10857 LINDA VISTA DRIVE
CUPERTINO, CALIFORNIA

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Austin Lin
SummerHill Homes
3000 Executive Parkway, Suite 450
San Ramon, CA 94583

PREPARED BY
ENGEO Incorporated

May 2, 2025

PROJECT NO.
25712.000.001

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Mr. Austin Lin
SummerHill Homes
3000 Executive Parkway, Suite 450
San Ramon, CA 94583

Subject: 10857 Linda Vista Drive
Cupertino, California

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

Dear Mr. Lin:

ENGEO prepared this design-level geotechnical report for the 10857 Linda Vista Drive project, as outlined in our agreement dated April 2, 2025. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical testing and observation services during construction. Please let us know when working drawings are nearing completion and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Lauren Loey, PE

Robert H. Boeche, CEG

lebl/jtr/rhb/cb

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for the design of the 10857 Linda Vista Drive project in Cupertino, California. As outlined in our agreement dated April 4, 2025, you authorized ENGEO to conduct the following scope of services.

- Subsurface field exploration
- Soil laboratory testing
- Data analysis and conclusions
- Report preparation

For our use, we received a preliminary grading plan (Sheet C4.0) and preliminary grading sections (Sheet C4.1) prepared by CBG for SummerHill Homes, dated February 3, 2025.

In 2024, we prepared a Preliminary Geotechnical Exploration Report (ENGEO, 2024a) and a Fault Exploration (ENGEO, 2024b) for the subject site.

This report was prepared for the exclusive use of SummerHill Homes, and its consultants, for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The approximately 2½-acre site is identified by the Assessor's Parcel Numbers (APNs) 356-06-001, 356-06-002, 356-06-003, and 356-06-004. The site is located at 10857-10887 Linda Vista Drive in Cupertino, California, as shown in the Vicinity Map (Figure 1). The site is bounded by the Cupertino Hills Swim and Racquet Club to the north, residential development to the south, Linda Vista Drive to the east, and the Deep Cliff Golf Course to the west. The western boundary of the site sits atop an approximately 2½:1 (horizontal:vertical) slope that is approximately 50 feet in height sloping down towards the Deep Cliff Golf Course.

The site currently consists of four single-family residential homes with five auxiliary structures. The Site Plan (Figure 2) shows the boundary of the project site and the approximate locations of our explorations.

1.3 PROPOSED DEVELOPMENT

Based on our conversation with you and our review of the project documents, we understand site improvements will include the following.

- Ten townhome structures with a total of 51 dwelling units up to three stories in height
- Underground utilities
- Sidewalks
- Landscaped areas, including bioretention areas

- Paved Streets
- Site walls and retaining walls

Preliminary grading plans show grading will be limited to minor cut and fill (up to 4 feet), with a retaining wall along the western boundary with a maximum retained height of 4 feet. We understand the proposed single-family structures will be at-grade and of conventional wood-frame construction. Structural loading information is not available at the time of this report writing. We assume building loads will be typical for this construction type.

2.0 FINDINGS

2.1 SITE HISTORY

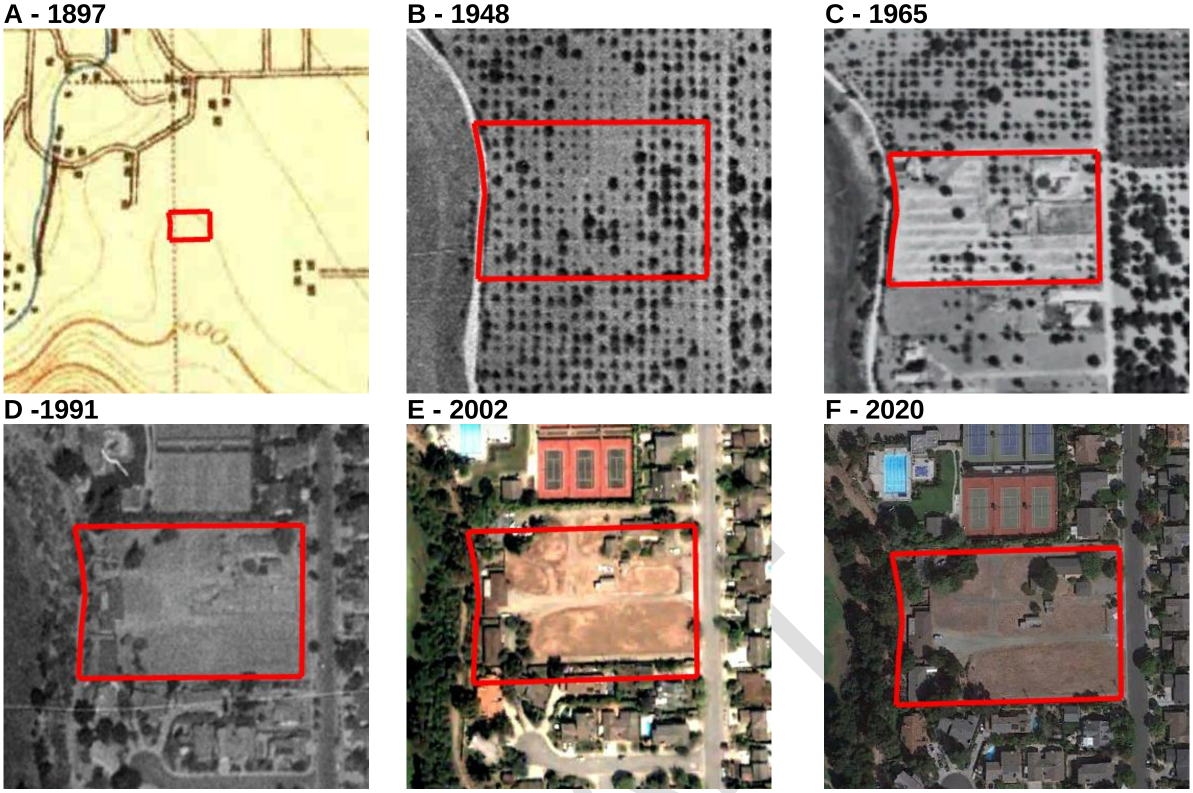
To understand site development history and geomorphology, we reviewed historical aerial photographs and topographic maps. We viewed numerous historical aerial photographs from 1948 through 2025 available on Google Earth, www.historicaerials.com, Environmental Data Resources (EDR), Frame Finder – UCSB Library, and NGMDB (USGS/AASG). We also viewed historical topographic maps dating back to 1897 to understand the site history before aerial photographic coverage was available. Topographic maps show the site's ground surface between approximately Elevation 400 feet and 375 feet (datum unknown), with no evidence of major topographic changes since early mapping. Our observations are summarized below in chronological order.

