CORNERSTONE EARTH GROUP

Type of Services	Preliminary Geotechnical Investigation
Project Name	East Palo Alto Library
Location	2474 Pulgas Ave Palo Alto, California
Client	David J. Powers & Associates
Client Address	1871 The Alameda, Suite 200 San Jose, California
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DRAFT

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SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of David J. Powers & Associates for the East Palo Alto Library project in Palo Alto, 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 an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. For our use, we were provided with the following documents:

- Conceptual development plans titled "EPA Library Community Workshop, 2474 Pulgas Avenue, Palo Alto, CA, 94303," prepared by wHY, dated December 12, 2018.
- A geotechnical report titled "Geotechnical Investigation Bains Property, 2470 Pulgas Avenue, East Palo Alto, California," prepared by Lowney Associates, dated October 29, 2004.

1.1 PROJECT DESCRIPTION

The project will consist of a new two-story library on the approximately 1.1-acre parcel. The planned library structure will likely be at-grade and of wood/steel frame construction. Based on our review of the provided conceptual development plans, a new community library development is planned and will occupy the middle and northern portions of the parcel. An at-grade parking lot is planned at the southern end of the site. We understand that the final building layout and footprint have not yet been determined. Appurtenant parking, utilities, retaining walls, landscaping and other improvements necessary for site development are also planned.

Structural loads are not currently known for the proposed structure(s); however, structural loads are expected to be typical of similar type structures. Grading plans are not available at this time; however, on a preliminary basis, we assume grading will consist of cuts and fills on the order of 2 to 5 feet.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated January 19, 2022 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 and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration program is presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of five Cone Penetration Tests (CPTs) advanced on June 28, 2022. The CPTs were advance to depths of 50 to 100 feet. Seismic shear wave velocity measurements were collected from CPT-3.

The CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our explorations are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 PREVIOUS FIELD EXPLORATION

Previous field exploration was performed by Lowney Associates in September, 2004, which consisted of one hollow-stem auger boring and two Cone Penetration Test (CPT) soundings using truck-mounted exploration equipment. The approximate locations of the previous explorations are also shown on the Site Plan, Figure 2. Copies of the previous exploration logs are included in Appendix B.

1.5 PREVIOUS LABORATORY TESTING PROGRAM

The previous laboratory test program, performed by Lowney Associates, included moisture contents, dry densities, grain size analyses, washed sieve analyses, and a Plasticity Index test. Details regarding the previous laboratory program are included in Appendix B.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 and 2 site assessments; environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Monte Vista-Shannon	6.3	10.2
San Andreas (1906)	8.0	12.9
Hayward (Total Length)	10.9	17.6
Hayward (Southeast Extension)	12.6	20.3

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

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The project site is bounded by the EPACENTER building to the northwest, a parking lot and warehouse buildings to the northeast, a former rail spur to the southeast, and Pulgas Avenue to the southwest. At the time of our field exploration, the site was an undeveloped lot and the northern portion was covered in gravel and occasionally used for parking. An approximately 10-foot wide strip along the southeast portion of the site was paved with asphalt.

The site is relatively level with Elevations ranging from approximately 10 to 12 feet above mean sea level (GoogleEarth, WSG84 datum).

3.2 SUBSURFACE CONDITIONS

Below the existing ground surface, our CPTs generally encountered stiff clays with interbedded layers of silt and sand to a depth of about 30 feet below existing grades. Beneath the clays, our CPTs generally encountered larger dense sand layers at depths ranging from 30 to 38 feet below current grades in CPT-1 and CPT-2 and between 64 to 73 feet in CPT-3, underlain by stiff to hard clays and silts to the maximum depth explored of 100 feet.

Previous boring EB-1 performed by Lowney Associates in 2004 generally encountered stiff to hard lean clays to a depth of 7 feet underlain by loose clayey sand to 8 feet and medium stiff to very stiff lean clay to the terminal boring depth of 40 feet. Previous CPT-1 and CPT-2 generally encountered stiff clays and silts with interbedded sand layers to the terminal depth explored of 50 feet. A dense sand layer was encountered in previous CPT-2 between 27 to 32 feet.

3.3 GROUNDWATER

Groundwater was inferred from CPT pore pressure measurements in our CPT-1 through CPT-4 at depths of approximately 4½ to 10½ feet below current grades. Groundwater was previously encountered in exploratory boring EB-1 performed by Lowney Associates in 2004 at a depth of approximately 10½ feet. 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 (Palo Alto 7.5-minute Quadrangle, 2006) at a depth of approximately 6 feet.

Based on the above, on a preliminary basis, we recommend a design groundwater depth of 4 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration $(PGA)_M$ was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2019 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 0.616g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Palo Alto Quadrangle, 2006). Our field and laboratory programs addressed this issue by testing potentially liquefiable layers to depths of at least 50 feet and evaluating CPT data.

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

4.3.1 Background

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

4.3.2 Analysis

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

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

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 non-liquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

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

The results of our CPT analyses (CPT-1 and CPT-5) are presented on Figures 4A and 4E of this report.

4.3.3 Summary

Our preliminary analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from ¼- to 1½-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 range from ¼ to 1-inch over a horizontal distance of 30 feet. We recommend that the potential for liquefaction induced seismic settlement be further evaluated during a 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. On a preliminary basis, the work of Youd and Garris (1995) indicates that the surficial soils are potentially liquefiable and likely to cause ground deformation and surficial cracking at CPT-1. Based on our analysis, at our other exploration locations, the potential for ground deformation and surficial cracking appears low. We recommend that the potential for ground deformation and significant surficial cracking be further evaluated during the design-level 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. 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 ½-mile inland from the San Francisco Bay shoreline and is approximately 10 to 12 feet above mean sea level. In addition, the site is not located within a tsunami zone based on CGS Maps (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.

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 findings/recommendations and develop detailed recommendations for design and construction. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for liquefaction-induced settlements
- Shallow groundwater
- Presence of moderately expansive soils
- Potential for undocumented fill

5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our preliminary liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Our preliminary analysis indicates that liquefaction-induced settlement on the order of ¼- to 1½ inches could occur, resulting in differential settlement up to 1 inch. In addition, the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is moderate in CPT-1 and low for the remaining CPTs; therefore, the above estimates could be significantly increased in the vicinity of CPT-1 during a strong seismic event. On a preliminary basis, it should be feasible to support the proposed buildings on shallow foundations; however, the building foundations will need to be designed to tolerate total and differential settlement due to static loads and liquefaction-induced settlement, and designed to mitigate potential ground deformation. We recommend the potential for liquefaction and ground deformation be further evaluated during the design-level investigation. Preliminary recommendations addressing this concern are presented in the “Foundations” section.

5.1.2 Shallow Groundwater

As discussed, groundwater was inferred from CPT pore pressure measurements at depths ranging from about 4½ to 10½ feet and was previously measured at about 10½ feet in Boring EB-1 performed by Lowney Associates in 2004. We anticipate that the historical high groundwater level will be on the order of 4 feet below current grades. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement 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 provided in the "Anticipated Earthwork" section of this report.

5.1.3 Potential for Presence of Moderately Expansive Soils

Based on our experience with other sites in the area and the previous exploratory boring EB-1 by Lowney Associates, we anticipate moderately expansive surficial 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. 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.

5.1.4 Potential for Undocumented Fill

We anticipate undocumented fill associated with prior site development may be present at the site. If undocumented fill is encountered during our design-level investigation and/or construction, we recommend all undocumented fills be removed and replaced as compacted engineered fill. Additional recommendations are provided in the anticipated earthwork section of this report.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this feasibility study were based on limited site development information and limited exploration, and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter CPTs 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

6.1 ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend any fills encountered during site grading be completely removed from within new building areas and to a lateral distance of at least 5 feet beyond the new building footprint areas or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. The depth and lateral extent of undocumented fills should be further evaluated during the design level investigation. The existing AC pavement should be removed from any future at-grade building areas. Any foundations, slabs, and/or abandoned underground utilities should be removed entirely from new building areas and the resulting excavations should be backfilled with engineered fill. Furthermore, native soils disturbed during demolition of existing improvements should also be removed and replaced as engineered fill.

Shallow groundwater is inferred from our CPTs at depths ranging from 4½ to 10½ feet below grade, and we anticipate could be as shallow as 4 feet. Temporary dewatering of deeper footings and utility trenches should be anticipated.

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. Biotreatment basins should be kept at least 10 feet away from buildings and, where possible, at least 5 feet from pavements and flatwork.

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 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 mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our explorations generally encountered stiff to hard clays and medium dense to dense sand deposits to a depth of 100 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-3, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 221 meters per second (724 feet per second). Therefore, we have classified the site as Soil Classification D. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code outlined below. 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 an exception can be taken per ASCE 7-16 Section 11.4.8.*

Table 2: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.472075°
Site Longitude	-122.131017°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.5g
1-second Period Mapped Spectral Acceleration[1], S_1	0.6g
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}	1.5g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.02g*
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.0g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.68g*
Site Amplification Factor at PGA – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.616g

**Per ASCE 7-16 Section 11.4.8*

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, and on a preliminary basis, the proposed structure may be supported on shallow foundations provided recommendations in the "Anticipated Earthwork" section above are followed. Foundation alternatives provided below should be evaluated further during the design-level investigation.

8.2 SHALLOW FOUNDATIONS OVERLYING GROUND IMPROVEMENT

If determined during the design-level geotechnical investigation that estimated total and differential seismic settlements and ground deformation potential, 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), and deep dynamic compaction (DDC) or similar densification techniques, should be designed to provide vertical support through the existing soils, as well as partial mitigation of the liquefaction potential.

8.2.1 Conventional Shallow Footings

On a preliminary basis, the planned structure may be supported on conventional shallow footings overlying ground improvement. Footings should bear on engineered fill overlying ground improvement and extend at least 15 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

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

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

8.2.2 Ground Improvement

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

SECTION 9: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of David J. Powers & Associates specifically to support the design of the East Palo Alto Library project in Palo Alto, 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.

David J. Powers & Associates may have provided Cornerstone with plans, reports and other documents prepared by others. David J. Powers & Associates 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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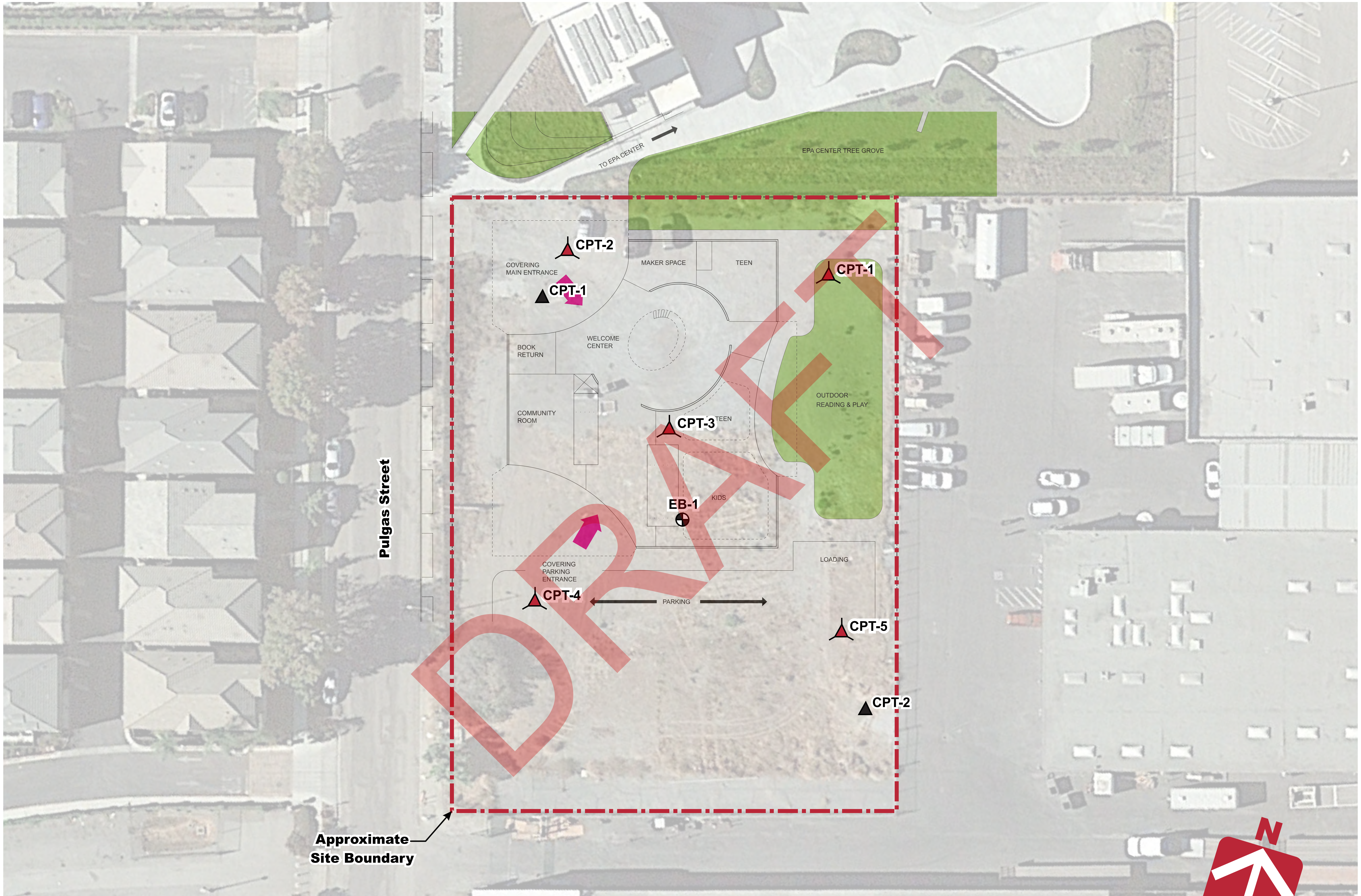
CORNERSTONE EARTH GROUP

