

Appendix E: Geology and Soils Supporting Information

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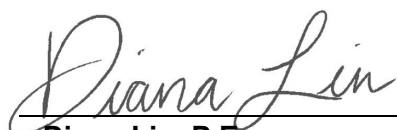
TYPE OF SERVICES	Geotechnical Preliminary Study
PROJECT NAME	Cozy Drive Residential Development
LOCATION	511 Cozy Drive San Jose, California
CLIENT	TTLIC San Jose - Glider, LLC
PROJECT NUMBER	648-25-2
DATE	March 18, 2022 (Revised June 13, 2023)



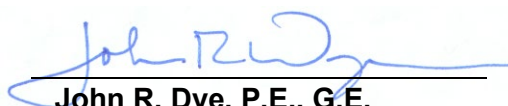
GEOTECHNICAL

Type of Services	Geotechnical Preliminary Investigation
Project Name	Cozy Drive Residential Development
Location	511 Cozy Drive San Jose, California
Client	TTLC San Jose - Glider, LLC
Client Address	12647 Alcosta Blvd., Suite 470 San Ramon, California
Project Number	648-25-2
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Prepared by



Diana Lin, P.E.
Project Engineer
Geotechnical Project Manager



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

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FIGURE 1: VICINITY MAP

FIGURE 2: SITE PLAN

FIGURE 3: REGIONAL FAULT MAP

APPENDIX A: FIELD INVESTIGATION

APPENDIX B: LABORATORY TEST PROGRAM

APPENDIX C: SITE CORROSIVITY EVALUATION

APPENDIX D: LIQUEFACTION ANALYSES CALCULATIONS

Type of Services	Preliminary Geotechnical Investigation
Project Name	Cozy Drive Residential Development
Location	511 Cozy Drive San Jose, California

SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of TTLC San Jose - Glider, LLC for the Cozy Drive San Jose in San Jose, 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.

1.1 PROJECT DESCRIPTION

The project site is located at 511 Cozy Drive, San Jose, California. The site is currently occupied by the vacant Glider Elementary School. We understand that single-family homes are currently planned for the site.

From prior experience, we anticipate the planned development will be two-story, at-grade structures. We anticipate the structures will be of wood-frame construction supported on shallow foundations. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also likely planned. Structural loads are not currently known for the proposed structures; however, structural loads are expected to be typical of similar type structures. Grading will likely consist of minor cuts and fills less than 3 feet.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated February 11, 2022 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, preliminary engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, and pavements, and preparation of this preliminary report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of five hand auger (HA) borings and seven Cone Penetration Tests (CPTs) advanced on March 4, 2022. The hand auger borings were advanced to a depth of approximately 5 feet. The CPTs were advanced to depths of approximately 45 feet. The CPTs were backfilled with cement grout in accordance with local requirements. 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 LABORATORY TESTING PROGRAM

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

1.5 CORROSION EVALUATION

Two samples from our borings from depths from 2 to 5 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 site assessments; environmental findings and conclusions are provided under separate covers.

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.

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	1.9	3.1
Hayward (Southeast Extension)	6.3	10.2
Calaveras	8.5	13.7
Sargent	8.5	13.7
San Andreas (1906)	9.6	15.5
Zayante Vergas	12.7	20.5
Hayward (Total Length)	14.5	23.4

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

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The approximately 10-acre site is located at 511 Cozy Drive and is currently designated as APN 689-08-001. The site is bounded by Glider Drive to the east, Cozy Drive to the south, and residential development to the west and to the north. The site is occupied by the recently closed Glider Elementary School, which consist of various one-story, at-grade buildings surrounded by an asphalt parking lot and an asphalt playground. The site is also occupied by an open grass field with solar panels on the north and western part.

Surface pavements were not cored and observed. Based on visual observations, the existing pavements are in fair condition within the parking lot area and in poor condition within the playground area with block and alligator cracking throughout the site.

3.2 SUBSURFACE CONDITIONS

Below the existing surface, our hand augers encountered undocumented fills of hard fat clay to a depth of 1 to 1½ feet underlain by native fat clays to the maximum depth explored of 5 feet. Based on interpretation of the Cone Penetration Test (CPT) soundings, the site is generally underlain by deep alluvial soils consisting of stiff to very stiff clays interbedded with occasional

discontinuous layers of medium dense to very dense sands to the maximum depth explored of 45 feet.

3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative surficial soil sample. The test result was used to evaluate expansion potential of surficial soils. The result of the surficial PI test indicated a PI of 28, indicating moderate to high expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 5 feet range from 5 to 15 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was inferred from our CPTs during pore pressure dissipation tests at depths of approximately between 8½ to 24 feet below current grades. All measurements were taken at the time of exploration and may not represent the stabilized levels that can be higher than the initial levels encountered. Based on our previous experience in the area and review of groundwater maps (CGS 2003), we anticipate that the high groundwater level will be between 10 to 20 feet below current grades.

On a preliminary basis, we used a design groundwater depth of 10 feet for our liquefaction analysis. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, or a Santa Clara County Fault Hazard Zone, or a City of San Jose Potential 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) was estimated for analysis using a value equal to $F_{PGA} \cdot PGA$, 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 liquefaction analysis we used a PGA_M of 0.884g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Santa Teresa Hills Quadrangle, 2003) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 45 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design groundwater depth of 10 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

The soil’s CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were not used in our analyses, as the “N” values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater

level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_c) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 to CPT-7) are presented on Figures D1 to D7 in Appendix D.

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 less than $\frac{1}{4}$ inch to $\frac{3}{4}$ 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 $\frac{1}{2}$ -inch over a horizontal distance of 30 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 10-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.

The current Canoas Creek runs approximately 270 feet north of the proposed site. The channel bottom is at approximately Elevation 159 feet according to Google Earth. Therefore, the channel bottom is approximately 8 feet deep relative to existing site grades within the lower portion of the site. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the vicinity of CPT-1 through CPT-7 based on the LDI calculations are considered negligible.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the 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 17 miles inland from the San Francisco Bay shoreline and is approximately 166 to 168 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 D, described as "Area with Flood Risk due to Levee". 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 not located within a dam failure inundation area.

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 prepared indicating where proposed structures are planned. 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.

- Presence of moderately to highly expansive soils
- Redevelopment considerations
- Presence of undocumented fill
- Soil corrosion potential

5.1.1 Expansive Soils

As discussed, moderately to highly expansive soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. If structures are underlain by expansive soils it is important that foundation systems be capable of tolerating or resisting any potentially damaging soil movements. 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. Grading and foundation recommendations that address this concern should be developed during the design-level investigation. On a preliminary basis, we recommend that residential structures be supported on post-tensioned mat foundations designed to tolerate seasonal soil movement.

5.1.2 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings as it was a previous elementary school and appurtenant flatwork, play structure, site fixtures, and landscaping. We understand that all of the existing improvements will be demolished for the construction of new residential structures and streets. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills..

5.1.3 Undocumented Fill

Our hand auger explorations encountered undocumented fills to a depth of up to 1½ below the existing surface. Our experience with similar previously developed site indicates a few feet of

undocumented fill may be locally present across the site, especially beneath the existing building. On a preliminary basis, we recommend that any existing undocumented fill be over-excavated and re-compacted beneath any new at-grade structures or prior to placing any new fill. The lateral extent and depth of existing fills should be further investigated during the design-level investigation.

On a preliminary basis, new buildings can likely be supported on a shallow foundation consisting of conventional footings or a rigid mat foundation, as discussed in the “Foundations” section.

5.1.4 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete needs no special requirements. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this feasibility study were based on limited site development information and limited exploration. 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: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend that existing foundations, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, native soils that are disturbed during demolition of the existing improvements should also be removed and replaced as engineered fill. Any existing undocumented fill encountered during grading should be over-excavated down to native soils within the proposed building footprints and 5 feet laterally beyond. On a preliminary basis and for conceptual planning and cost estimating, undocumented fill over-excavation on the order of 2 feet below current site grades should be considered.

On-site soils below the stripped layer appear to be suitable for use as fill at the site, provided they are determined to be suitable for re-use from an environmental viewpoint. Imported fill

material for use as general fill should have a Plasticity Index of 15 or less and have at least 10 percent silt or clay fines to prevent sloughing or caving during construction. Existing asphalt and concrete materials can likely be pulverized and re-used as granular base material at the site; however, asphalt grindings should not be re-used beneath residential lots or building footprints. All fill as well as scarified surface soils in areas to receive fill or slabs-on-grade, and subgrade, and trench backfill, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 percent above optimum moisture. Aggregate materials should be compacted to at least 95 percent in pavement areas, and 90 percent in flatwork areas.

It should be noted that excavations extending greater than 5 feet below current site grades may encounter loose to medium dense sands that could be susceptible to slough or caving. Excavation contractors may need to plan to lay back deeper trench excavations at inclinations of 1:1 (horizontal:vertical) or flatter in accordance with OSHA requirements.

6.2 SURFACE DRAINAGE

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 2 to 3 percent away from buildings. Due to the high expansion potential of the near-surface soils, retention, detention, or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. The near-surface soils at the site consist of moderately to highly expansive clays that would likely be categorized as Hydrologic Soil Group D, and are expected to have infiltration rates on the order of 0.1 to 0.2 inches per hour or less. In our opinion, the shallow clayey soils will significantly limit stormwater infiltration.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, the proposed residential structures could be supported on shallow foundations consisting of rigid mat foundations.

7.2 SEISMIC DESIGN CRITERIA

We assume the project design will be based on the 2022 California Building Code (CBC). The 2022 CBC provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficient" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our experience in the vicinity and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot. Therefore, the site is designated as Site Class D (determined). The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at <https://hazards.atcouncil.org/>, based on the site coordinates presented

below and the site classification. We assume the structural engineer would intend to take Exception 2 per Chapter 20 of ASCE 7-16, and as such, a full Ground Motion Hazard Analysis will not be required. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 2: Preliminary 2022 CBC Site Data

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.2333225°
Site Longitude	-121.825279°
0.2-second Period Mapped Spectral Acceleration ¹ , S_s	1.951g
1-second Period Mapped Spectral Acceleration ¹ , S_1	0.695g
Short-Period Site Coefficient – F_a	1
Long-Period Site Coefficient – F_v	Null
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	1.951g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	Null
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.301g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	Null

7.3 POST-TENSIONED MAT FOUNDATIONS

The planned at-grade residential structures may be supported on post-tensioned (PT) concrete mat foundations designed in accordance with the procedures developed by the Post-Tensioning Institute (latest edition) and the 2022 California Building Code.

To reduce potential differential movement, on a preliminary basis, mats should be designed for a maximum average areal bearing pressure of 500 psf for dead plus live loads; at column or wall loading, the maximum localized allowable bearing pressure should be limited to about 3,000 psf. When evaluating wind and seismic conditions, allowable bearing pressure may be increased by one-third. Additional reinforcing steel may be required to help span irregularities and differential settlement.

On a preliminary basis, we estimate that differential static settlements will be on the order of $\frac{1}{2}$ to $\frac{3}{4}$ inch or less across a typical mat foundation. Final settlement and foundation deflection criteria should be determined during the design-level geotechnical investigation.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 EXTERIOR FLATWORK

On a preliminary basis, exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of non-expansive fill material overlying prepared subgrade.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgment considering the native clay soils and variable surface conditions.

