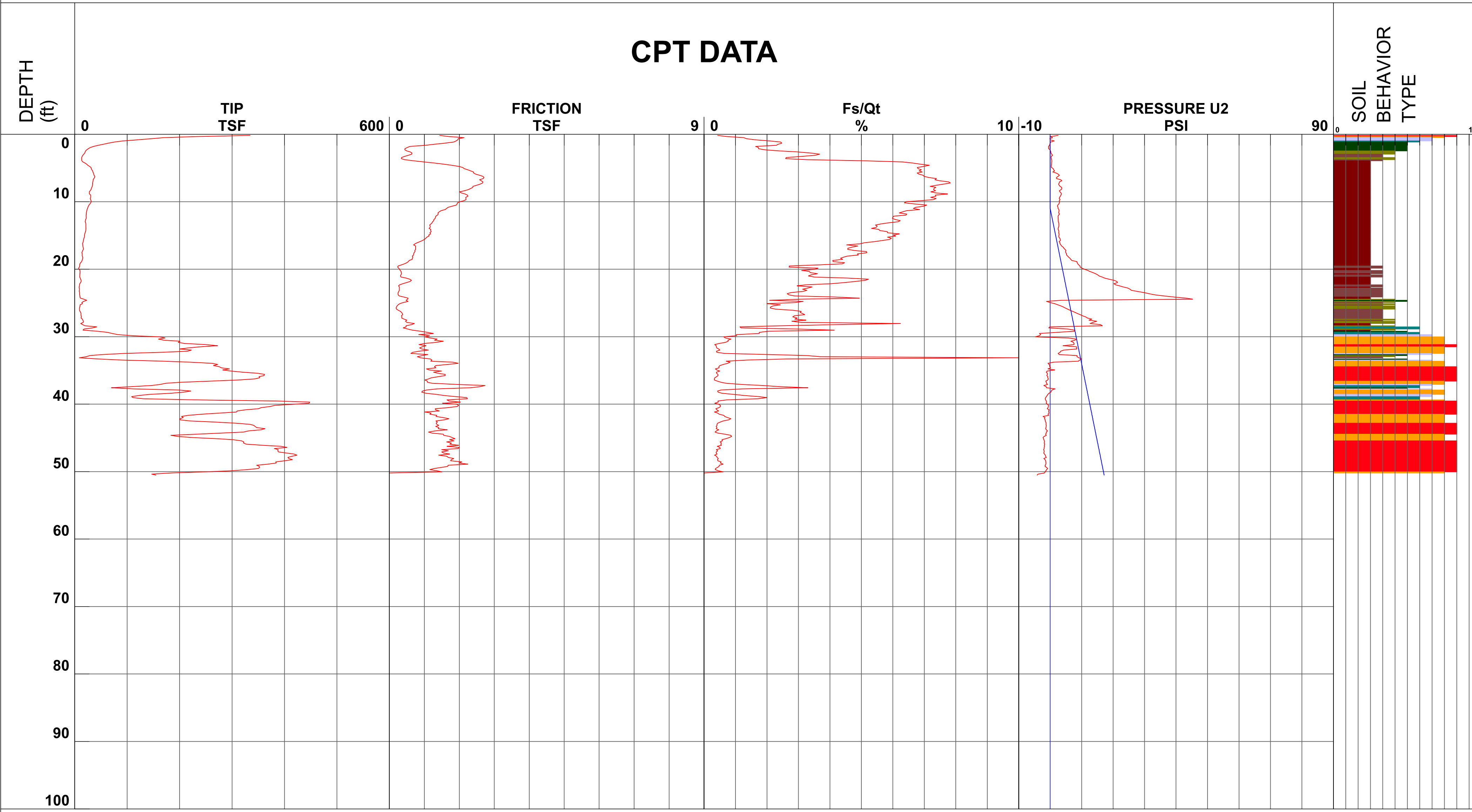


Cornerstone Earth Group

Project	211-215 River Oaks Parkway GI	Operator	AJ-ER	Filename	SDF(739).cpt
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	6/2/2023 1:01:20 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		11.00 ft			