Vicinity Map	Project Number 118-128-2
East Palo Alto Library 2474 Pulgas Avenue East Palo Alto, CA	Figure Number: Figure 1
	Date: July 2022 / Drawn By: RRN



Legend

- Approximate location of cone penetration test (CPT) (Cornerstone, current investigation)
- Approximate location of previous exploratory boring (EB) (Lowney, 2004)
- Approximate location of previous cone penetration test (CPT) (Lowney, 2004)

0 40 80
APPROXIMATE SCALE (FEET)

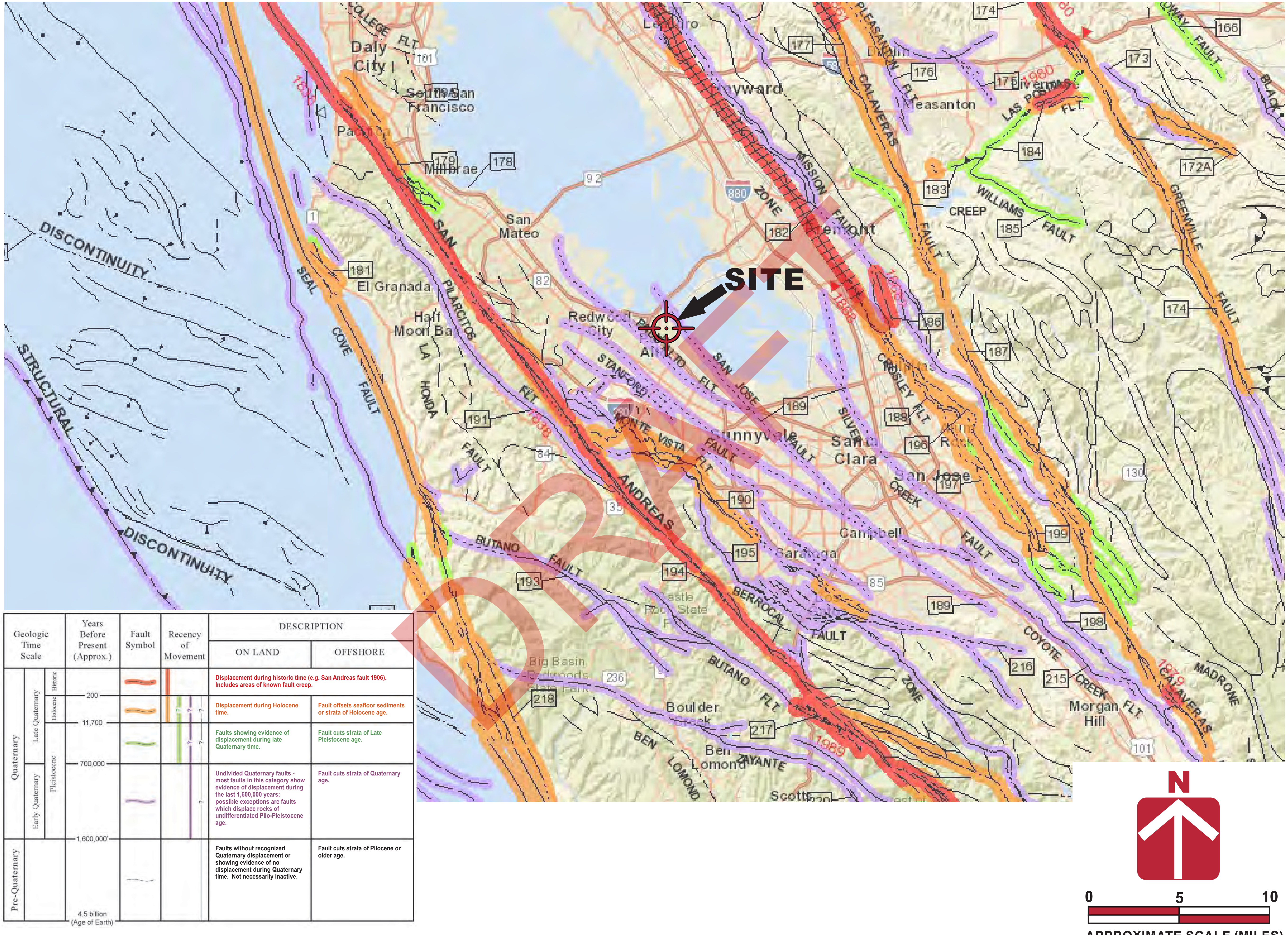
Base by Google Earth, dated 09/27/2021
Overlay by Why, EPA Library, Option 1, dated 12/12/2018

Project Number	118-128-2
Figure Number	Figure 2
Drawn By	RRN
Date	July 2022

Site Plan

East Palo Alto Library
2474 Pulgas Avenue
East Palo Alto, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene			?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene			?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary	Pleistocene			?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000				
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 Number 118-128-2

Figure Number Figure 3

Date July 2022

Drawn By RRN

Regional Fault Map

East Palo Alto Library
2474 Pulgas Avenue
East Palo Alto, CA

CORNERSTONE EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	East Palo Alto Library Prelim
Project No.	118-128-2
Project Manager	SCO

SEISMIC PARAMETERS

Controlling Fault	San Andreas (SAN+SAP)
Earthquake Magnitude (Mw)	7.6
PGA (Amax)	0.616 (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	4.5
Design Water Depth (feet)	4
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

1.48 (Inches)

TOTAL SEISMIC SETTLEMENT 1.5 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 1000.0

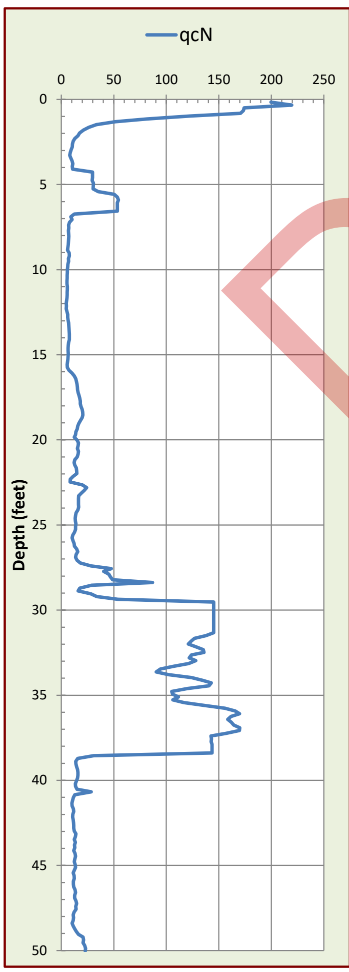
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

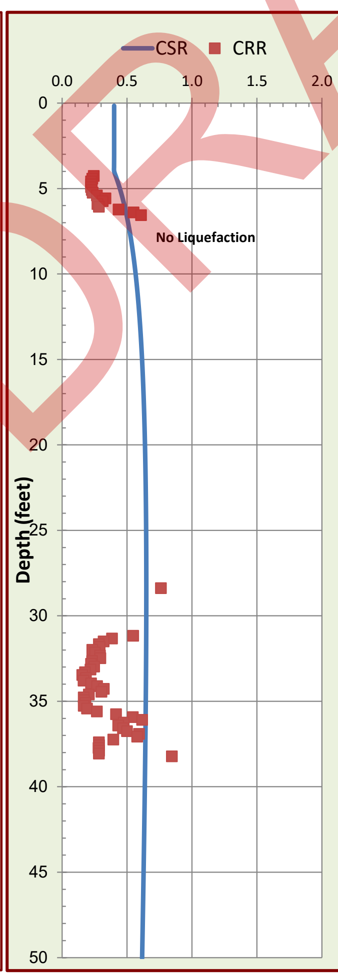
EXPECTED RANGE OF DISPLACEMENT

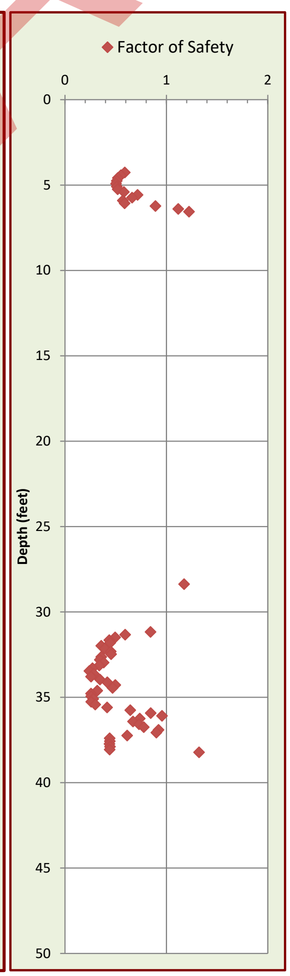
0.0 to 0.0 feet

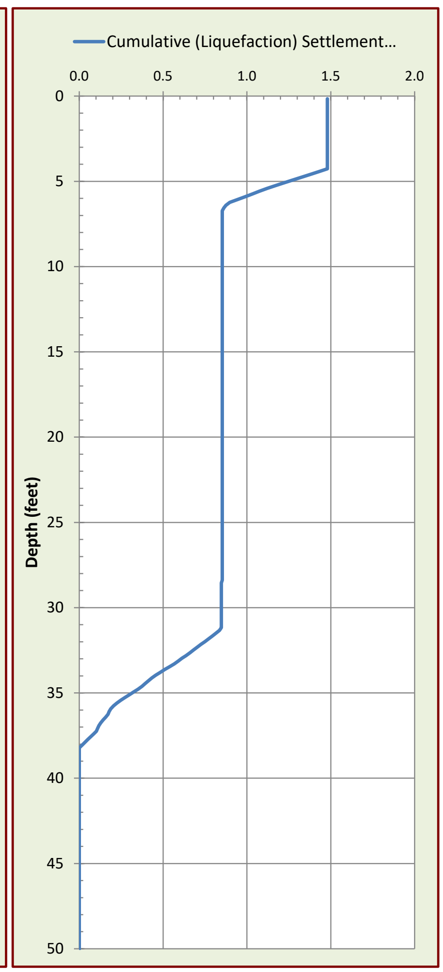
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title	East Palo Alto Library Prelim
Project No.	118-128-2
Project Manager	SCO

SEISMIC PARAMETERS

Controlling Fault	San Andreas (SAN+SAP)
Earthquake Magnitude (Mw)	7.6
PGA (Amax)	0.616 (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	7
Design Water Depth (feet)	4
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 FEET

0.01 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.62 (Inches)

TOTAL SEISMIC SETTLEMENT 0.6 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 1000.0

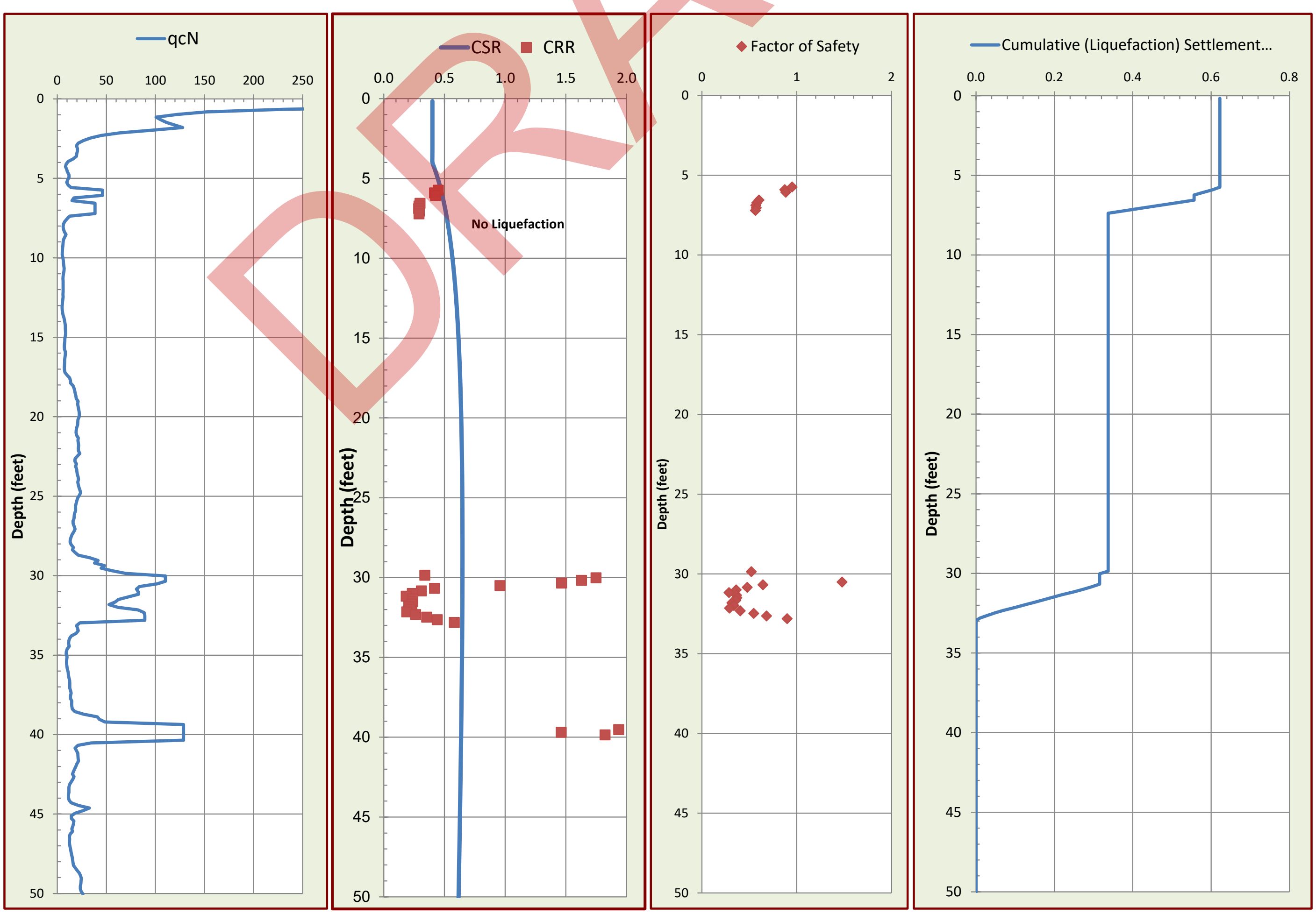
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