Table 3: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	8.0	10.5
4.5	2.5	10.0	12.5
5.0	3.0	10.0	13.0
5.5	3.0	12.0	14.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

Note: Caltrans Class 2 aggregate base; minimum R-value of 78
Preliminary subgrade R-value assumed to be 5

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of TTLC San Jose - Glider, LLC specifically to support the preliminary design of the Cozy Drive Residential Development 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.

Preliminary 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.

TTLIC San Jose - Glider, LLC may have provided Cornerstone with plans, reports and other documents prepared by others. TTLIC San Jose - Glider, 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.

Preliminary 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 11: REFERENCES

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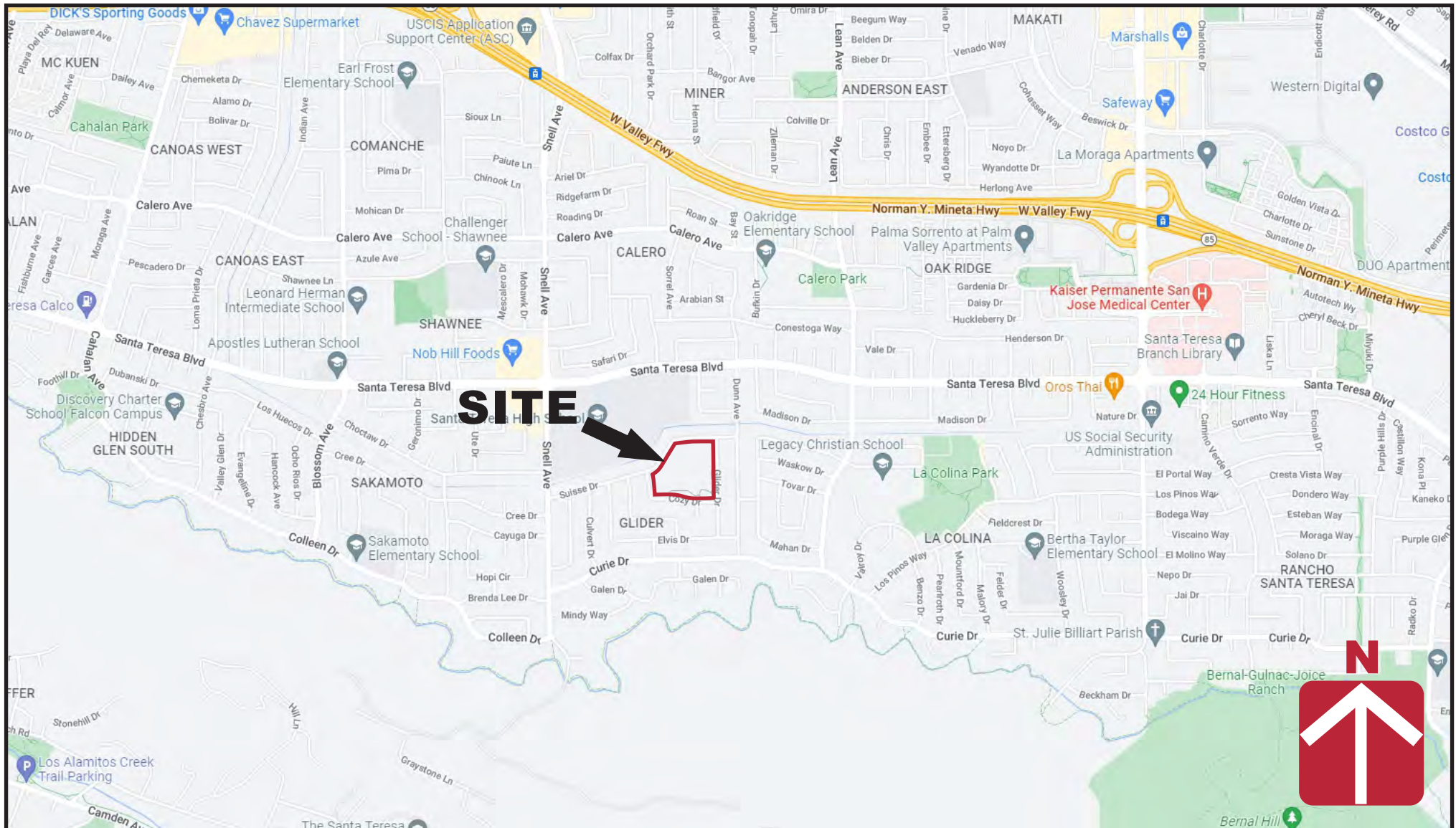
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CORNERSTONE
EARTH GROUP

Vicinity Map

511 Cozy Drive
San Jose, CA

Project Number

648-25-2

Figure Number

Figure 1

Date

March 2022

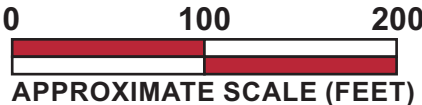
Drawn By

RRN

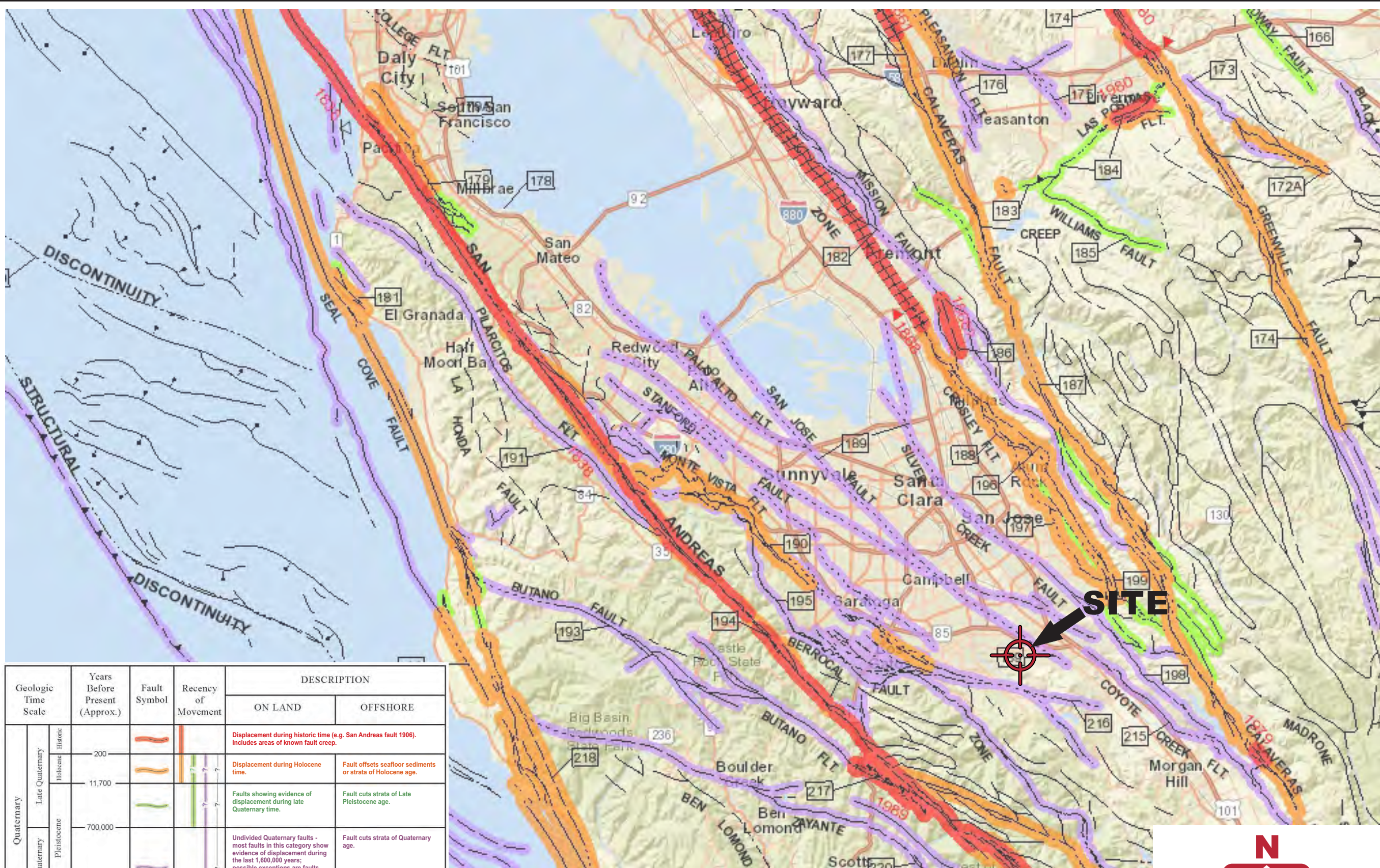


Base by Google Earth, dated 08/15/2020

- Legend**
- Approximate location of hand-auger boring (HA)
 - Approximate location of cone penetration test (CPT)

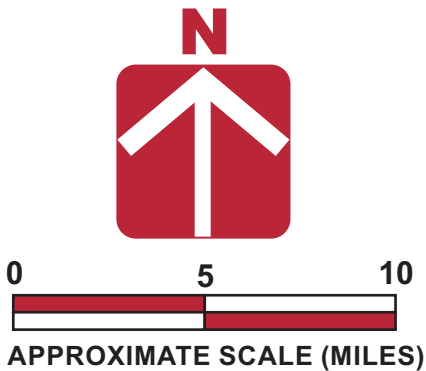


	Site Plan	
	511 Cozy Drive San Jose, CA	
	Project Number 648-25-2	
Figure Number		Figure 2
Date	March 2022	Drawn By RRN



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	Holocene	200		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			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					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					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.
		4.5 billion (Age of Earth)				

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)



Project Number648-25-2

Figure NumberFigure 3

DateMarch 2022

Drawn ByRRN

Regional Fault Map

511 Cozy Drive
San Jose, CA

**CORNERSTONE**
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using 20-ton truck-mounted Cone Penetration Test equipment. Seven CPTs were performed in accordance with ASTM D 5778-95 (revised, 2002) on March 4, 2022 to depth of approximately 45 feet below the existing ground surface. Five 3-inch-diameter hand auger borings were advanced to a depth of about 5 feet to sample the surficial soils on the same day. The approximate exploration locations are shown on the Site Plan, Figure 2.