Net Area Ratio .8

CPT DATA



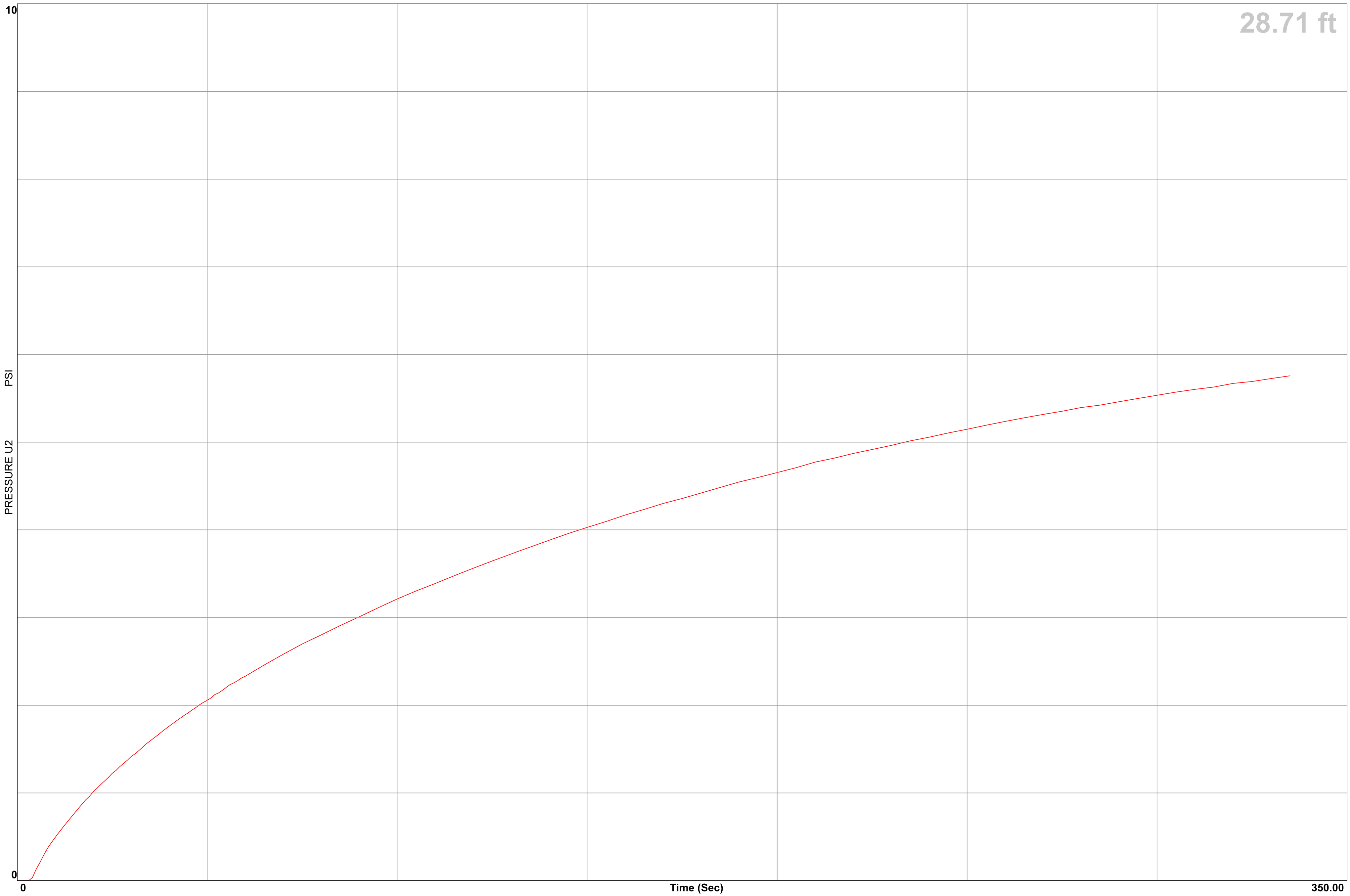
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey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

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	211-215 River Oaks Parkway Gl	Operator	AJ-ER		
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	6/2/2023 1:01:20 PM		
Equilized Pressure	5.7	EST GW Depth During Test	15.4		