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	East Palo Alto Library Prelim
Project No.	118-128-2
Project Manager	SCO

SEISMIC PARAMETERS

Controlling Fault	San Andreas (SAN+SAP)	
Earthquake Magnitude (Mw)	7.6	
PGA (Amax)	0.616	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	7.7
Design Water Depth (feet)	4
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM	4	FEET
	0.00	(Inches)
LIQUEFACTION SETTLEMENT FROM	50	FEET
	0.59	(Inches)
TOTAL SEISMIC SETTLEMENT	0.6	INCHES

POTENTIAL LATERAL DISPLACEMENT

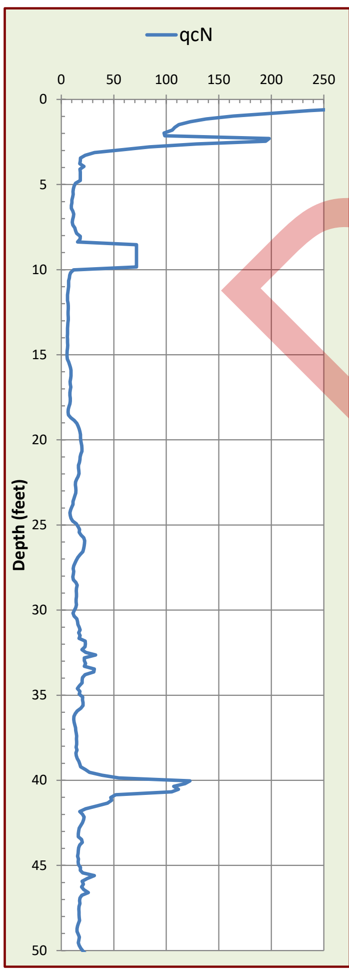
LDI[2]	0.00	L/H	1000.0
LDI[1] Corrected for Distance	0.00	(4 < L/H < 40)	

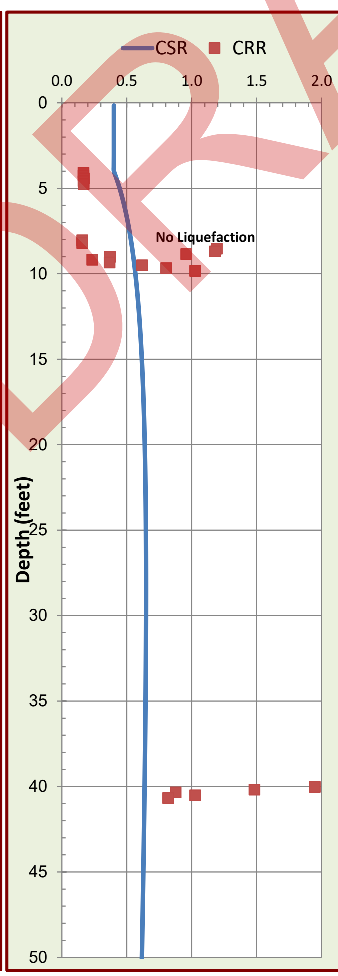
EXPECTED RANGE OF DISPLACEMENT

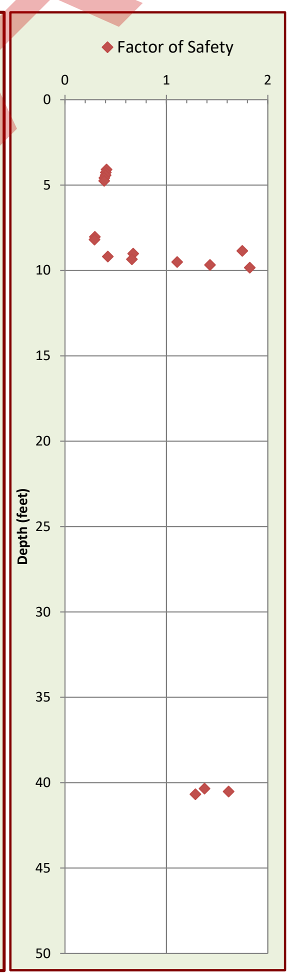
0.0 to 0.0 feet

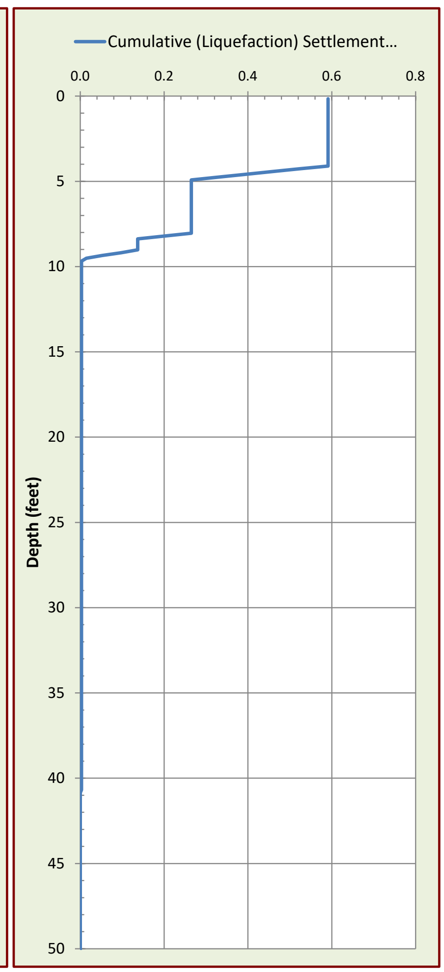
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	East Palo Alto Library Prelim
Project No.	118-128-2
Project Manager	SCO

SEISMIC PARAMETERS

Controlling Fault	San Andreas (SAN+SAP)	
Earthquake Magnitude (Mw)	7.6	
PGA (Amax)	0.616	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	10.5
Design Water Depth (feet)	4
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 4 FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.46 (Inches)

TOTAL SEISMIC SETTLEMENT 0.5 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 L/H 1000.0

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

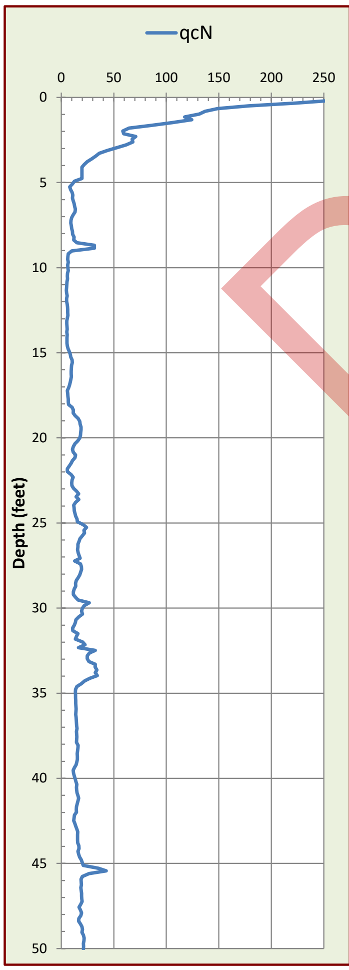
EXPECTED RANGE OF DISPLACEMENT

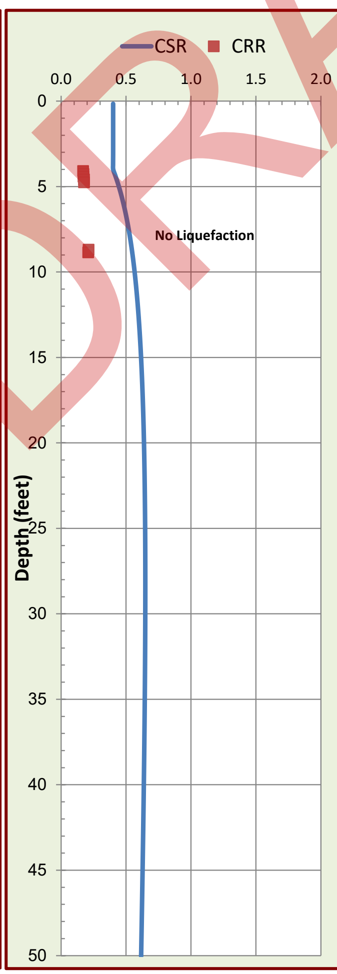
0.0 to 0.0 feet

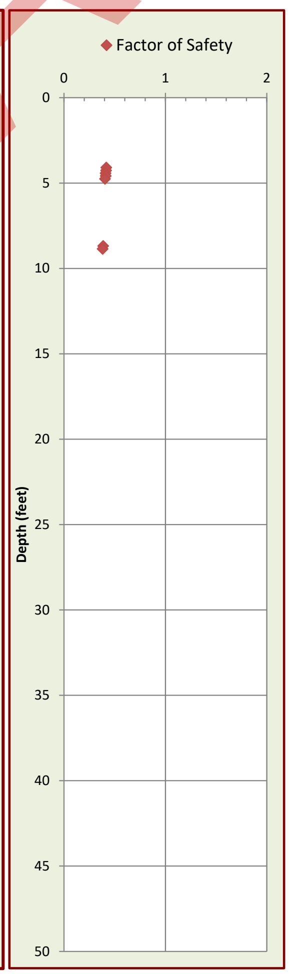
[1]Not Valid for L/H Values < 4 and > 40.

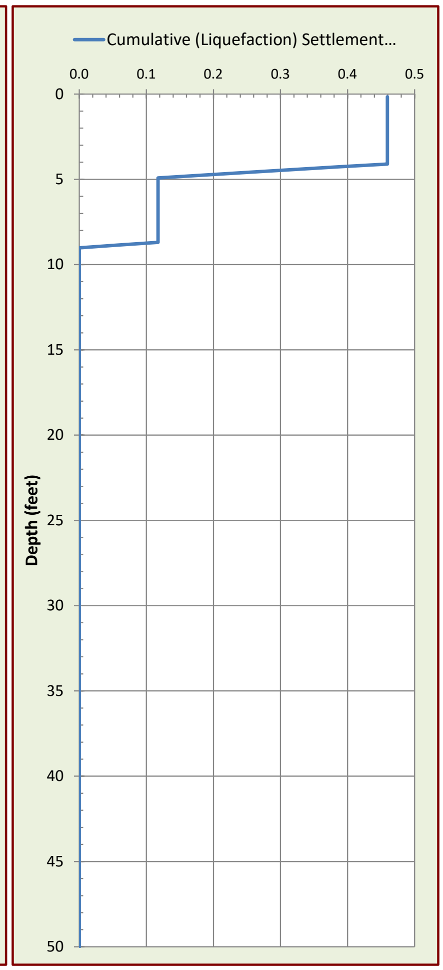
[2]LDI Values Only Summed to 2H Below Grade.