Exploration locations were approximated using existing site boundaries, and other site features as references. CPT and boring elevations were not determined. The locations of the CPTs and hand auger borings 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

	Poorly-Graded Sand with Clay		Sand
	Clayey Sand		Silt
	Sandy Silt		Well Graded Gravelly Sand
	Artificial/Undocumented Fill		Gravelly Silt
	Poorly-Graded Gravelly Sand		Asphalt
	Topsoil		Boulders and Cobble
	Well-Graded Gravel with Clay		
	Well-Graded Gravel with Silt		

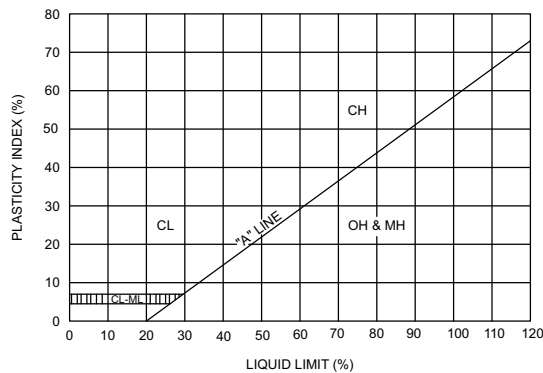
SAMPLER TYPES

	SPT		Shelby Tube
	Modified California (2.5" I.D.)		No Recovery
	Rock Core		Grab Sample

ADDITIONAL TESTS

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

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

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

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

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



CORNERSTONE EARTH GROUP

BORING NUMBER HA-1

PAGE 1 OF 1

DATE STARTED 3/4/22 DATE COMPLETED 3/4/22

DRILLING CONTRACTOR _____

DRILLING METHOD Hand AugerLOGGED BY JDS

NOTES _____

PROJECT NAME 511 Cozy DrivePROJECT NUMBER 648-25-2PROJECT LOCATION San Jose, CAGROUND ELEVATION _____ BORING DEPTH 5 ft.

LATITUDE _____ LONGITUDE _____

GROUND WATER LEVELS:▽ AT TIME OF DRILLING Not Encountered▼ AT END OF DRILLING Not Encountered

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

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Fat Clay with Sand (CH) [Fill]
moist, dark gray and gray mottled, fine sand,
high plasticity

Fat Clay (CH)
moist, gray to light gray, some fine sand, high
plasticity

Bottom of Boring at 5.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAxIAL

1.0 2.0 3.0 4.0



CORNERSTONE EARTH GROUP

BORING NUMBER HA-2

PAGE 1 OF 1

DATE STARTED 3/4/22 DATE COMPLETED 3/4/22

DRILLING CONTRACTOR _____

DRILLING METHOD Hand AugerLOGGED BY JDS

NOTES _____

PROJECT NAME 511 Cozy DrivePROJECT NUMBER 648-25-2PROJECT LOCATION San Jose, CAGROUND ELEVATION _____ BORING DEPTH 5 ft.

LATITUDE _____ LONGITUDE _____

GROUND WATER LEVELS:▽ AT TIME OF DRILLING Not Encountered▼ AT END OF DRILLING Not Encountered

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

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Fat Clay with Sand (CH) [Fill]
hard, moist, dark gray, fine sand, high plasticity

Fat Clay (CH)
moist, dark gray to light gray, some fine sand,
high plasticity

Bottom of Boring at 5.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
PCFNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAxIAL

1.0 2.0 3.0 4.0 >4.5



CORNERSTONE EARTH GROUP

BORING NUMBER HA-3

PAGE 1 OF 1

DATE STARTED 3/4/22 DATE COMPLETED 3/4/22

DRILLING CONTRACTOR _____

DRILLING METHOD Hand AugerLOGGED BY JDS

NOTES _____

PROJECT NAME 511 Cozy DrivePROJECT NUMBER 648-25-2PROJECT LOCATION San Jose, CAGROUND ELEVATION _____ BORING DEPTH 5 ft.

LATITUDE _____ LONGITUDE _____

GROUND WATER LEVELS:▽ AT TIME OF DRILLING Not Encountered▼ AT END OF DRILLING Not Encountered

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

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Fat Clay with Sand (CH) [Fill]
hard, moist, light gray, fine to medium sand,
high plasticity
Liquid Limit = 56, Plastic Limit = 28

Fat Clay (CH)
moist, gray to light gray, some fine sand, high
plasticity

Bottom of Boring at 5.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAxIAL

1.0 2.0 3.0 4.0



CORNERSTONE EARTH GROUP

BORING NUMBER HA-4

PAGE 1 OF 1

DATE STARTED 3/4/22 DATE COMPLETED 3/4/22

DRILLING CONTRACTOR _____

DRILLING METHOD Hand AugerLOGGED BY JDS

NOTES _____

PROJECT NAME 511 Cozy DrivePROJECT NUMBER 648-25-2PROJECT LOCATION San Jose, CAGROUND ELEVATION _____ BORING DEPTH 5 ft.

LATITUDE _____ LONGITUDE _____

GROUND WATER LEVELS:▽ AT TIME OF DRILLING Not Encountered▼ AT END OF DRILLING Not Encountered

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

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Fat Clay with Sand (CH) [Fill]
moist, dark gray, fine to coarse sand, high plasticity

Fat Clay (CH)
moist, gray to light gray, some fine sand, high plasticity

Bottom of Boring at 5.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
PCFNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAxIAL

1.0 2.0 3.0 4.0



CORNERSTONE EARTH GROUP

BORING NUMBER HA-5

PAGE 1 OF 1

DATE STARTED 3/4/22 DATE COMPLETED 3/4/22

DRILLING CONTRACTOR _____

DRILLING METHOD Hand AugerLOGGED BY JDS

NOTES _____

PROJECT NAME 511 Cozy DrivePROJECT NUMBER 648-25-2PROJECT LOCATION San Jose, CAGROUND ELEVATION _____ BORING DEPTH 5 ft.

LATITUDE _____ LONGITUDE _____

GROUND WATER LEVELS:▽ AT TIME OF DRILLING Not Encountered▼ AT END OF DRILLING Not Encountered

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

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Fat Clay with Sand (CH) [Fill]
moist, dark gray, fine to coarse sand, high plasticity

Fat Clay (CH)
moist, gray to light gray, some fine sand, high plasticity

Bottom of Boring at 5.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAxIAL

1.0 2.0 3.0 4.0



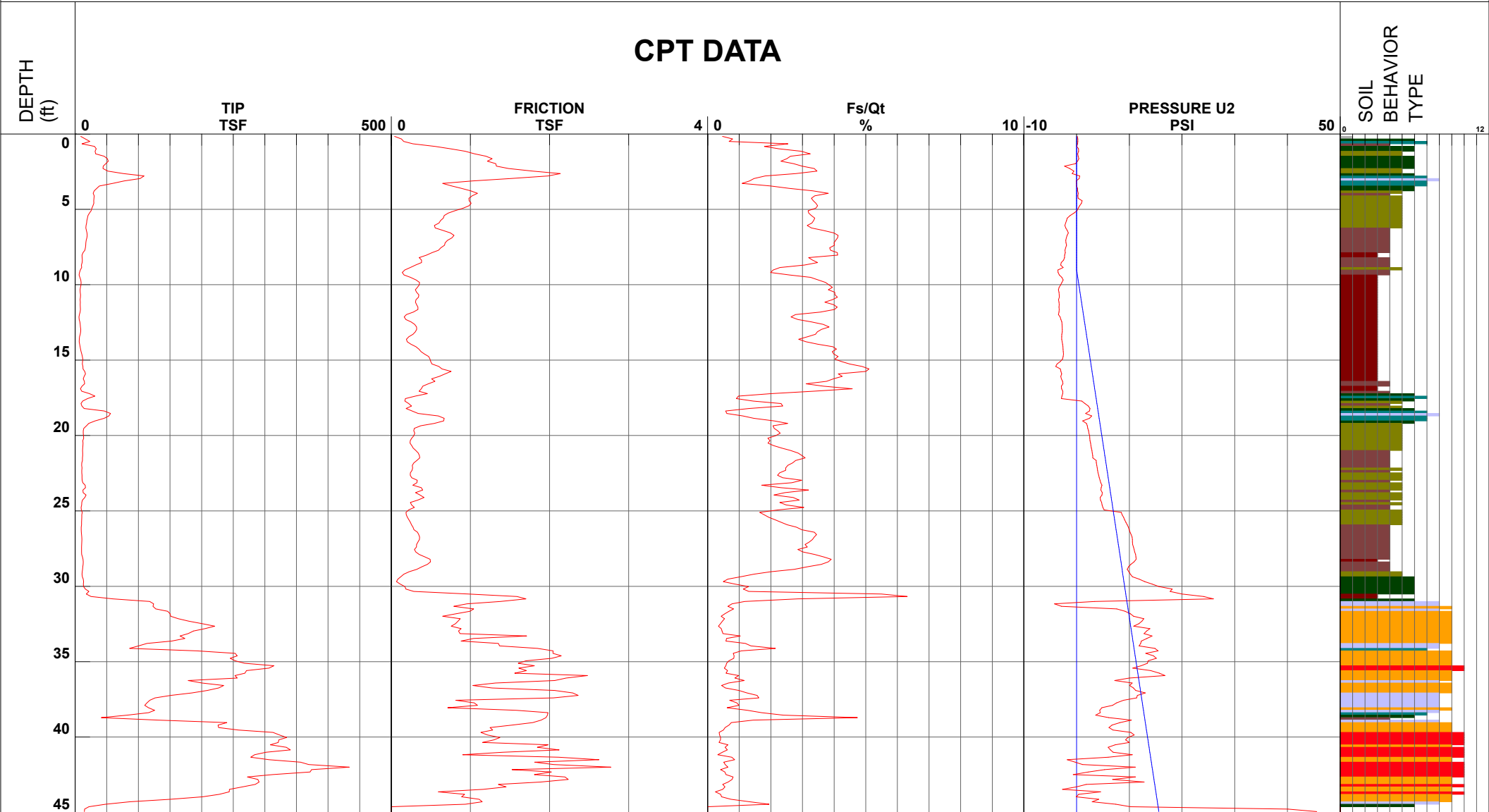
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-01
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 9:58:27 AM
9.00 ft

Filename SDF(835).cpt
GPS
Maximum Depth 44.95 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