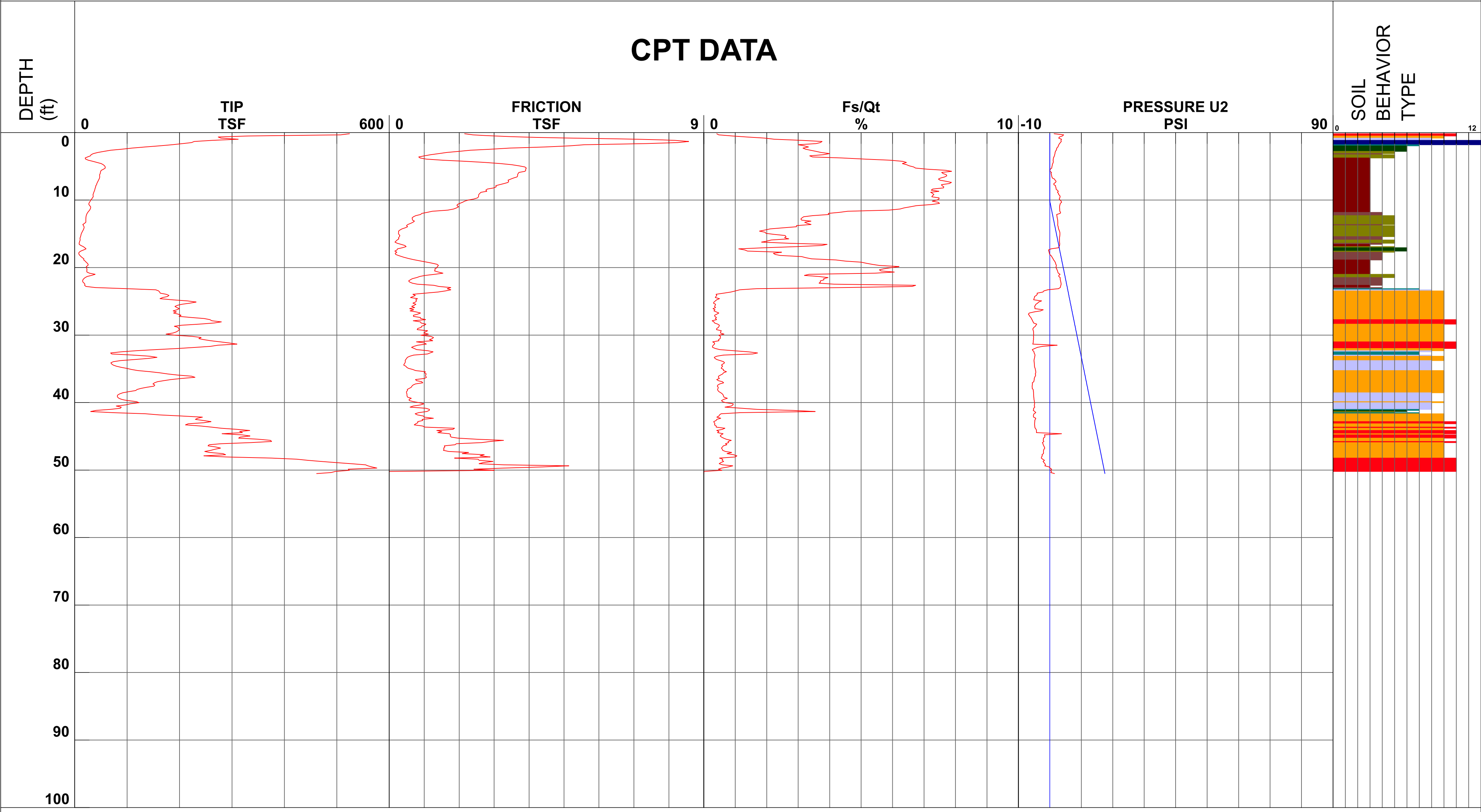


Cornerstone Earth Group

Project	211-215 River Oaks Parkway GI	Operator	AJ-ER	Filename	SDF(740).cpt
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-05	Date and Time	6/2/2023 1:45:47 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		10.20 ft			