At least late 1890s to early 1980s: The topographic maps from 1897 and 1902 show no structures or roads mapped within the site (Exhibit 2.1-1, A - 1897). By the 1948 aerial photograph, structures were still not present within the site, and no roads were visible. The site and surrounding area appear to have been used for agricultural cultivation, specifically orchards (Exhibit 2.1-1, B - 1948). By 1953, two residential structures were built along the northern boundary of the site (10857 and 10867 Linda Vista Drive) (Exhibit 2.1-1, C - 1965). The southern portions of the site are still being utilized for orchards at this time, and other residential houses were developed to the south of the site.

1980s through mid-1990s: The structure at 10887 Linda Vista Drive was built by 1968 and 10877 Linda Vista Drive by 1982 and the orchard within the southern portions of the site was removed (Exhibit 2.1-1, D - 1991). The area surrounding the site continued to be developed with more residential structures (generally to the south, east, and north). Some trees are present within the site in landscaped areas.

Early 1990s to Present: The current site conditions generally remain unchanged from 1991 to present day.

EXHIBIT 2.1-1: Topographic Maps and Aerial Photographs



2.2 GEOLOGY AND SEISMICITY

2.2.1 Regional Geology

The site is located within the Coast Ranges geomorphic province of California. The Coast Ranges province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluvial valleys. Bedrock in the Coast Ranges consists of igneous, metamorphic, and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-known faults.

According to regional mapping by Brabb et al. (2000, Figure 3), the site is underlain primarily by Pleistocene alluvial fan and fluvial deposits and lies just northeast of the Monte Vista-Shannon Fault zone. Dibblee (2007) mapped the site as being primarily underlain by Quaternary aged older surficial sediments derived from hills to the west comprised of Santa Clara Formation and Franciscan Formation bedrock.

2.2.2 Seismicity

The San Francisco Bay Area contains numerous active faults. An active fault is defined by the California Geologic Survey as one that has had surface displacement within Holocene time (about the last 11,700 years) (CGS, 2018). Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. However, the site is located in a Santa Clara County Fault Rupture Hazard Zone (2012, Figure 6) associated with the Monte Vista-Shannon fault.

The site is within a seismically active region. To identify nearby active faults that can generate strong seismic ground shaking at the site, we utilized the USGS Earthquake Hazard Toolbox and the 2018 National Seismic Hazard Model (NSHM) to perform a disaggregation of the hazard at the peak ground acceleration (PGA) for a 2,475-year return period, with the resulting faults listed below in Table 2.2.2-1.

TABLE 2.2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site (Latitude: 37.31014 Longitude: -122.06033)

FAULT NAME	RUPTURE DISTANCE, R_{RUP}		MOMENT MAGNITUDE (M_W)
	KILOMETERS	MILES	
Monte Vista-Shannon (3)	1.8	1.1	7.2
Monte Vista-Shannon (4)	1.9	1.2	6.7
San Andreas (Peninsula) (2)	6.0	3.7	7.8
San Andreas (Peninsula) (1)	6.4	4.0	7.4
Monte Vista-Shannon (3)	6.7	4.2	6.6
San Andreas (Santa Cruz Mountains) (0)	15.9	9.9	7.2
Hayward (South) (1)	23.0	14.3	7.3

* Based on USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

These results represent known fault sources contributing at least 1 percent to the seismic hazard in our disaggregation. The rupture distances (R_{RUP}) and mean moment magnitudes (M_W) listed are based on values assigned according to the 2018 NSHM, and the numbers in parentheses after the fault names correspond to fault subsections assigned by the NSHM. Note that the above fault table is not an exhaustive list and other faults in the region may generate seismic shaking at the project site.