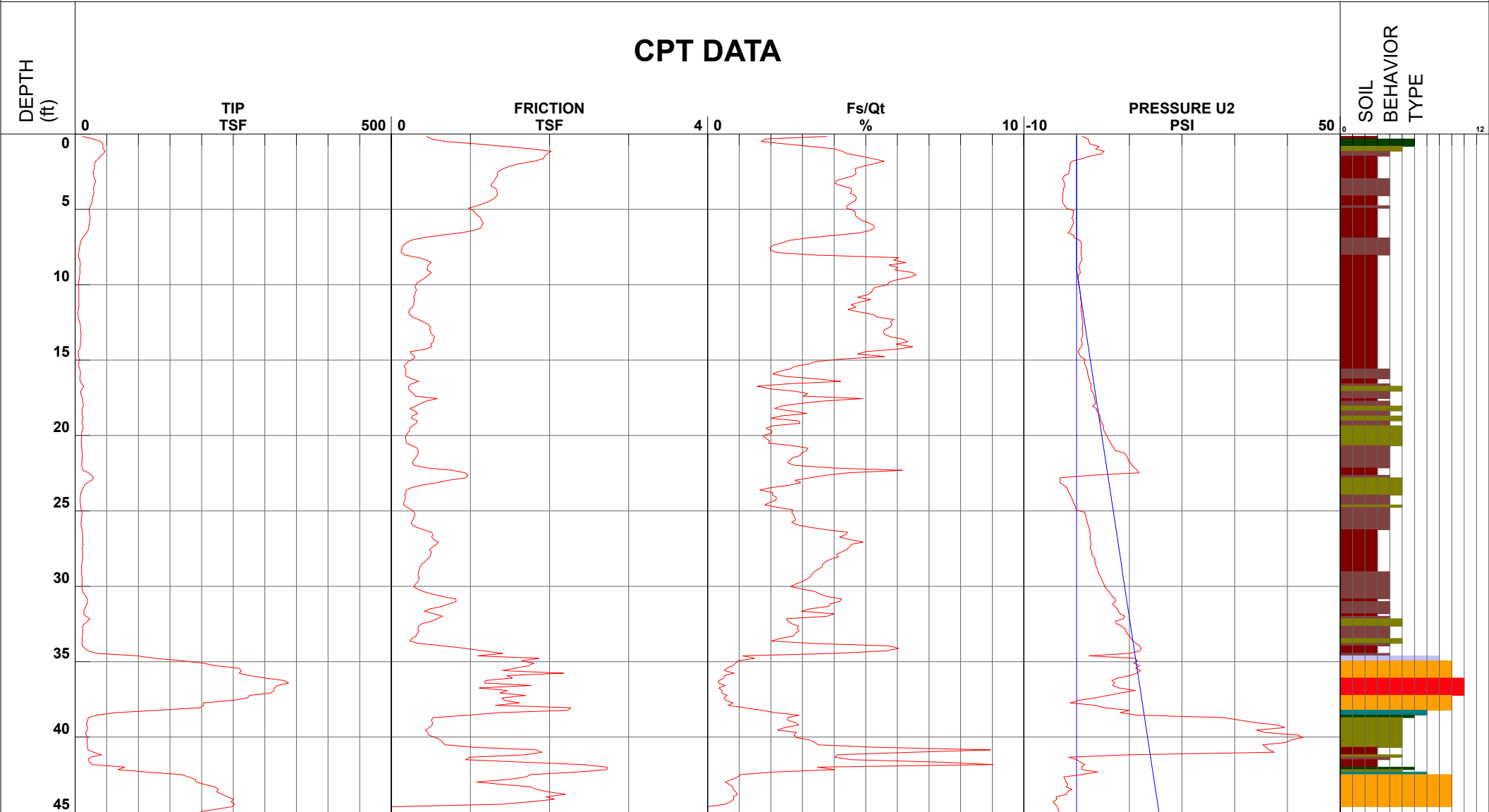
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-02
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 10:41:23 AM
9.00 ft

Filename SDF(836).cpt
GPS
Maximum Depth 44.95 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



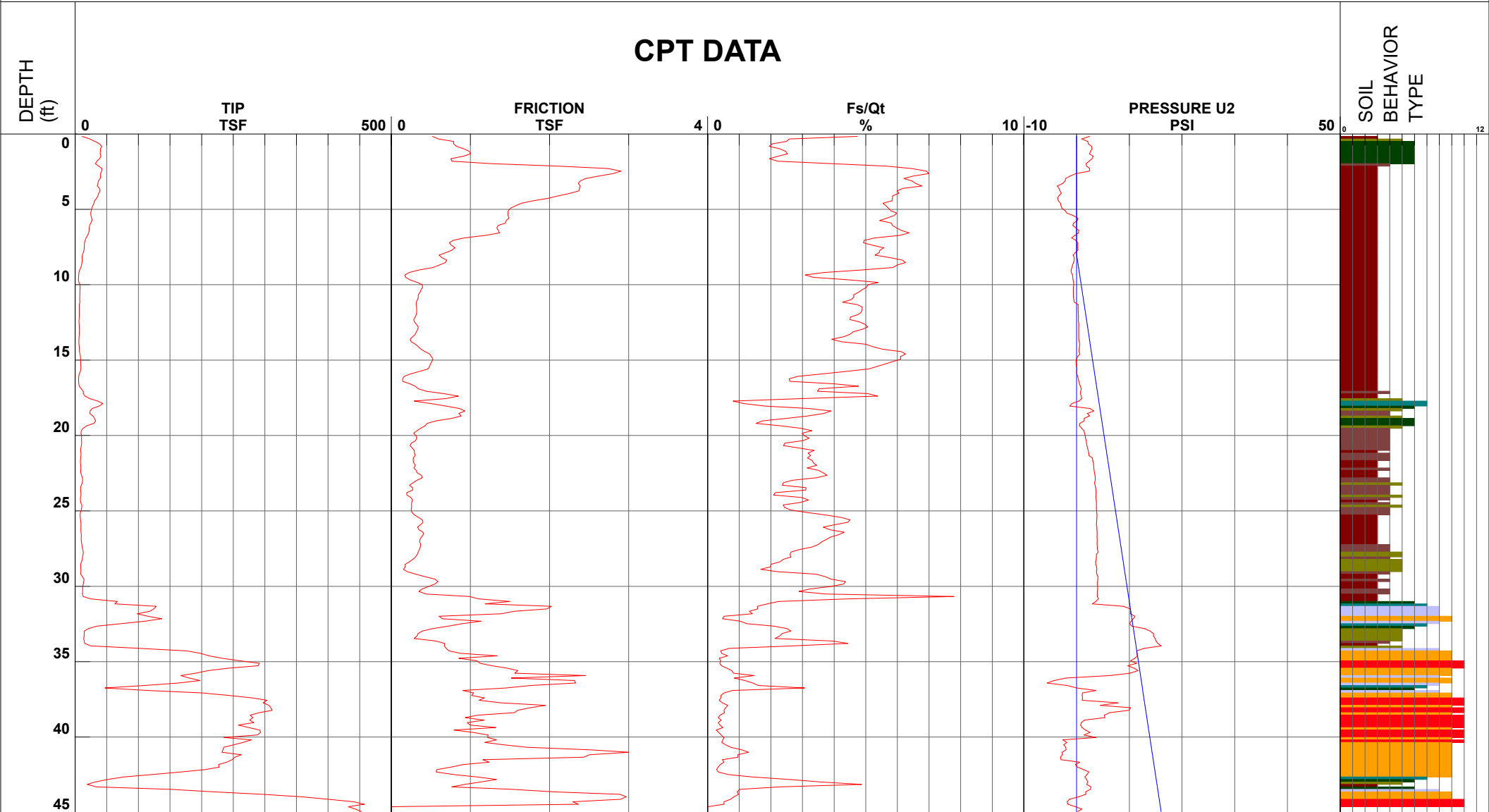
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-03
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 12:07:05 PM
8.00 ft

Filename SDF(838).cpt
GPS
Maximum Depth 44.95 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



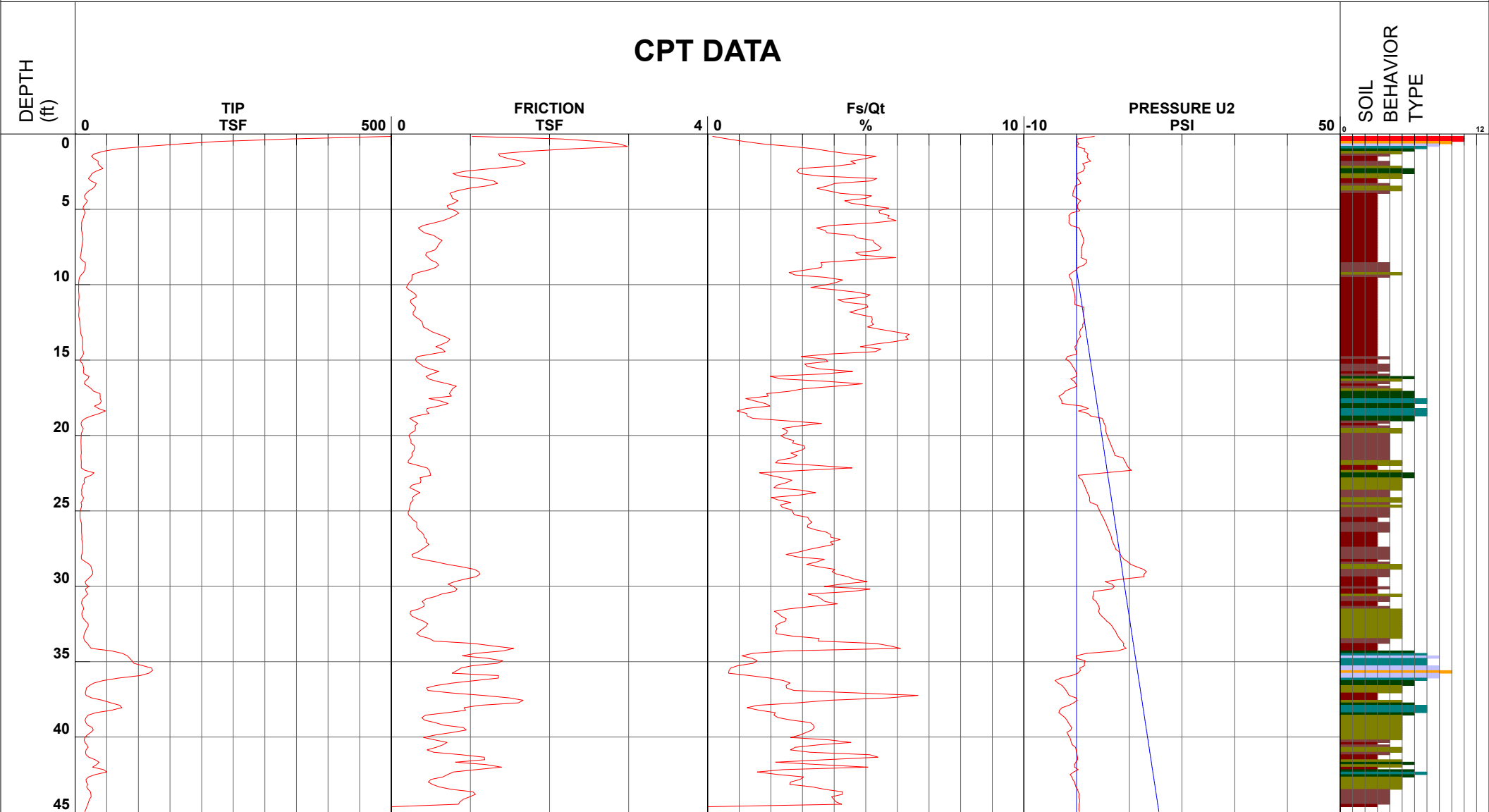
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-04
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 11:26:05 AM
9.00 ft

Filename SDF(837).cpt
GPS
Maximum Depth 44.95 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