Net Area Ratio .8

CPT DATA



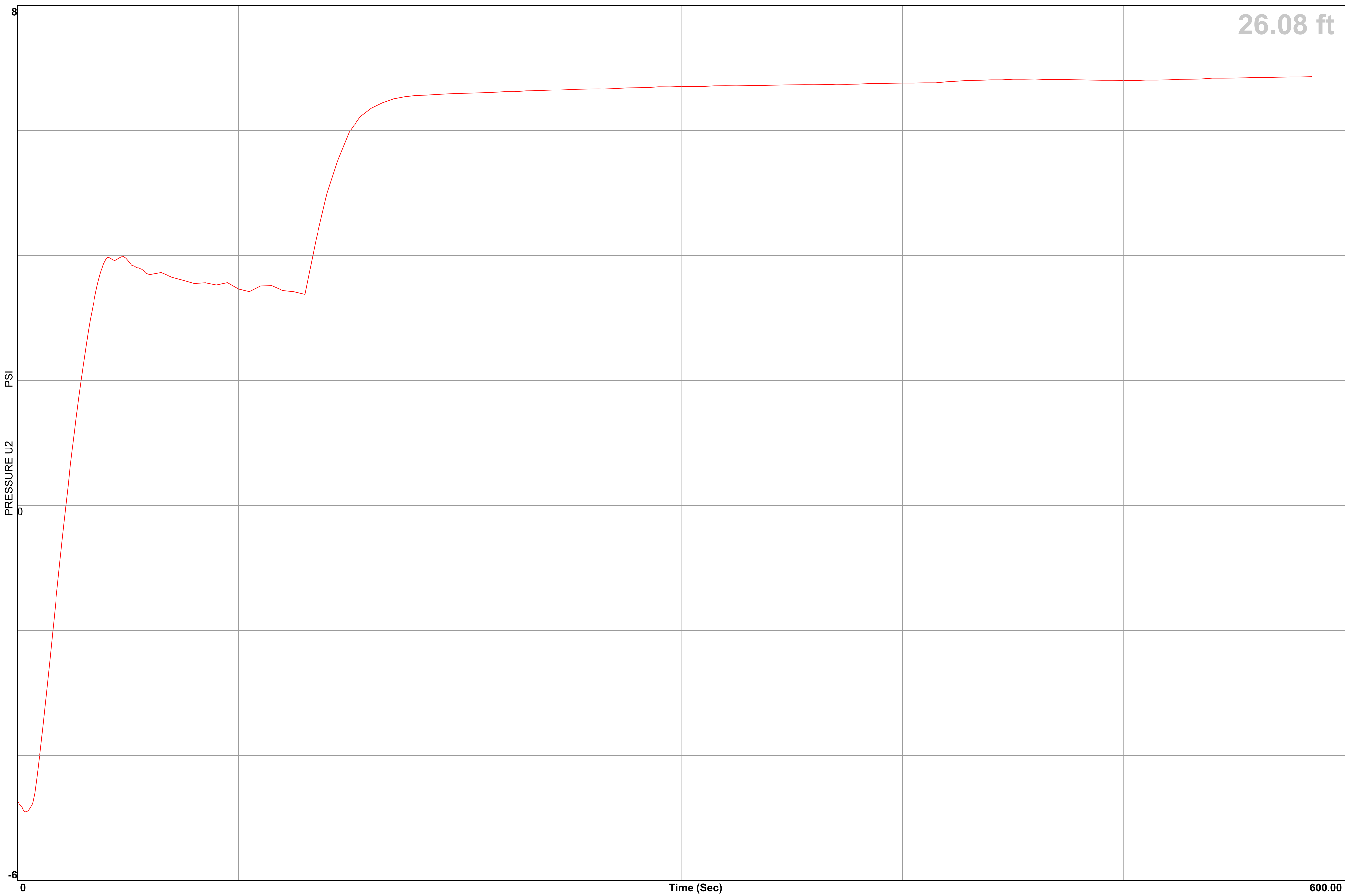
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	211-215 River Oaks Parkway GI	Operator	AJ-ER		
Job Number	384-16-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-05	Date and Time	6/2/2023 1:45:47 PM		
Equilized Pressure	6.8	EST GW Depth During Test	10.2		



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