DRAFT









CORNERSTONE EARTH GROUP

FIGURE	4E
CPT NO.	5

PROJECT/CPT DATA

Project Title	East Palo Alto Library Prelim
Project No.	118-128-2
Project Manager	SCO

SEISMIC PARAMETERS

Controlling Fault	San Andreas (SAN+SAP)	
Earthquake Magnitude (Mw)	7.6	
PGA (Amax)	0.616	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	10.5
Design Water Depth (feet)	4
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM	4	FEET
	0.00	(Inches)
LIQUEFACTION SETTLEMENT FROM	50	FEET
	0.18	(Inches)
TOTAL SEISMIC SETTLEMENT	0.2	INCHES

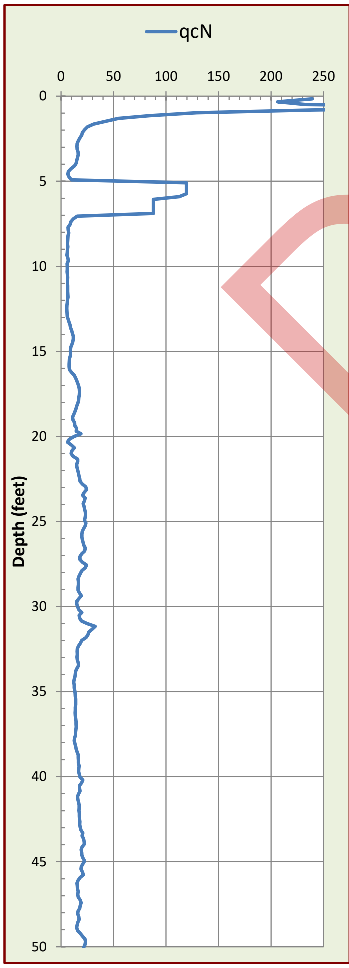
POTENTIAL LATERAL DISPLACEMENT

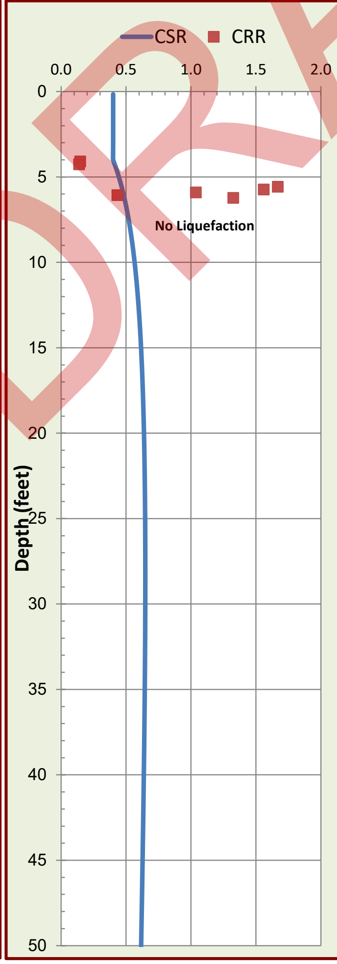
LDI[2]	0.00	L/H	1000.0
LDI[1] Corrected for Distance	0.00	(4 < L/H < 40)	

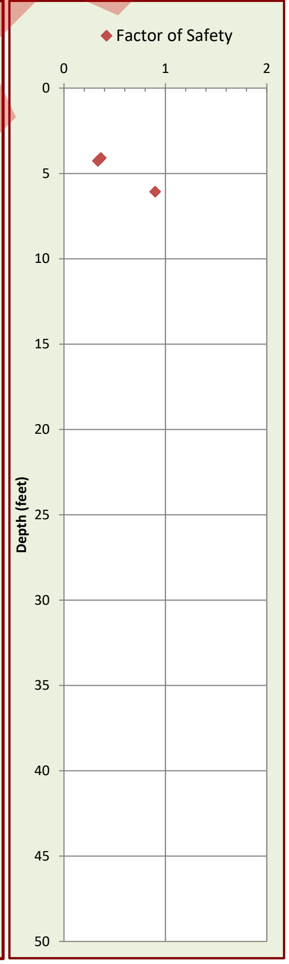
EXPECTED RANGE OF DISPLACEMENT

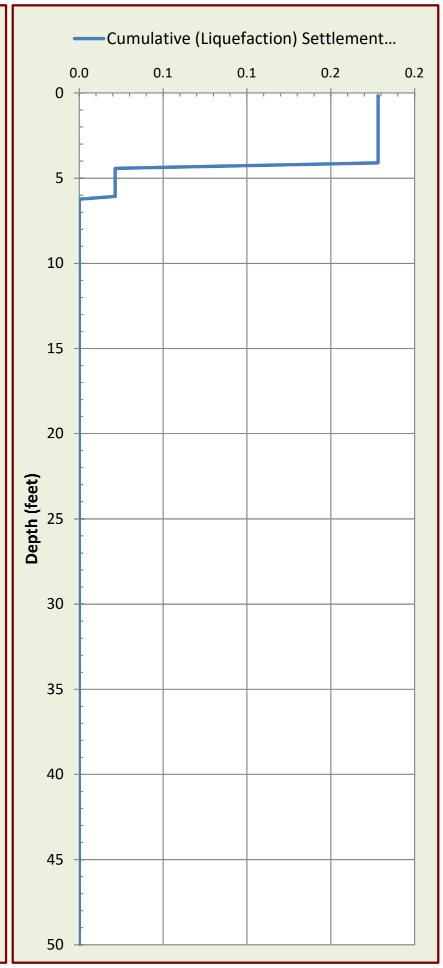
0.0 to 0.0 feet

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









CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using 25-ton truck-mounted Cone Penetration Test equipment. Five CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on June 28, 2022, to depths ranging from 50 to 100 feet. The approximate locations of CPTs are shown on the Site Plan, Figure 2.

CPT locations were approximated using existing site boundaries, and other site features as references. CPT elevations were not determined. The locations 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.

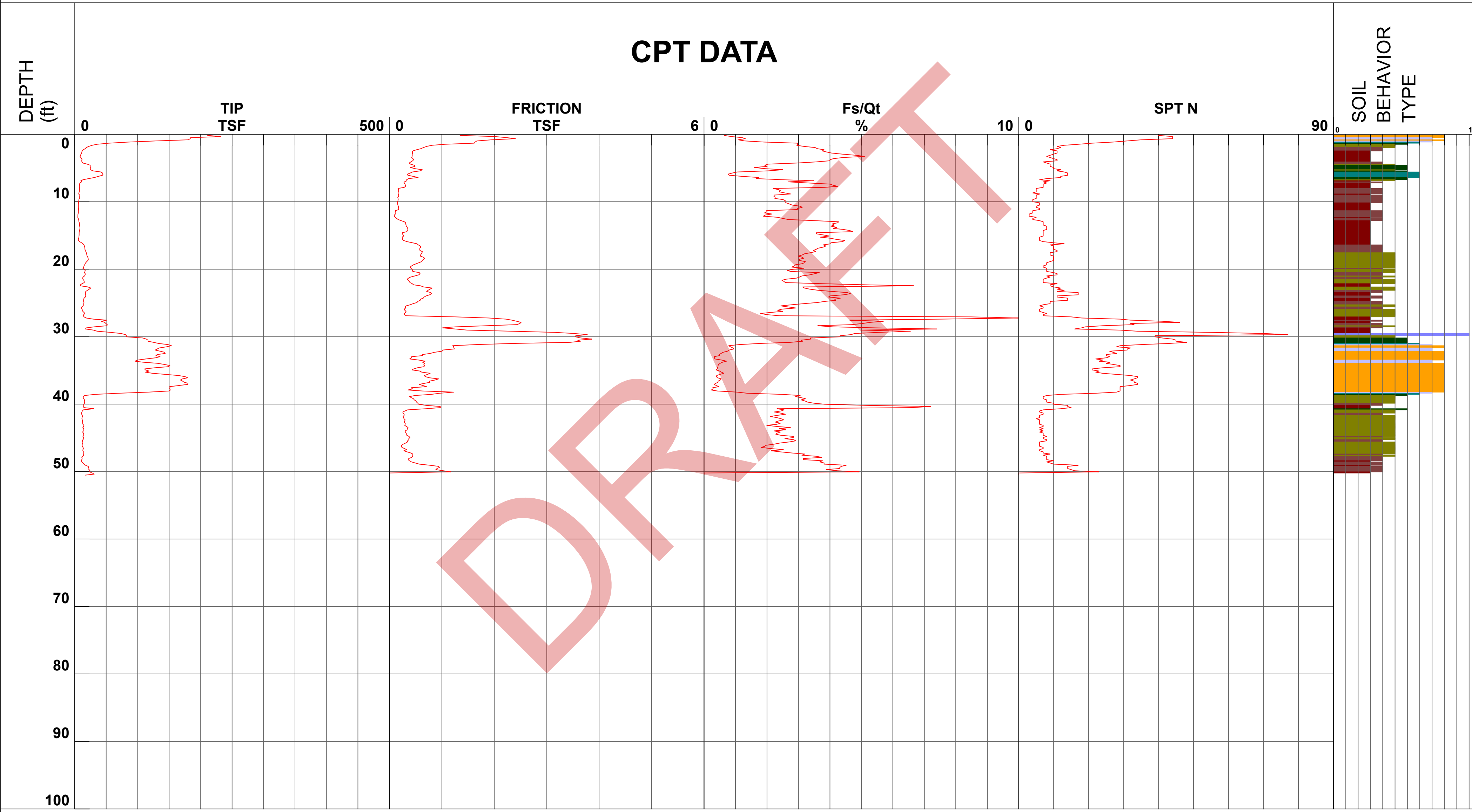
Attached CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these CPT locations. The passage of time may result in altered subsurface conditions 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	East Palo Alto Library Prelim GI	Operator	AJ-IM	Filename	SDF(937).cpt
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	6/28/2022 2:02:11 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		6.00 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 (*)

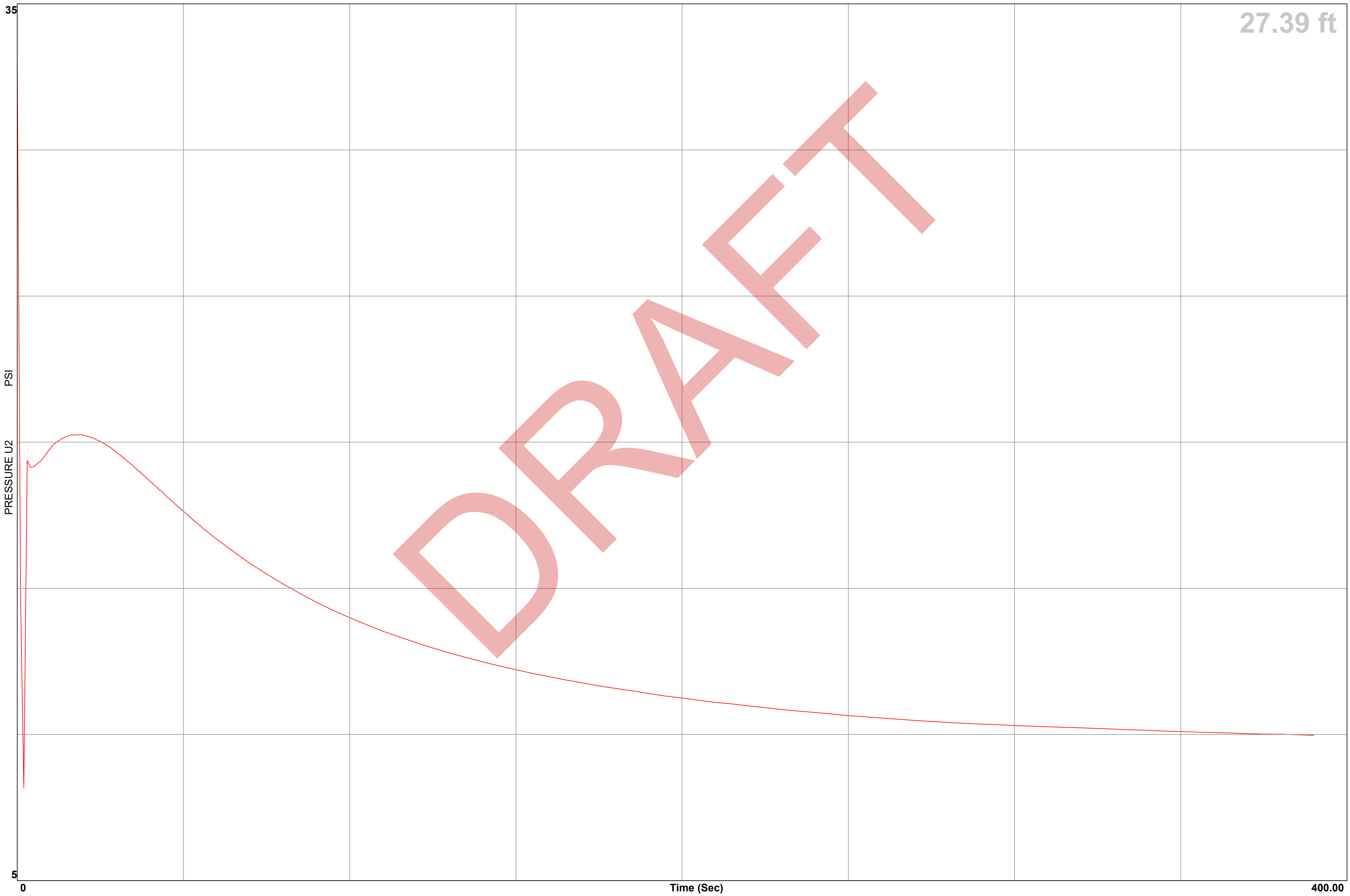
Cone Size 15cm squared

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

Cornerstone Earth Group

Location	East Palo Alto Library Prelim GI	Operator	AJ-IM
Job Number	118-128-2	Cone Number	DDG1587
Hole Number	CPT-01	Date and Time	6/28/2022 2:02:11 PM
Equilized Pressure	9.8	EST GW Depth During Test	4.5