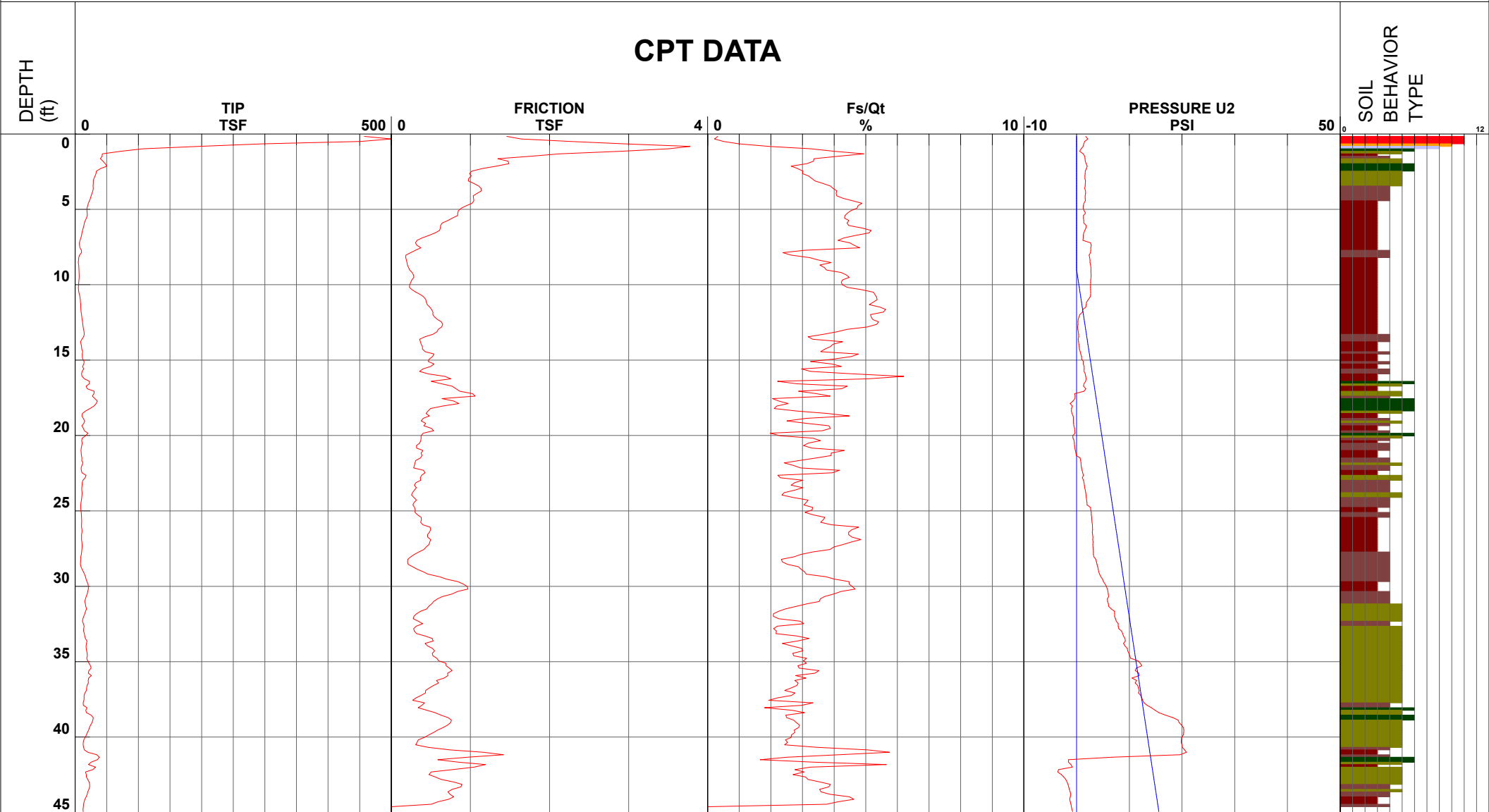
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-05
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 2:37:17 PM
9.00 ft

Filename SDF(841).cpt
GPS
Maximum Depth 44.95 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

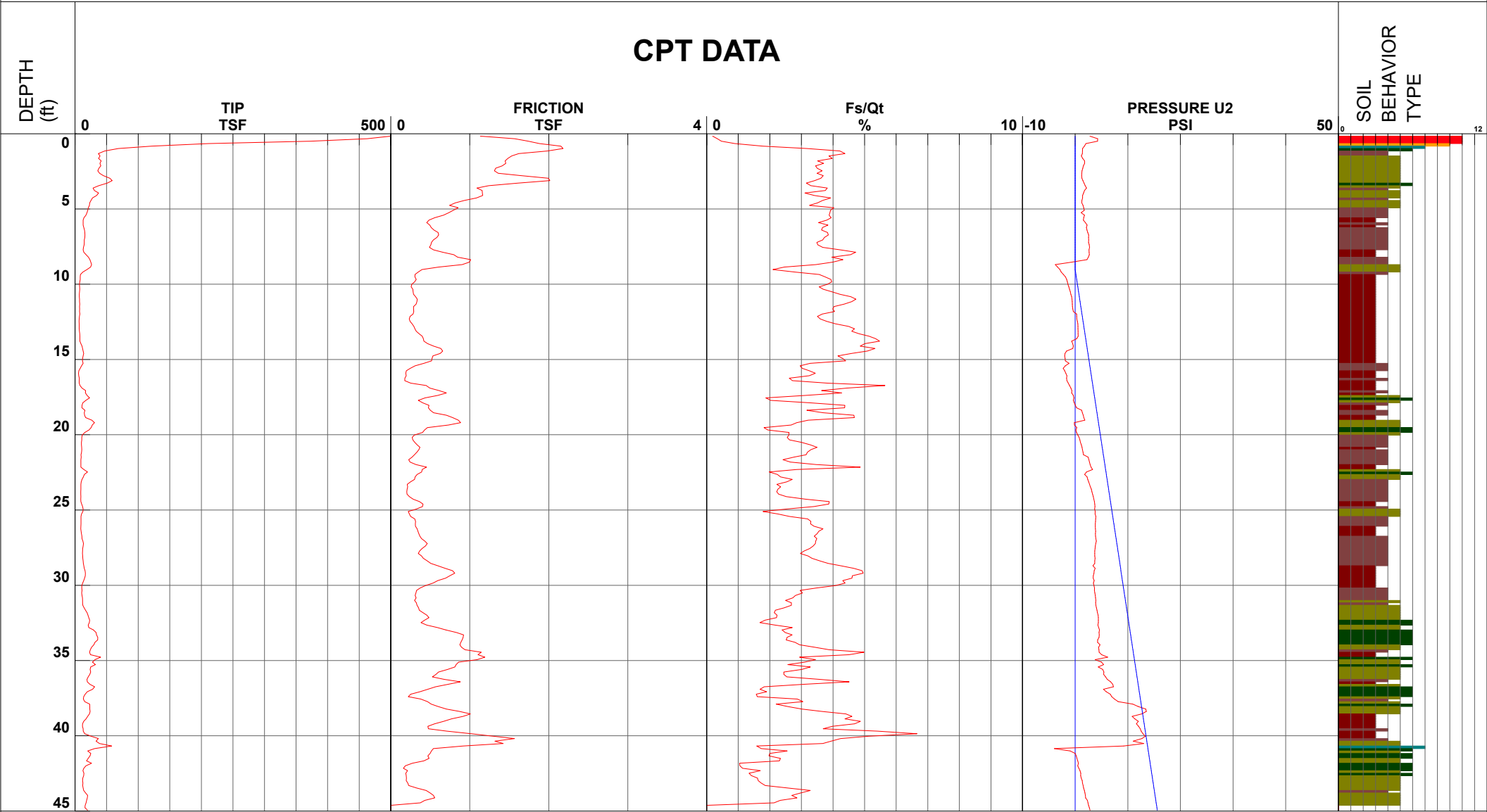
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Job Number 648-25-2
Hole Number CPT-06
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
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9.00 ft

Filename SDF(839).cpt
GPS
Maximum Depth 44.95 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



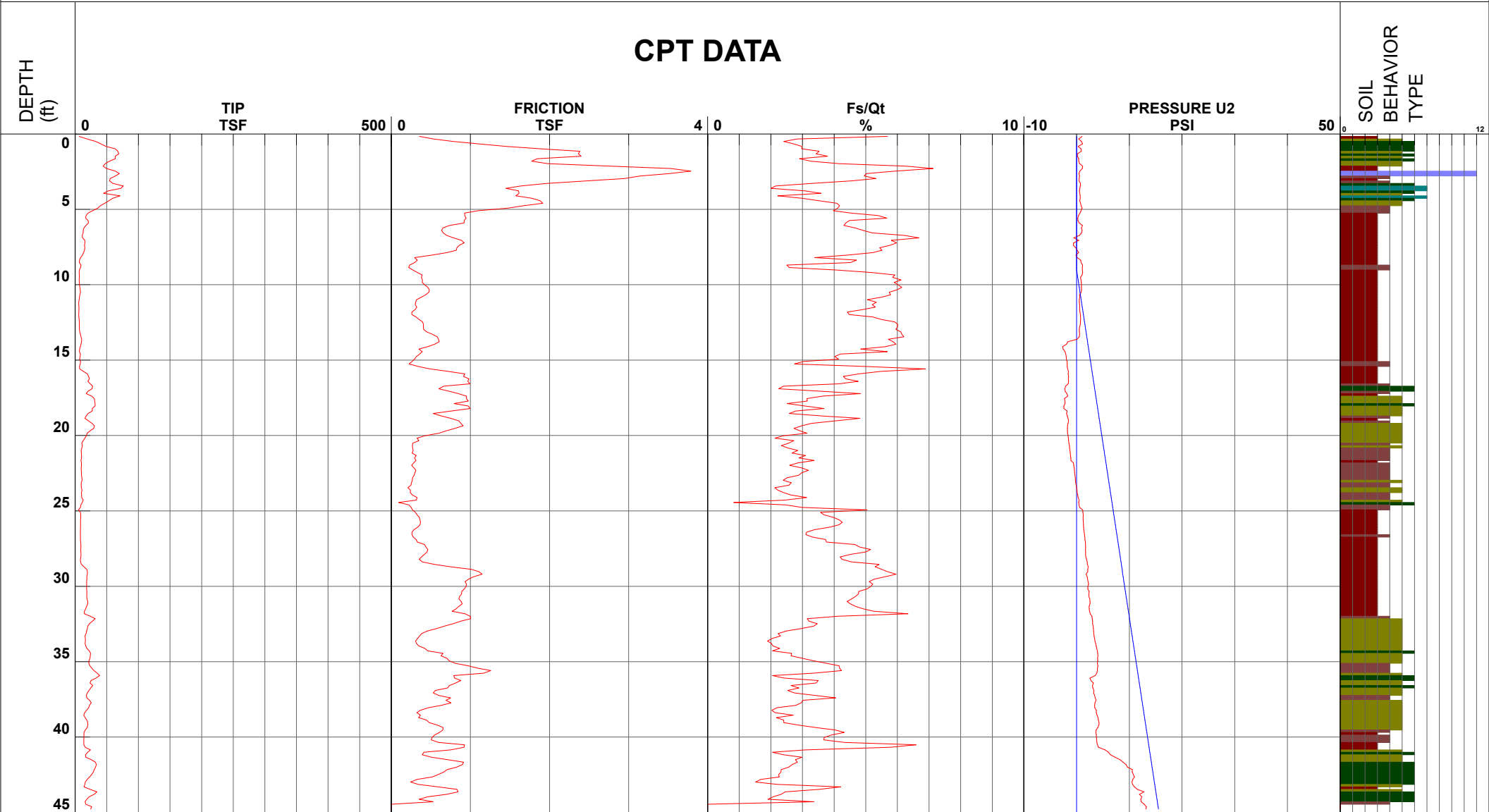
Cornerstone Earth Group

Project 511 Cozy Drive
Job Number 648-25-2
Hole Number CPT-07
EST GW Depth During Test

Operator JM-AJ
Cone Number DDG1589
Date and Time 3/4/2022 1:33:16 PM
9.00 ft

Filename SDF(840).cpt
GPS
Maximum Depth 44.78 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

APPENDIX B: LABORATORY TEST PROGRAM

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

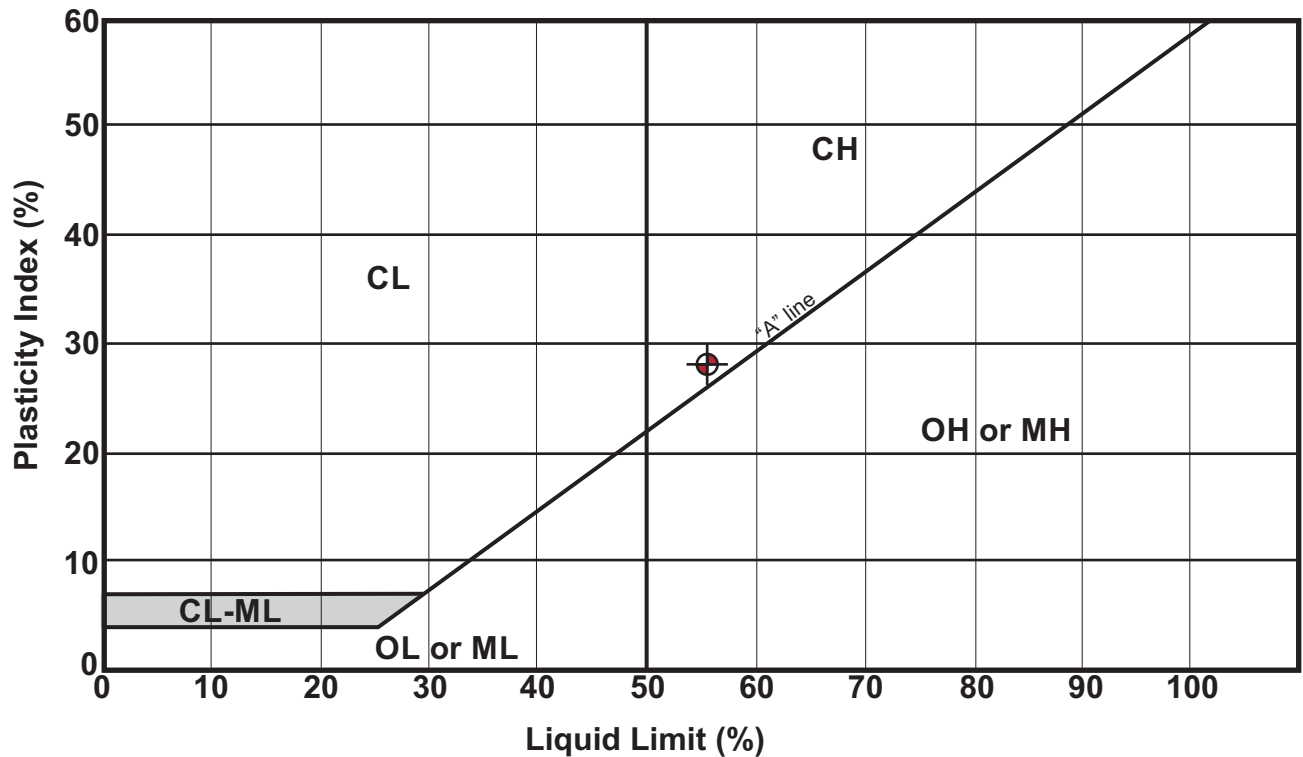
Moisture Content: The natural water content was determined (ASTM D2216) on 22 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on two samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one sample of the subsurface soils to aid in the classification of these soils. Result of this test is shown on the boring log at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
	HA-3	1.0	26	56	28	28	—	Fat Clay with Sand (CH) [Fill]

APPENDIX C: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED MARCH 17, 2022

March 17, 2022

Cornerstone Earth Group, Inc.
1220 Oakland Boulevard, Suite 220
Walnut Creek, CA 94596

Attention: **John R. Dye, P.E., G.E.**
Principal Engineer

Subject: **Site Corrosivity Evaluation**
511 Cozy Drive
San Jose, CA
Project: 648-25-2

Dear John,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Two (2) soil samples from the project site were chemically analyzed for corrosivity by **Cornerstone Earth Group**. Each sample was analyzed for chloride and sulfate concentration, pH and resistivity at 100% saturation. The test results are presented in Cornerstone Earth Group Geotechnical Report dated 2/14/2022. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	26 – 46 mg/kg	Non-corrosive*
Sulfates	71 – 91 mg/kg	Non-corrosive**
pH	7.7	Non-corrosive*
Resistivity at 100% Saturation	1,376 – 1,736 ohm-cm	Corrosive*

* With respect to bare steel or ductile iron.

** With respect to mortar coated steel

Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be “corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warranties or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to **Cornerstone Earth Group** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,