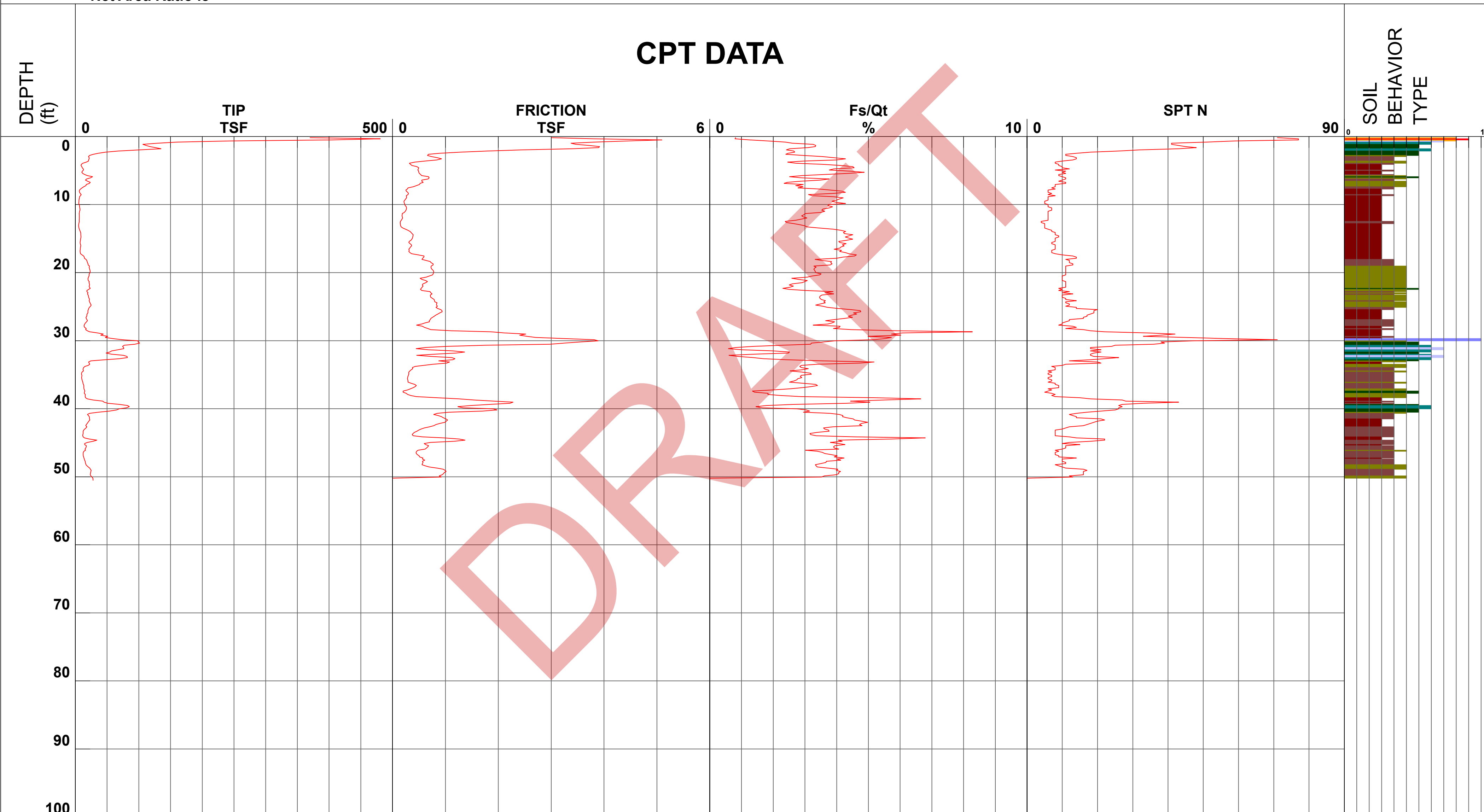
GPS



Cornerstone Earth Group

Project	East Palo Alto Library Prelim GI	Operator	AJ-IM	Filename	SDF(938).cpt
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	6/28/2022 2:41:56 PM	Maximum Depth	50.52 ft
EST GW Depth During Test			9.00 ft		

Net Area Ratio .8



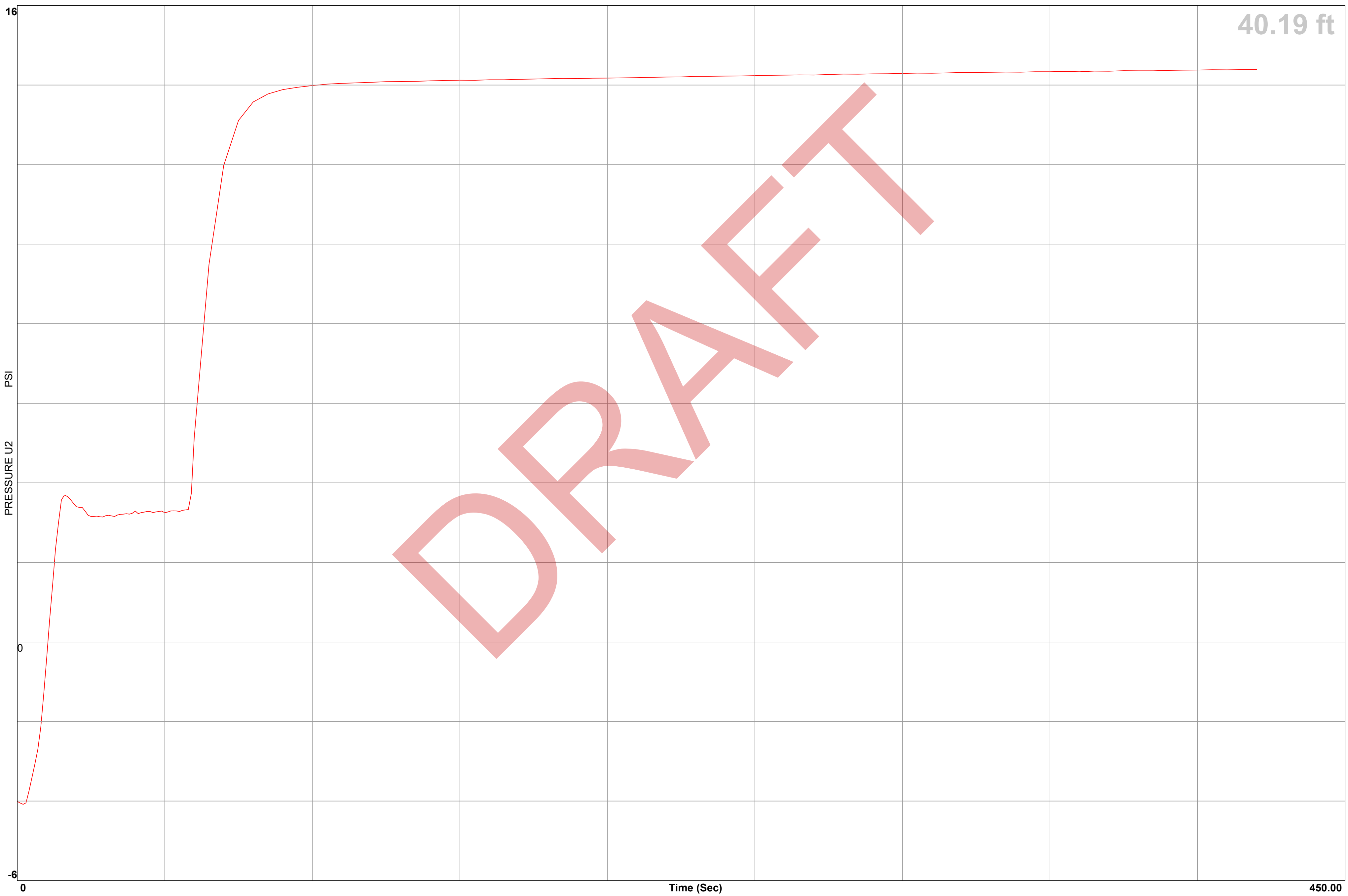
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

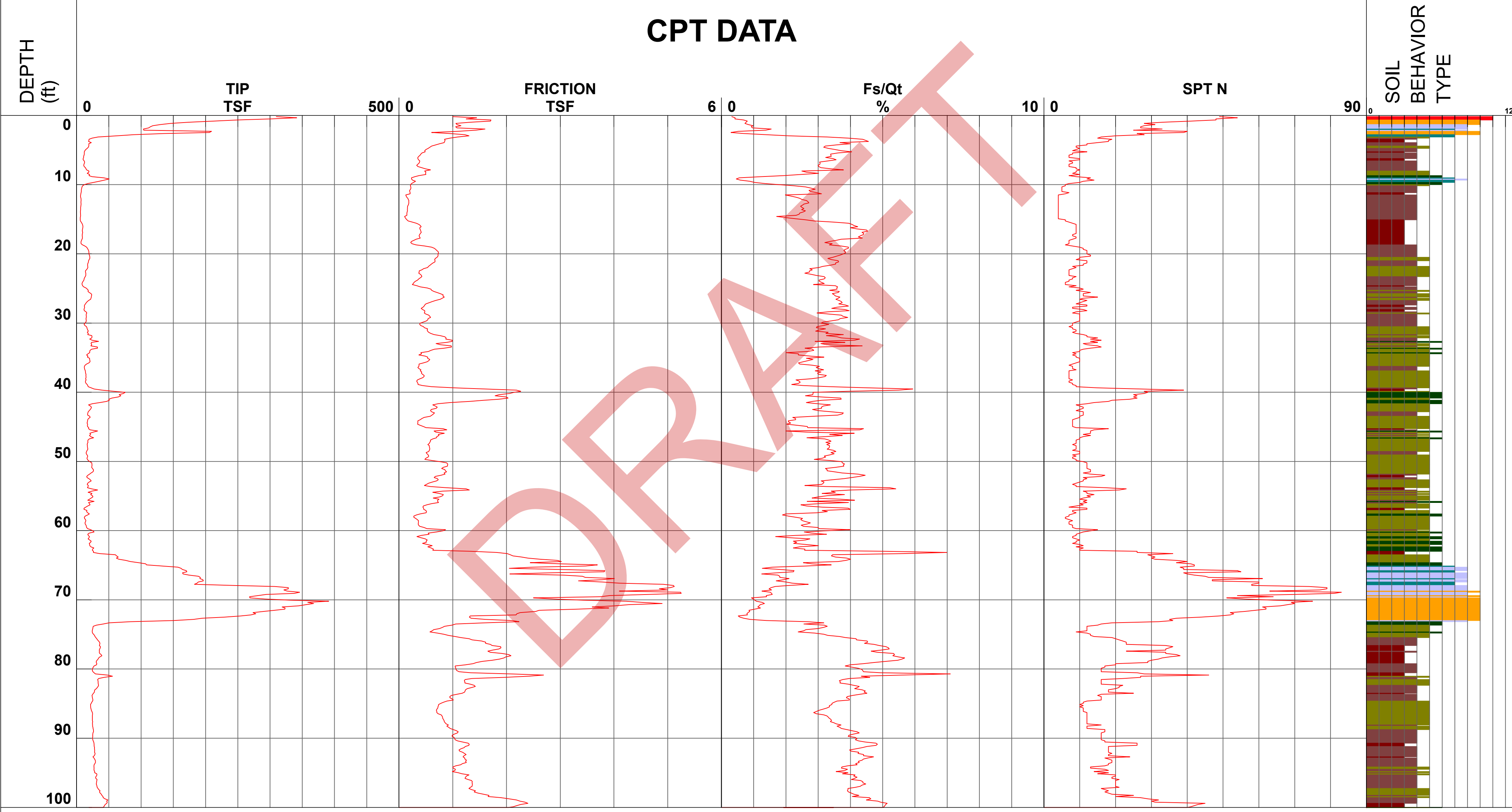
Location	East Palo Alto Library Prelim GI	Operator	AJ-IM		
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	6/28/2022 2:41:56 PM		
Equilized Pressure	14.3	EST GW Depth During Test	7.0		



Cornerstone Earth Group

Project	East Palo Alto Library Prelim GI	Operator	AJ-IM	Filename	SDF(934).cpt
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	6/28/2022 10:33:49 AM	Maximum Depth	101.38 ft
EST GW Depth During Test			9.00 ft		

Net Area Ratio .8

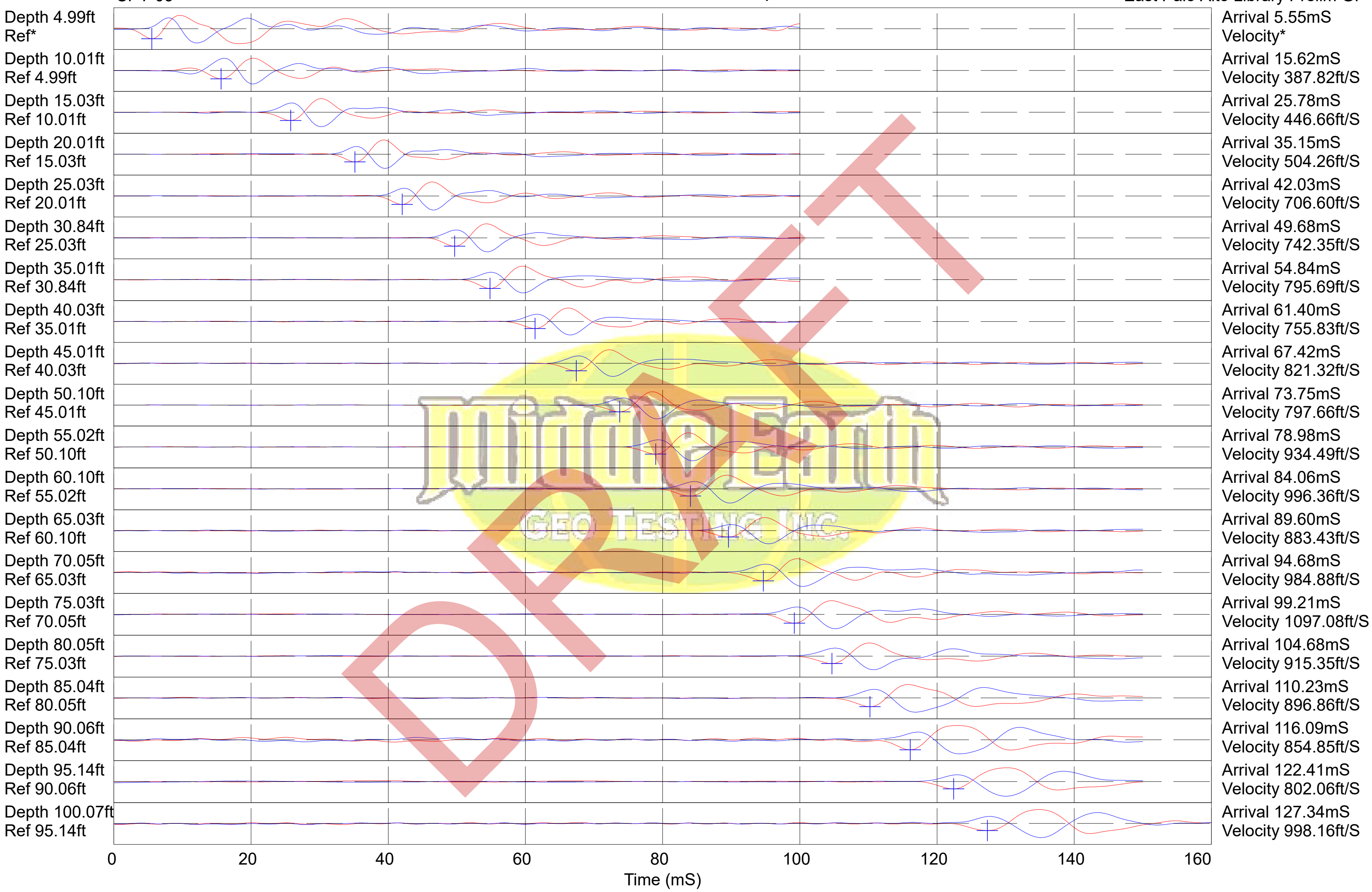


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

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

CPT-03 | Cornerstone Earth Group | East Palo Alto Library Prelim GI



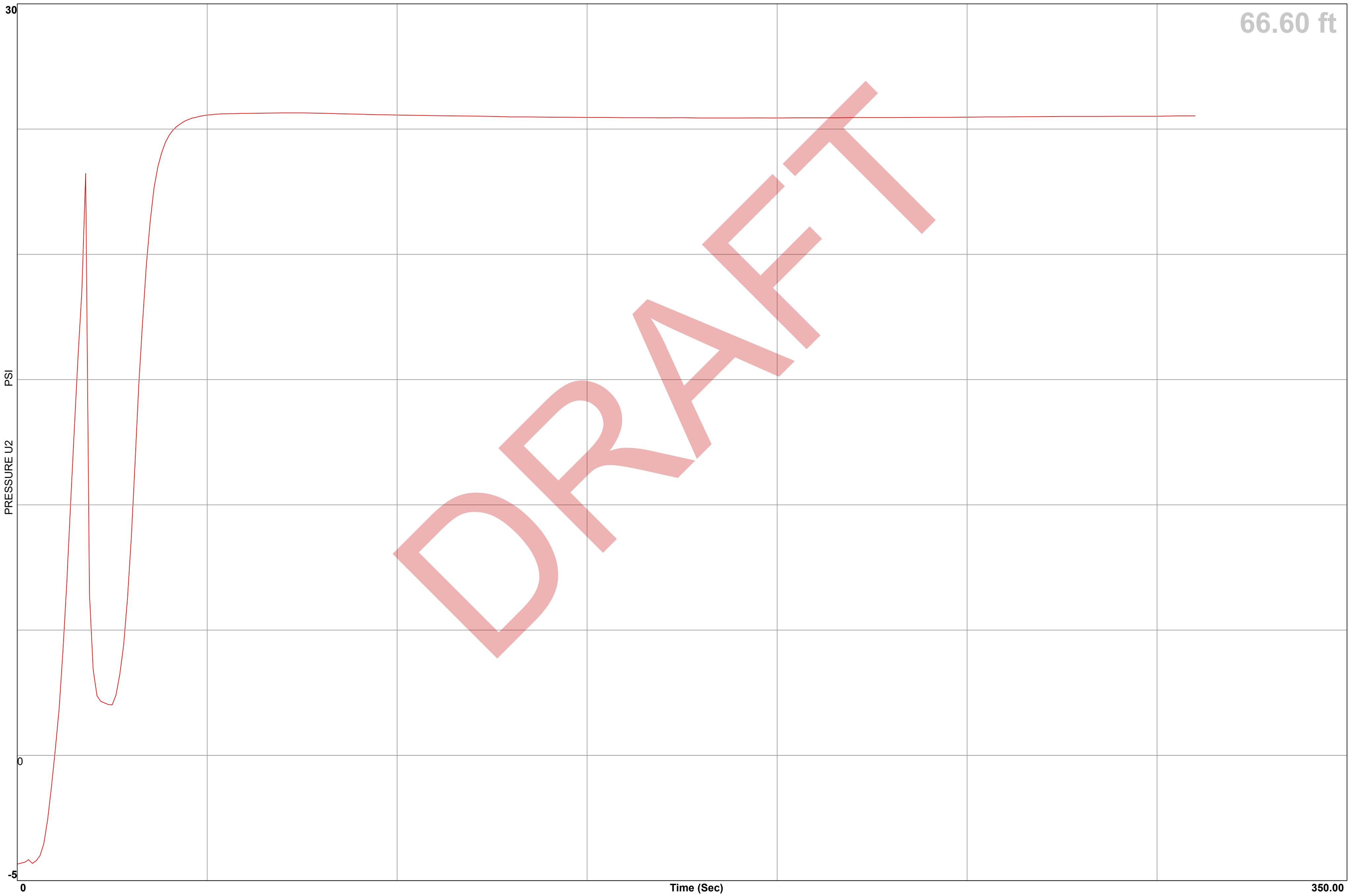
Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Cornerstone Earth Group

Location	East Palo Alto Library Prelim GI	Operator	AJ-IM
Job Number	118-128-2	Cone Number	DDG1587
Hole Number	CPT-03	Date and Time	6/28/2022 10:33:49 AM
Equilized Pressure	25.4	EST GW Depth During Test	7.7

GPS

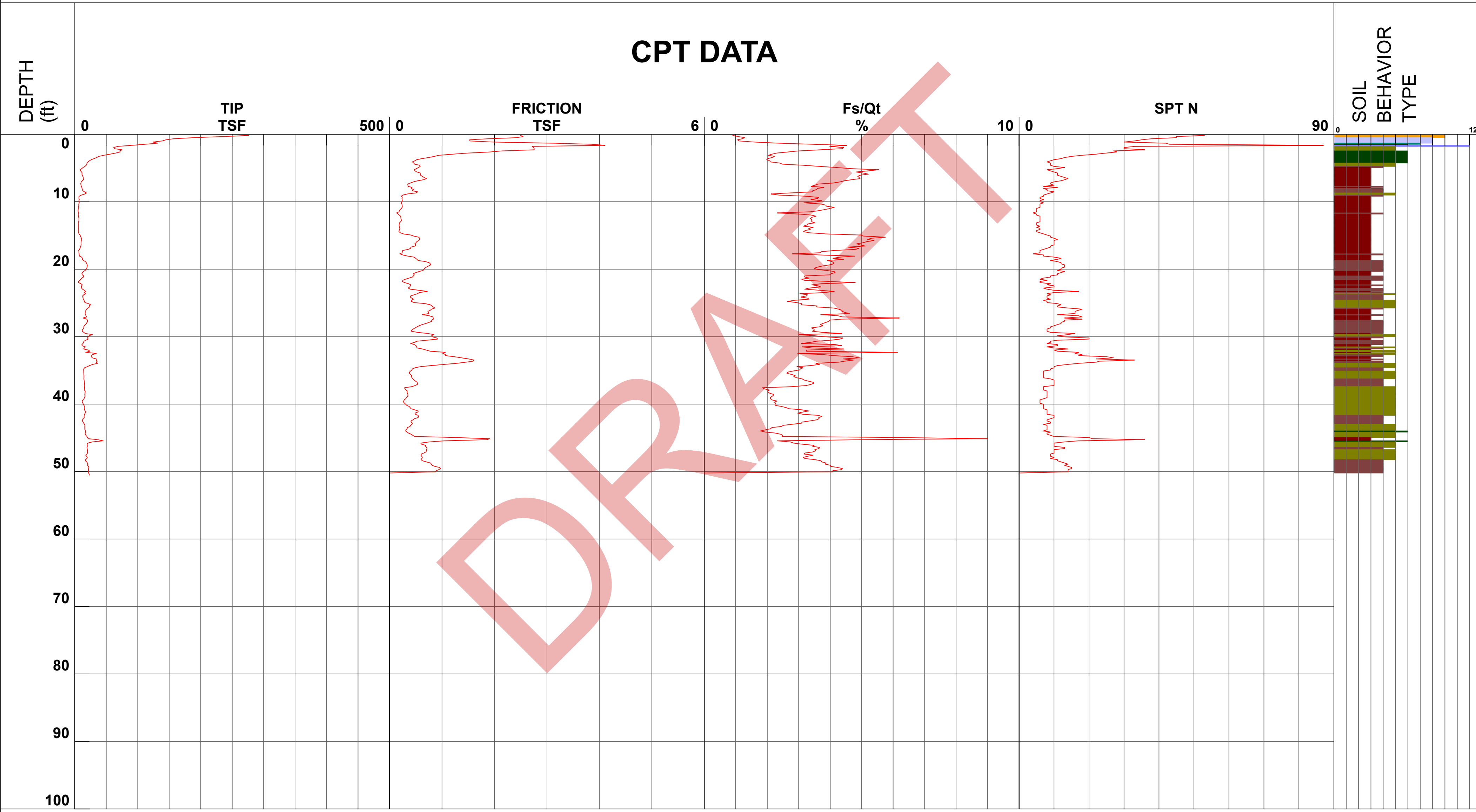


Cornerstone Earth Group

Project	East Palo Alto Library Prelim GI	Operator	AJ-IM	Filename	SDF(935).cpt
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	6/28/2022 12:36:18 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		9.00 ft			

Net Area Ratio .8

CPT DATA



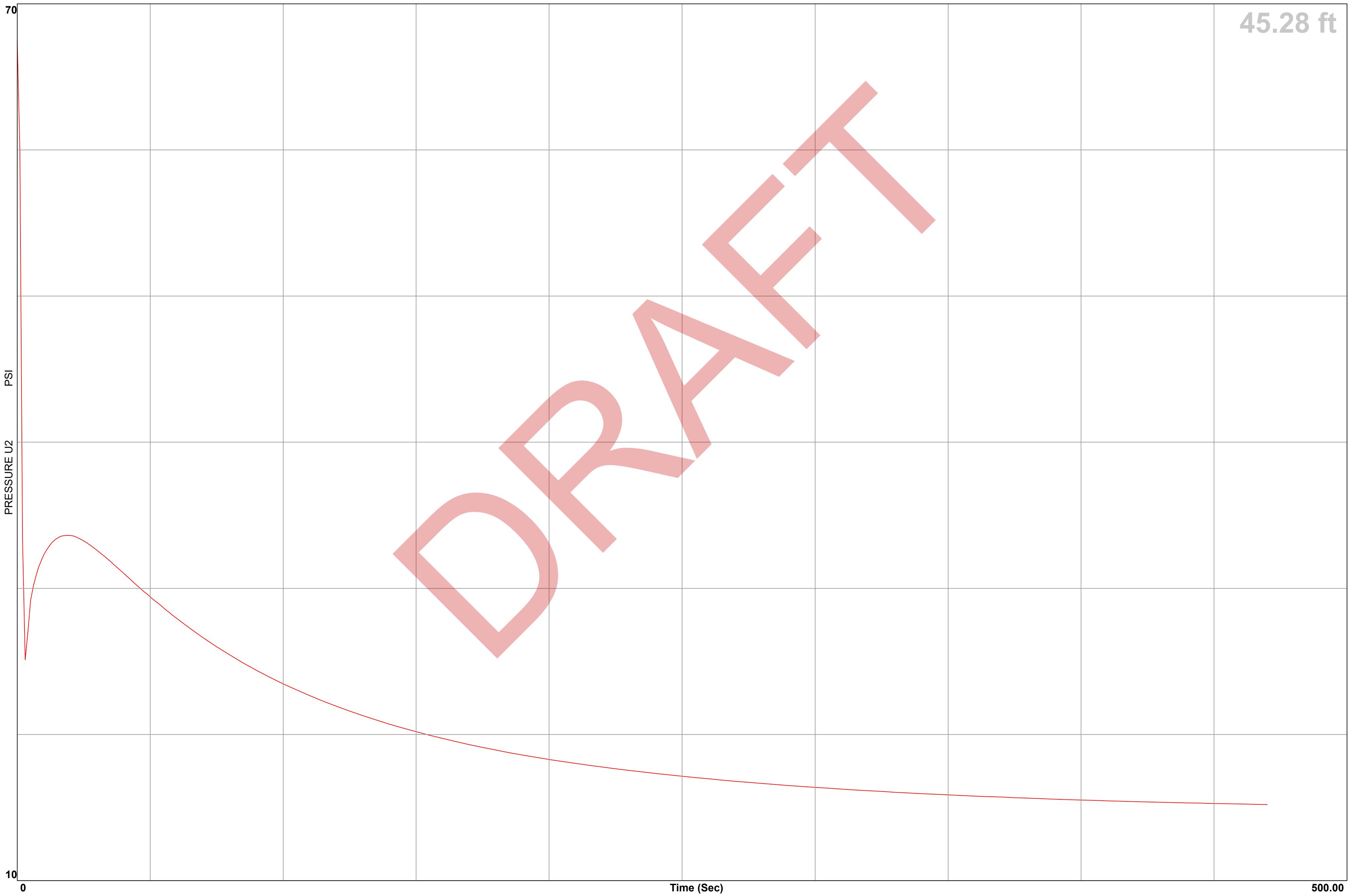
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