Brendon Hurley
JDH CORROSION CONSULTANTS, INC.
Field Technician

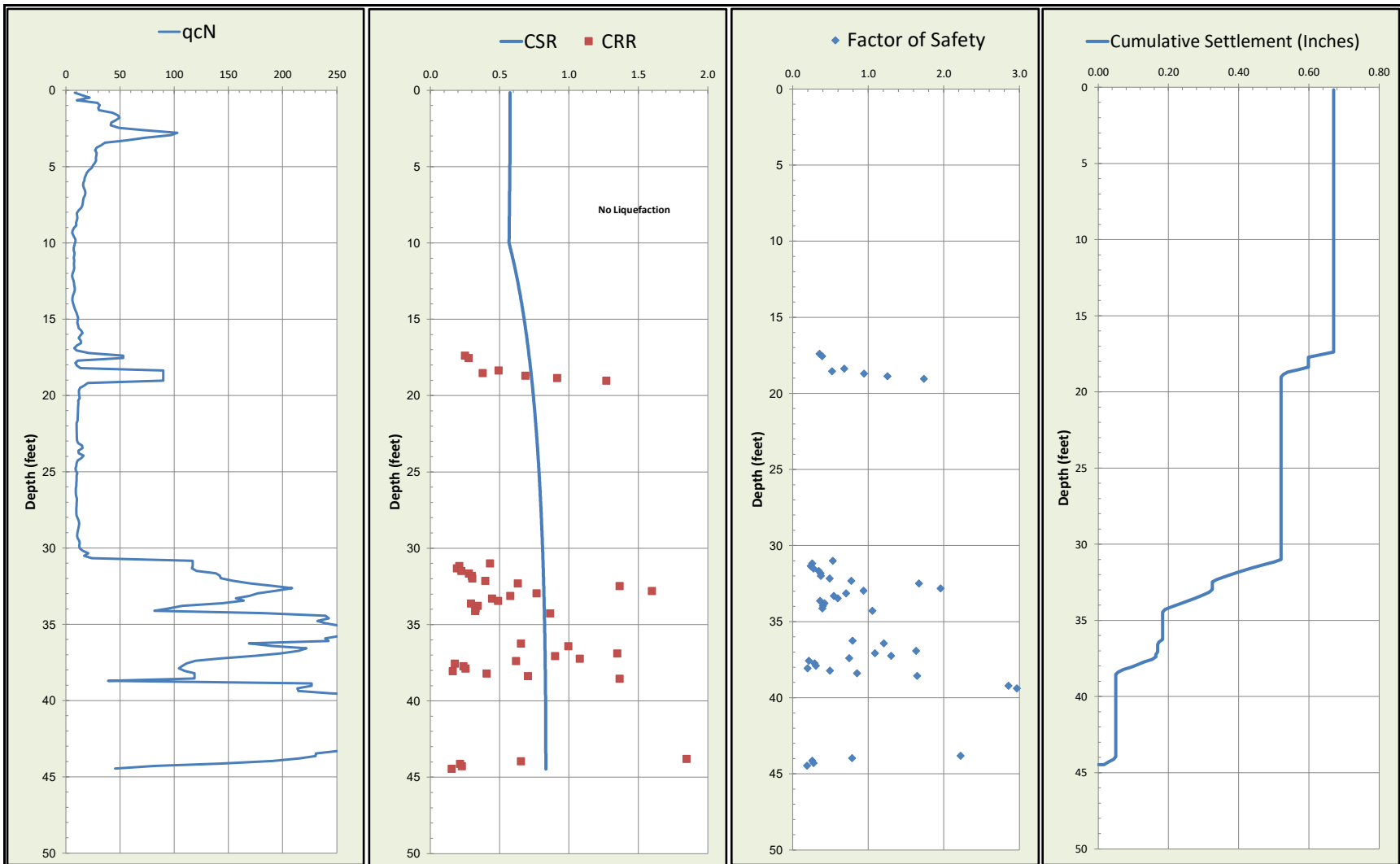
Mohammed Ali

Mohammed Ali., P.E.
JDH Corrosion Consultants, Inc.
Senior Corrosion Engineer



CC: File 2022095

APPENDIX D: LIQUEFACTION ANALYSES CALCULATIONS



**CORNERSTONE
EARTH GROUP**

Liquefaction Analysis Summary

**511 Cozy Drive
San Jose, California**

Project Number

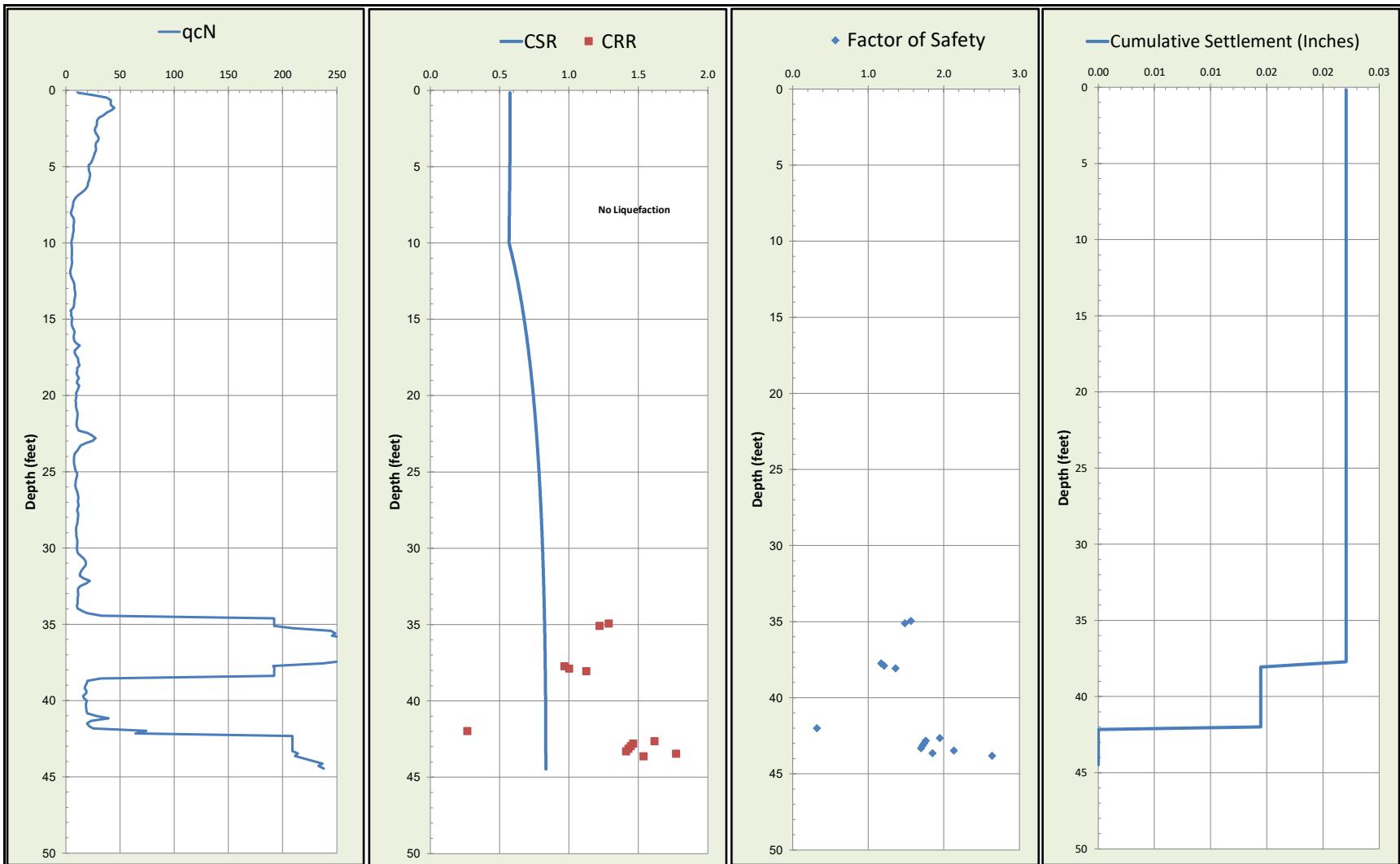
648-25-2

Figure Number

Figure D1

3/18/2022

CPT No. 1



**CORNERSTONE
EARTH GROUP**

Liquefaction Analysis Summary

**511 Cozy Drive
San Jose, California**

Project Number

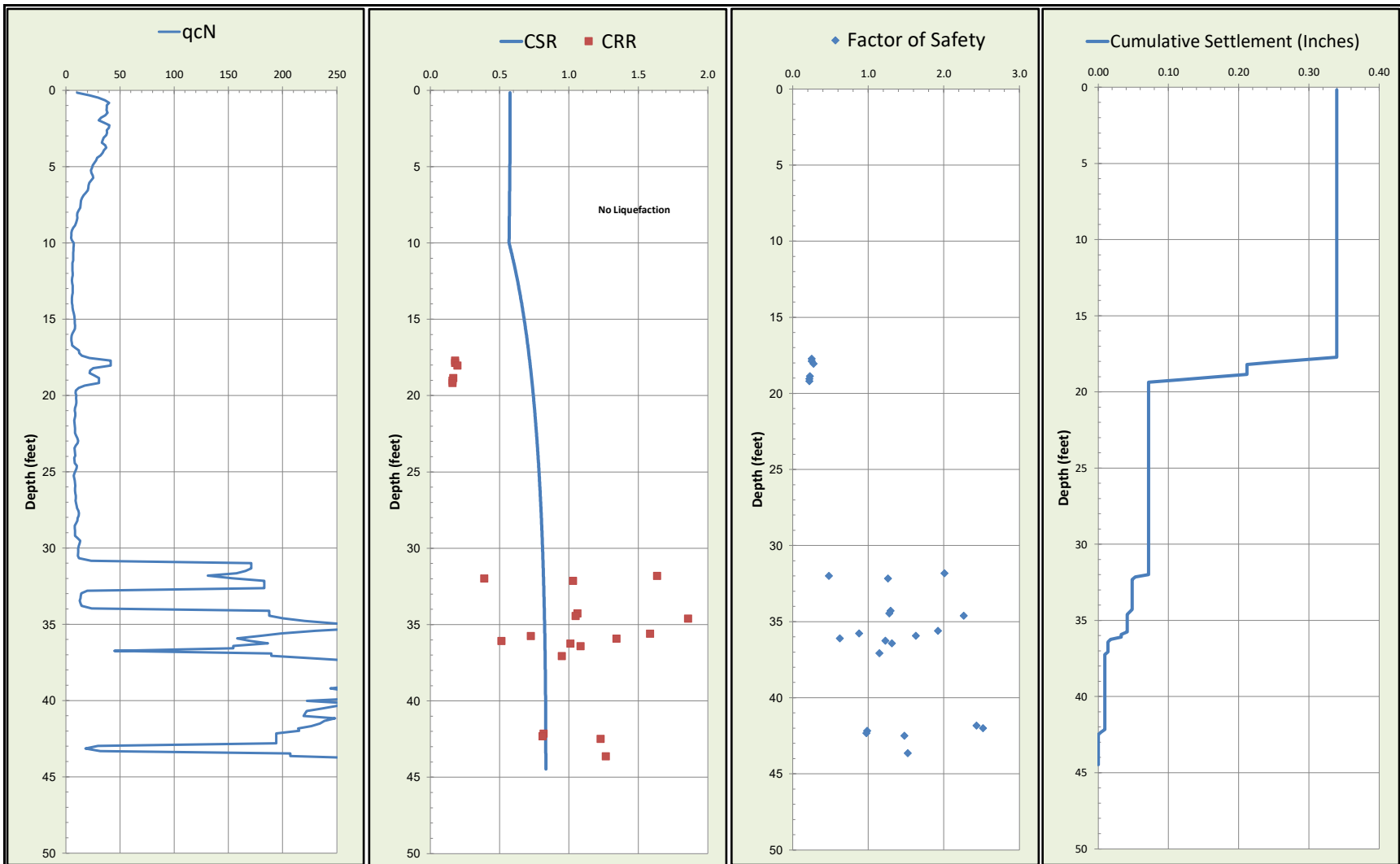
648-25-2

Figure Number

Figure D2

3/18/2022

CPT No. 2



Liquefaction Analysis Summary

511 Cozy Drive
San Jose, California

Project Number

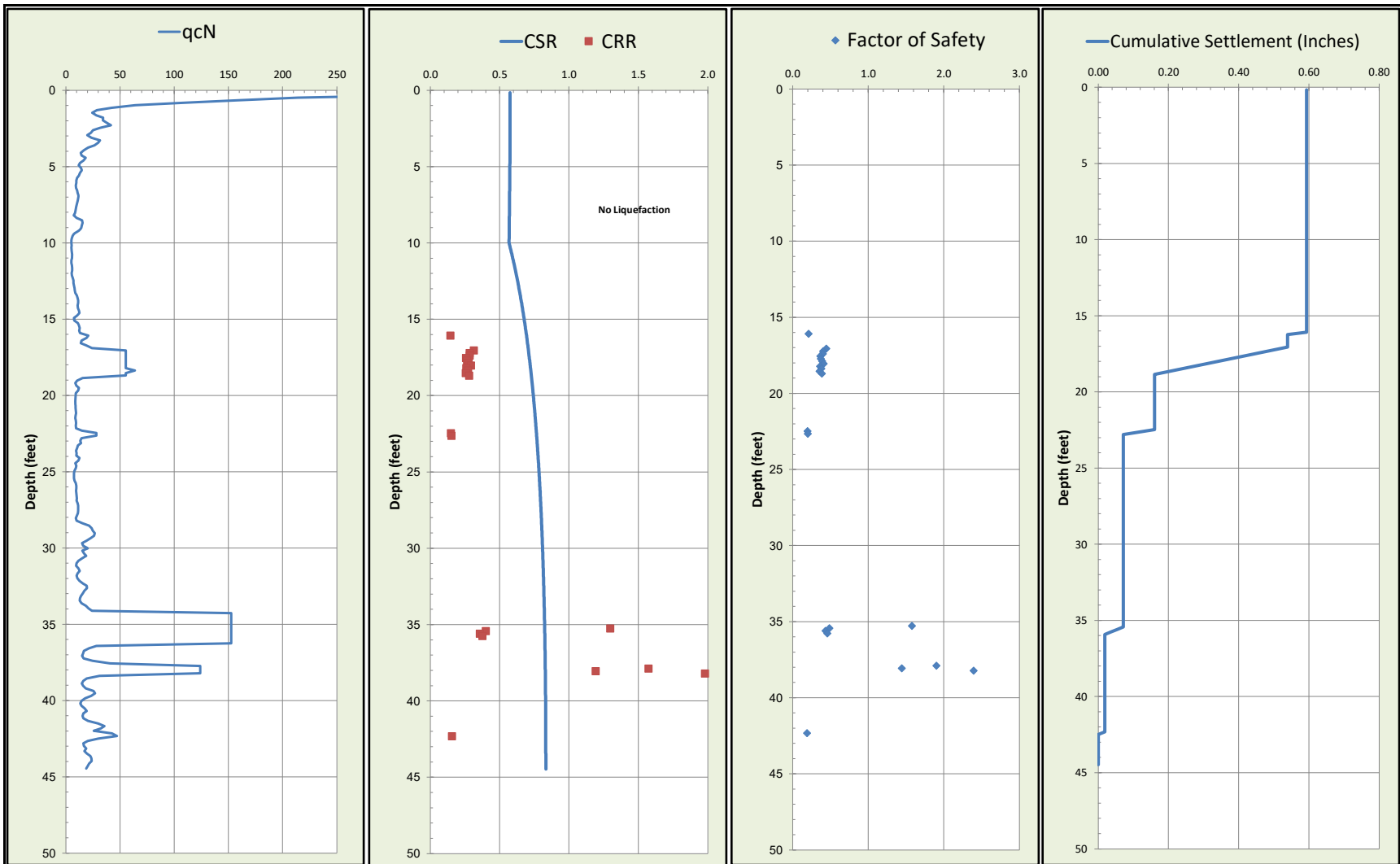