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

Cornerstone Earth Group

Location	East Palo Alto Library Prelim GI	Operator	AJ-IM		
Job Number	118-128-2	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	6/28/2022 12:36:18 PM		
Equilized Pressure	15.0	EST GW Depth During Test	10.5		



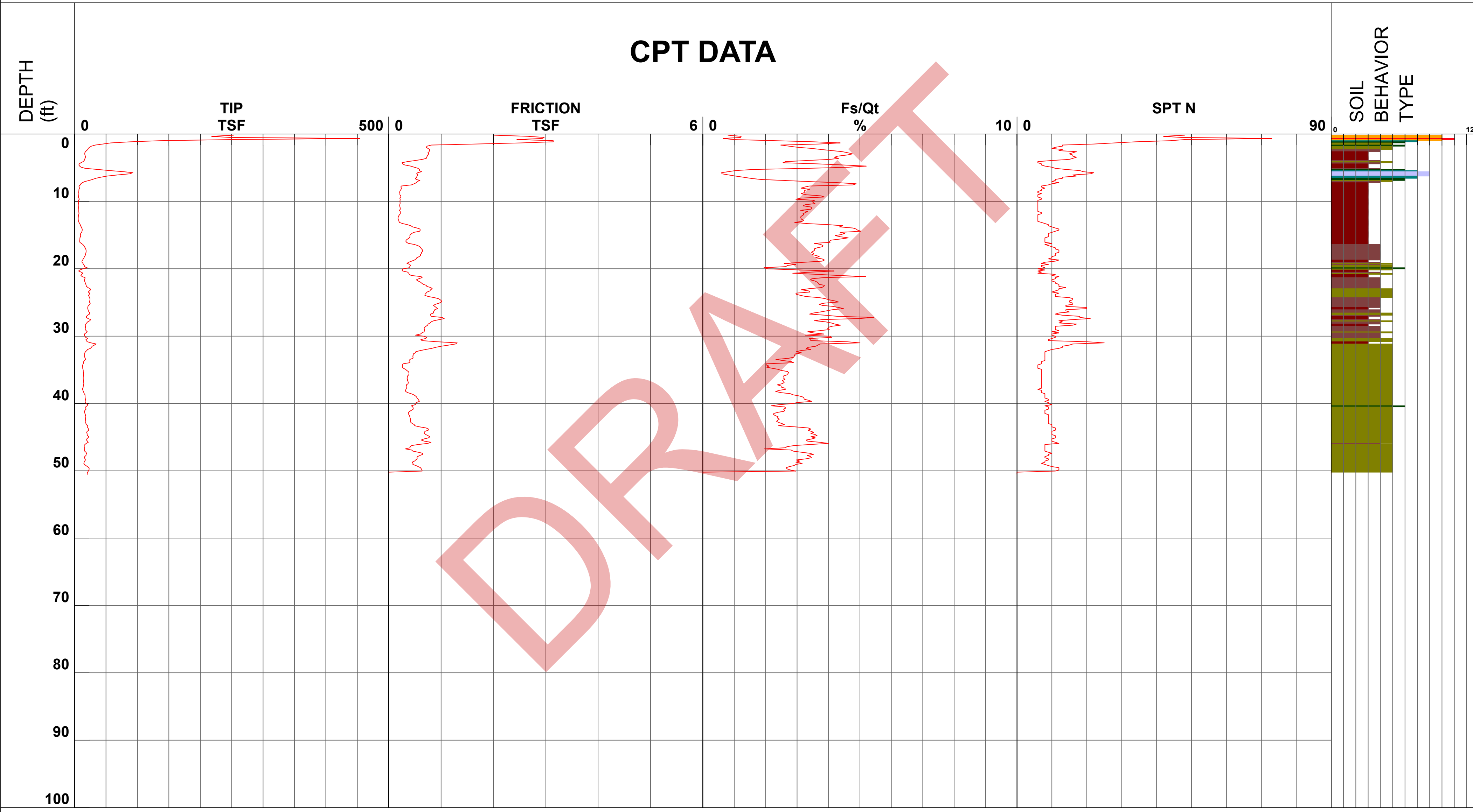
Cornerstone Earth Group

Project	East Palo Alto Library Prelim GI	Operator	AJ-IM
Job Number	118-128-2	Cone Number	DDG1587
Hole Number	CPT-05	Date and Time	6/28/2022 1:18:42 PM
EST GW Depth During Test			9.00 ft

Filename	SDF(936).cpt
GPS	
Maximum Depth	50.52 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

APPENDIX B: PREVIOUS LABORATORY TEST PROGRAM BY OTHERS

APPENDIX A
FIELD INVESTIGATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted hollow-stem auger drilling and cone penetration test (CPT) equipment. One 8-inch-diameter exploratory boring was drilled on September 17, 2004 to a maximum depth of 40 feet. Two CPTs were advanced to a maximum depth of 50 feet on September 16, 2004. CPT data was obtained at 0.16 feet intervals, and consisted of cone tip resistance, sleeve friction and other parameters. The data obtained was correlated using the references cited, to determine the indicated soil type, shear strength, equivalent Standard Penetration Test (SPT), N-value (blows per foot), and other parameters. The approximate locations of the boring and CPTs are shown on the Site Plan, Figure 3. The soils encountered were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Our boring and CPT logs, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of the boring and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the boring and CPTs were not determined. The locations of the boring and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and SPT 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. The various samplers are denoted at the appropriate depth on the boring log and symbolized as shown on Figure A-1.

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

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

* * * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt-mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50 %		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50 %		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"	
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			
	0.08	0.4	2	5	19	76mm		

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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FIGURE A-1

EXPLORATORY BORING: EB-1

DRILL RIG: MOBILE B-56
BORING TYPE: 8 INCH HOLLOW STEM AUGER
LOGGED BY: MQL
START DATE: 9-17-04 FINISH DATE: 9-17-04

PROJECT NO: 2047-1C
PROJECT: 2470 PULGAS AVENUE
LOCATION: EAST PALO ALTO, CA
COMPLETION DEPTH: 40.0 FT.

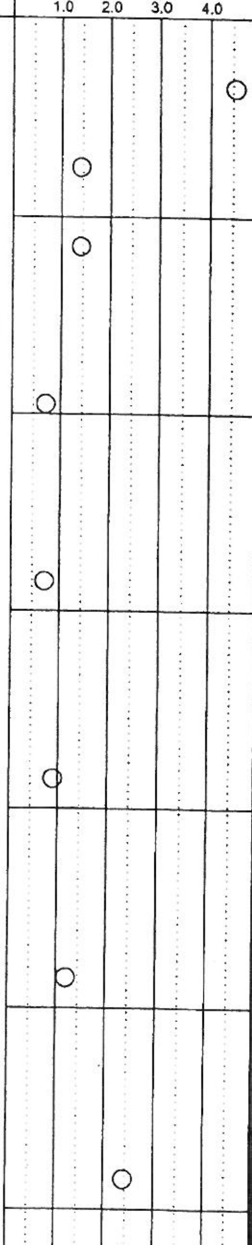
This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description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

ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE

SURFACE ELEVATION:

Depth (ft)	Material Description and Remarks	Soil Type	Penetration Resistance (blows/ft.)	Moisture Content (%)	Dry Density (PCF)	Percent Passing No. 200 Sieve
0	**POORLY GRADED SAND WITH SILT AND GRAVEL (GP-GM) [FILL]** medium dense, dry, gray-brown, fine to medium sand, fine gravel	FILL, SP-SM				
	LEAN CLAY (CL) hard, moist, dark brown, some fine sand, low to moderate plasticity	CL	10	15	99	
	Plasticity Index = 18, Liquid Limit = 36		8	19	101	
5	**LEAN CLAY WITH SAND (CL)** stiff, moist, brown, fine sand, low plasticity	CL	11	20	105	
	CLAYEY SAND (SC) loose, wet, light brown, fine to coarse sand, trace fine gravel	SC	9	24 29	94	14
	SANDY LEAN CLAY (CL) medium stiff, wet, brown, fine sand, low plasticity	CL				
10	**LEAN CLAY TO FAT CLAY (CL-CH)** medium stiff, wet, brown, trace fine sand, moderate to high plasticity	CL-CH	7	30	91	
15			9	41	75	
	FAT CLAY (CH) medium stiff, wet, dark gray, high plasticity	CH				
20			10	30	87	
	LEAN CLAY WITH SAND (CL) stiff to very stiff, moist, brown and light brown mottled, fine sand, low to moderate plasticity	CL				
25			9			
30			18	22	103	

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression



Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 10.5 FEET

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EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-56	PROJECT NO: 2047-1C
BORING TYPE: 8 INCH HOLLOW STEM AUGER	PROJECT: 2470 PULGAS AVENUE
LOGGED BY: MQL	LOCATION: EAST PALO ALTO, CA
START DATE: 9-17-04 FINISH DATE: 9-17-04	COMPLETION DEPTH: 40.0 FT.

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

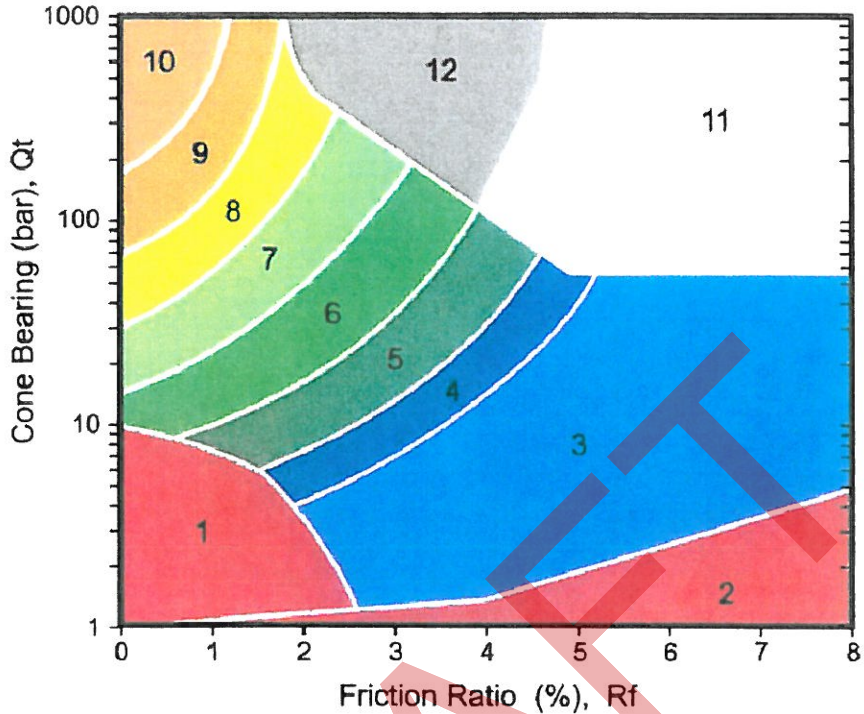
Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30		**LEAN CLAY WITH SAND (CL)** stiff to very stiff, moist, brown and light brown mottled, fine sand, low to moderate plasticity						
	35			CL	13				
	40		**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand	SC	19		20		28
			Bottom of Boring at 40 feet						
	45								
	50								
	55								
	60								

GROUND WATER OBSERVATIONS:

⊻ : FREE GROUND WATER MEASURED DURING DRILLING AT 10.5 FEET

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

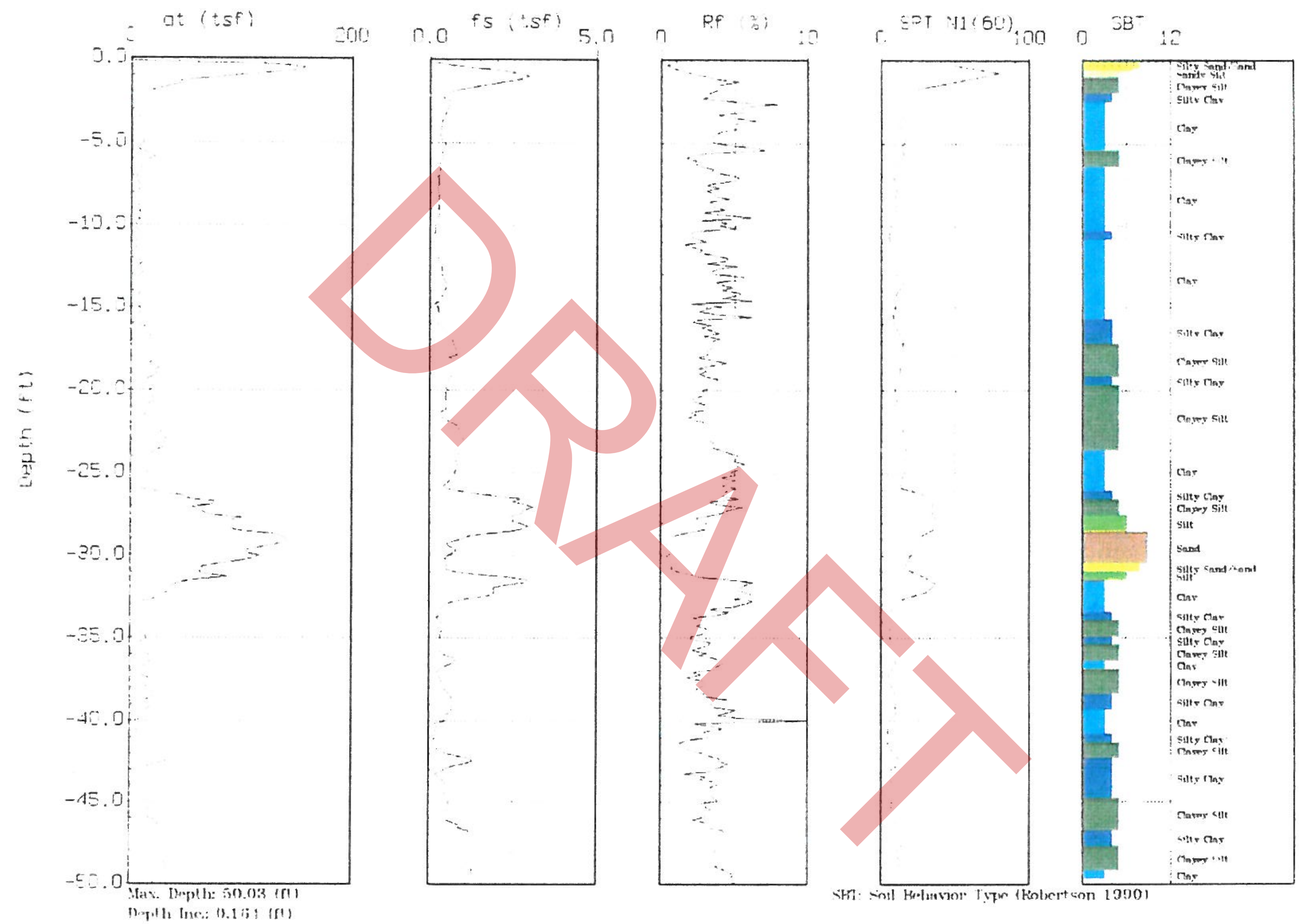


Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

Robertson (1990)

KEY TO CONE PENETROMETER TEST



10/04*EB

CONE PENETRATION TEST - CPT-1 (1 of 2)

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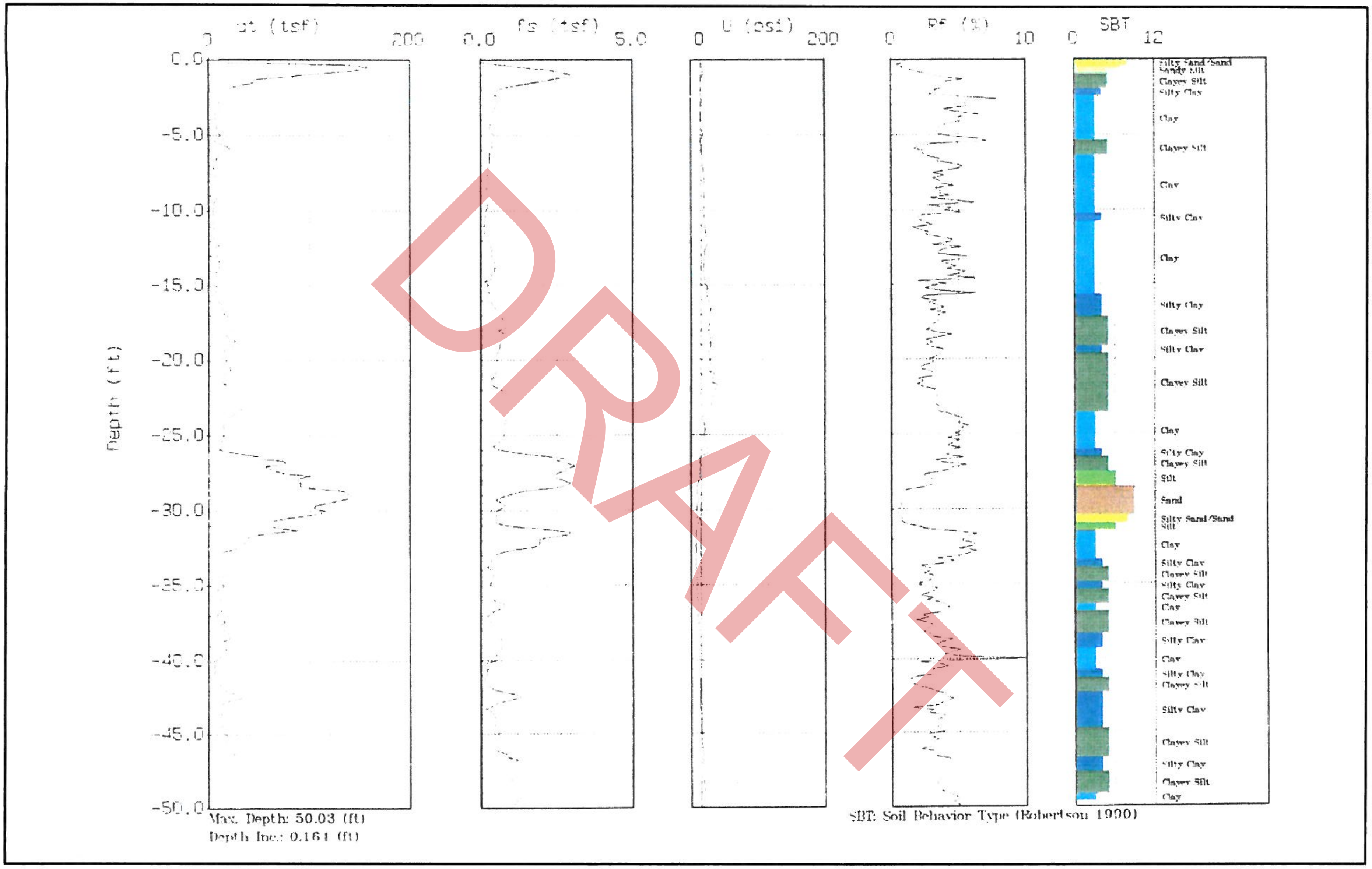
CPT-1 (1 of 2)
2047-1C



10/04*EB

CONE PENETRATION TEST - CPT-1 (2 of 2)

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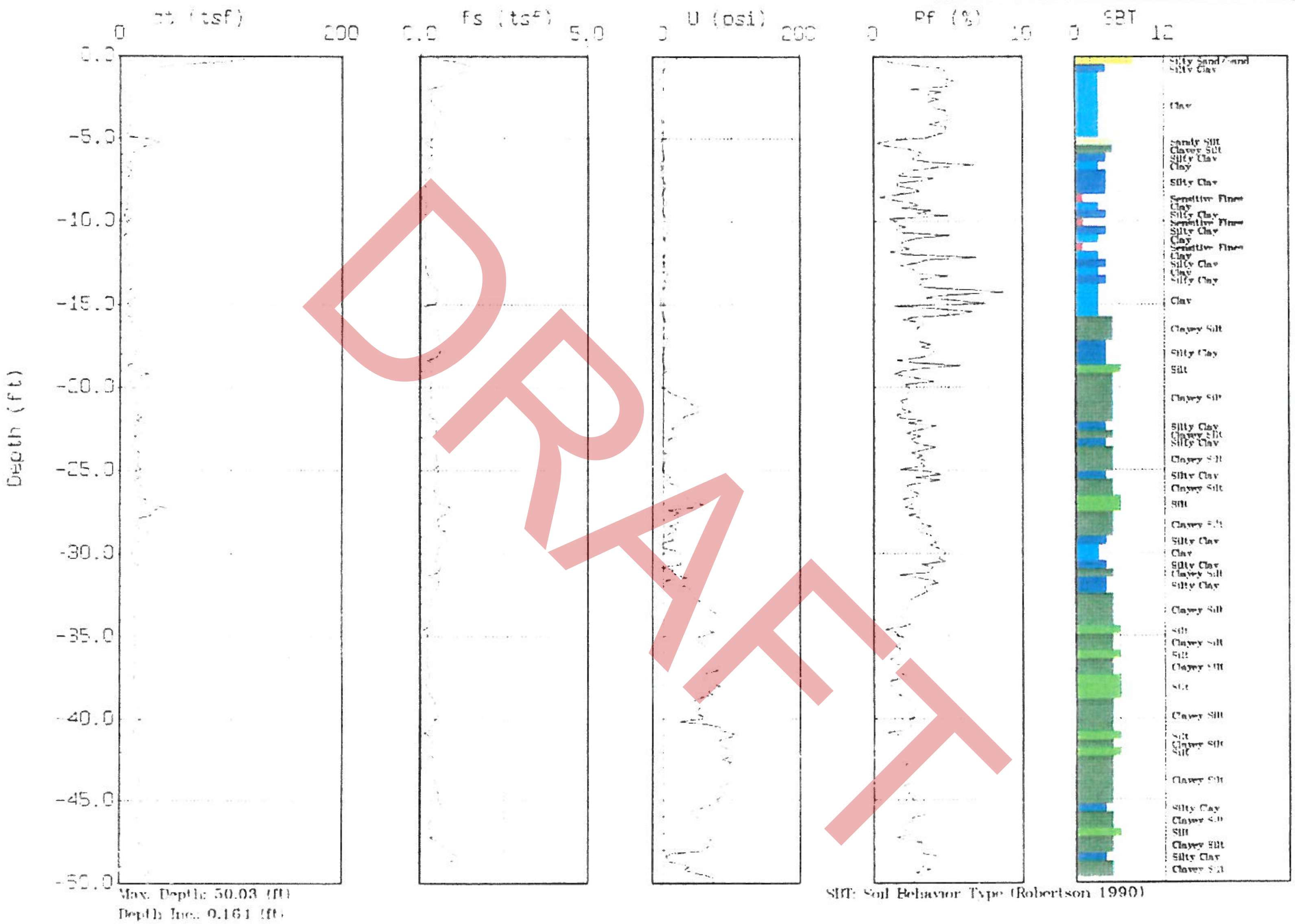
CPT-1 (2 of 2)
2047-1C



10/04*EB

CONE PENETRATION TEST - CPT-2 (2 of 2)

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CPT-2 (2 of 2)
2047-1C

APPENDIX B
LABORATORY PROGRAM

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

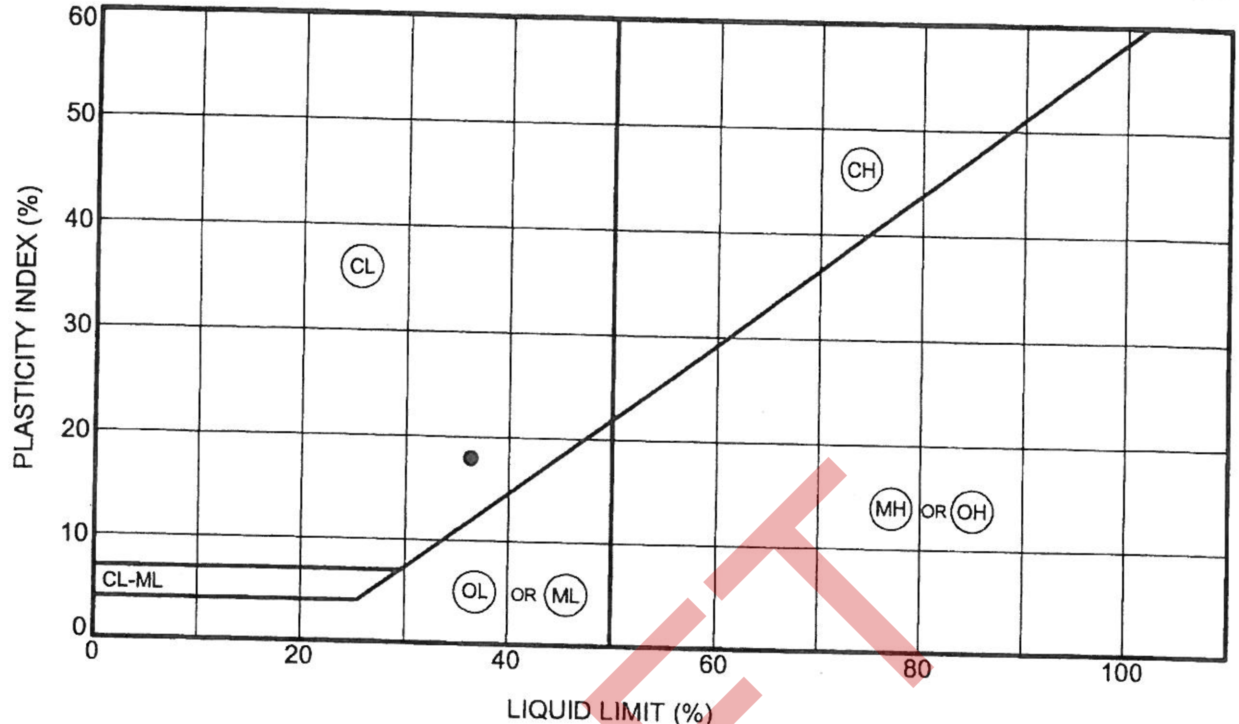
Moisture Content: The natural water content was determined (ASTM D2216) on 10 soil samples recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on the Plasticity Chart (Figure B-1) of this appendix and on the log of the boring at the appropriate sample depth.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on two samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

* * * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-1**	**1.5**	**15**	**36**	**18**	**18**		**LEAN CLAY (CL)**

LA CORP.GDT 10/21/04 MV* FLL

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PLASTICITY CHART AND DATA

Project: 2470 PULGAS AVENUE

Location: EAST PALO ALTO, CA

Project No.: 2047-1C

FIGURE B-1