648-25-2

Figure Number

Figure D3

3/18/2022

CPT No. 3



Liquefaction Analysis Summary

511 Cozy Drive
San Jose, California

Project Number

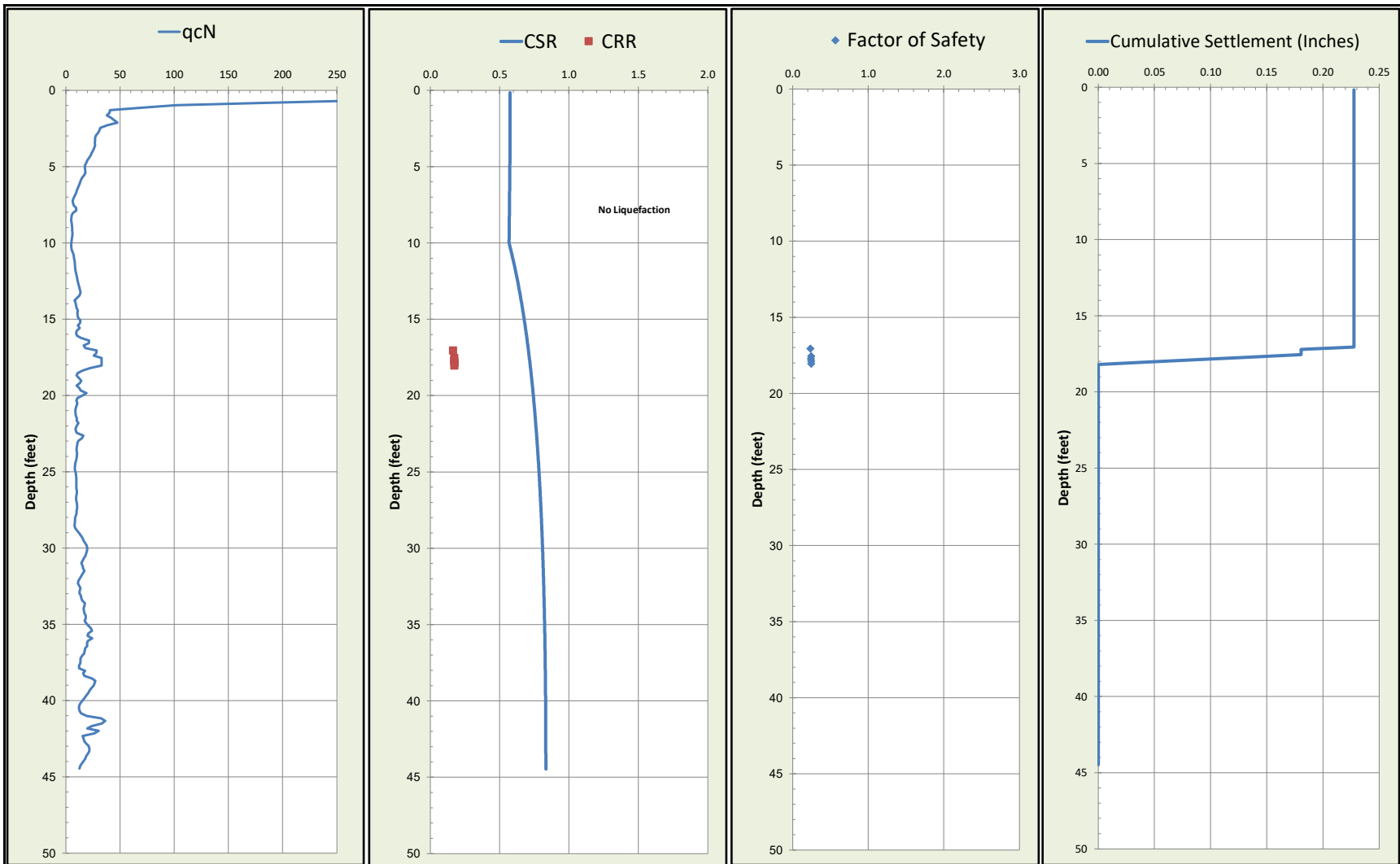
648-25-2

Figure Number

Figure D4

3/18/2022

CPT No. 4



Liquefaction Analysis Summary

511 Cozy Drive
San Jose, California

Project Number

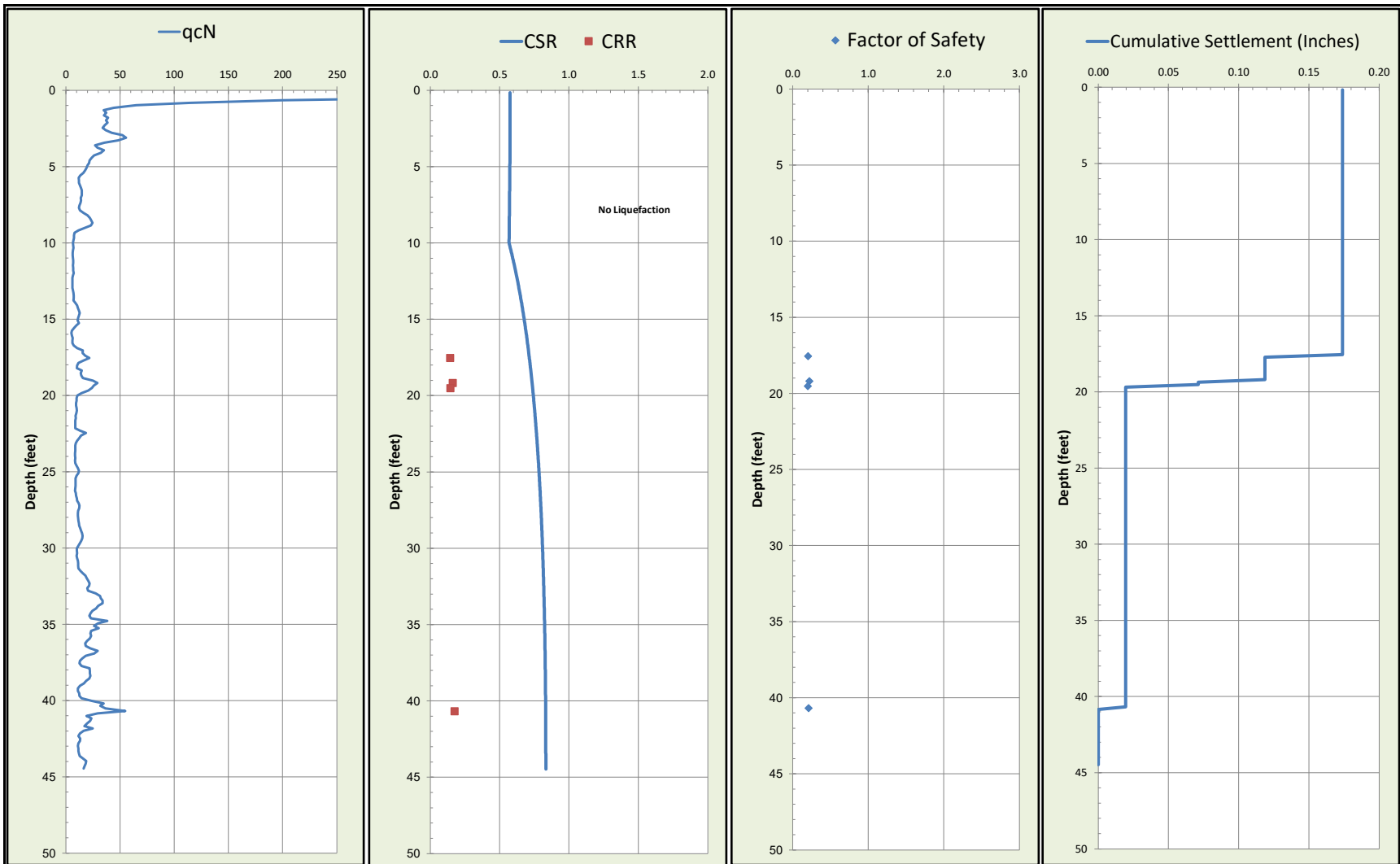
648-25-2

Figure Number

Figure D5

3/18/2022

CPT No. 5



Liquefaction Analysis Summary

511 Cozy Drive
San Jose, California

Project Number

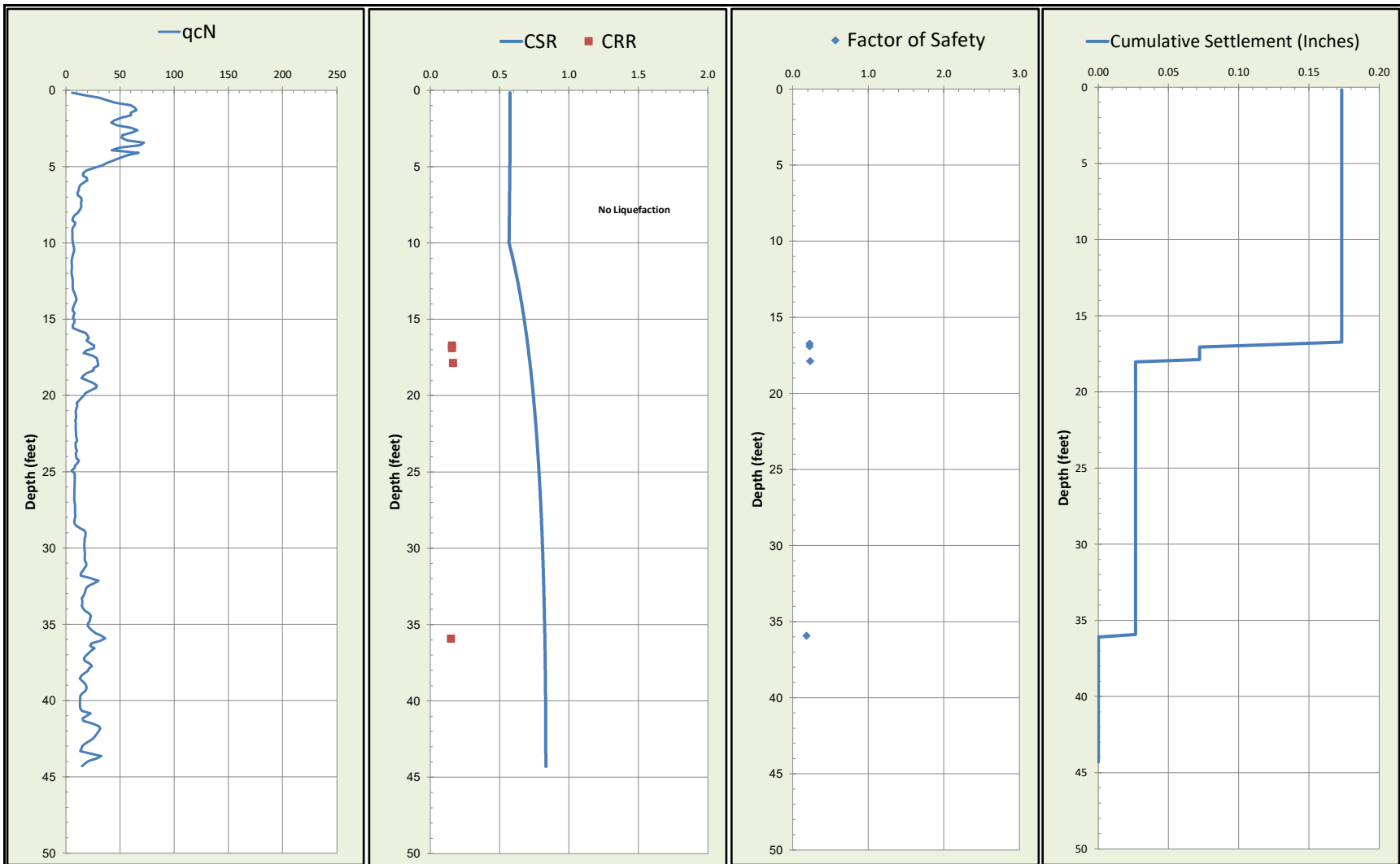
648-25-2

Figure Number

Figure D6

3/18/2022

CPT No. 6



Liquefaction Analysis Summary

511 Cozy Drive
San Jose, California

Project Number

648-25-2

Figure Number

Figure D7

3/18/2022

CPT No. 7

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