The Uniform California Earthquake Rupture Forecast (UCERF3) (Field et al, 2015) estimates the 30-year probability for a magnitude 6.7 or greater earthquake in Northern California at approximately 95 percent, considering the known active seismic sources in the region.

2.3 PREVIOUS FIELD EXPLORATIONS

2.3.1 ENGEO. 2024a. Preliminary Geotechnical Exploration.

As mentioned in Section 1.1, we previously prepared a preliminary geotechnical exploration at the subject site for the current project. We retained the services of a licensed C-57 contractor operating a truck-mounted cone penetration test (CPT) rig to advance CPTs at the four locations shown in Figure 2. The CPTs were performed in general accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the friction sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). The CPTs were advanced to practical refusal ranging between approximate depths of 9 feet to 58½ feet below ground surface (bgs). Pore pressure dissipation tests were performed at three CPT locations; none encountered groundwater.

The contractor performed shear-wave velocity (V_S) measurements in 1-CPT1 using procedures in general accordance with ASTM D5778 and ASTM D7400. We present the CPT logs and the V_S profile in Appendix B.

We incorporated the data from this previous exploration into our analysis and findings of this report.

2.3.2 ENGEO. 2024b. Fault Exploration.

We previously prepared a fault exploration report for the subject property which involved excavating three fault trenches in the southeastern portion of the site to evaluate the potential for surface fault rupture at the site and within the proposed development areas. The trenches were excavated along a roughly east-northeast trend to be perpendicular to the mapped trace of the Monte Vista-Shannon Fault, as shown in Figure 2. The trenches extended to a maximum depth of 10 feet bgs and constituted roughly 280 linear feet of excavation. We encountered older alluvial deposits consistent with the geologic maps and previous studies.

We found no evidence of a surface fault rupture within our trenches across the project site and our study found no evidence to suggest that the Monte Vista-Shannon Fault is present at the site.

We present the trench logs in Appendix B. We incorporated the data from this previous exploration into our analysis and findings of this report.

2.4 CURRENT FIELD EXPLORATION

Our current field exploration included drilling three borings at various locations on the site. We performed our field exploration on April 3, 2025. The approximate locations of our borings are shown in Figure 2.

We estimated the locations of our explorations using consumer-grade global positioning system (GPS) and their proximity to existing site features; therefore, the locations shown should be considered accurate only to the degree implied by the method used. We permitted our explorations with Valley Water (formerly Santa Clara Valley Water District).

We retained the services of a drilling subcontractor who provided a crew operating a truck-mounted drill rig to advance the borings using 4-inch-diameter solid-flight-auger and mud rotary drilling methods. We advanced the borings to depths ranging from approximately 30 to 48 feet bgs. A representative of our firm observed the drilling and logged the subsurface conditions at each location.

We obtained soil samples at various depth intervals using either standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D.) split-spoon sampler or California Modified samplers with 2½-inch inside diameter (I.D.) fitted with sampling liners. We advanced the driven samplers with an automatic trip, 140-pound hammer with a 30-inch free fall and recorded the penetration of the sampler in the field as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring log shows the number of blows required for the last 1 foot of penetration, or the number of blows per depth of penetration for samples that met driving refusal. We did not adjust the blow counts shown in the boring logs using any energy correction or sample size factors.

The boring logs are presented in Appendix A. The logs depict interpreted subsurface conditions within the borings at the time the exploration was conducted. The stratification lines in our logs represent the approximate boundaries between soil types and the actual material transitions may be more gradual. Subsurface conditions at other locations may differ from the conditions noted at these boring locations.

2.5 SURFACE CONDITIONS

There was no appreciable difference at the project site between our current exploration in April 2025 and our previous exploration in August 2024. The surface conditions at the site generally consisted of residential structures and vegetated areas. Site grades range from Elevation 391 in the northeast to Elevation 397 in the southwest (NAVD88). The site is generally flat with a gentle slope from west to east. The western boundary of the site sits atop an approximately 2½:1 (horizontal:vertical) slope that is approximately 50 feet in height. Approximately 8 feet below the top of the slope is a bench that makes up the existing Stevens Creek Trail. The bench is approximately 8½ feet wide and ranges in elevation from Elevation 388 feet in the south and Elevation 389½ feet in the north as the trail parallels the project site boundary.

Please refer to the Site Plan, Figure 2, for more information on site features.

2.6 SUBSURFACE CONDITIONS

Based on our interpretation of the CPT data, fault trenches, and borings, the subsurface conditions at the site generally consist of loose to medium dense clayey sand and stiff to very stiff lean clay in the upper 3 to 7 feet of the site, underlain by medium dense to very dense poorly graded sand with clay, clayey gravel, and clayey sand with interbedded lenses of very stiff to hard fine-grained material and dense to very dense silty sand to the maximum depth explored of 58½ feet bgs. As noted in the fault evaluation, deposits encountered were consistent with Pleistocene age deposits.

The results of our plasticity index (PI) testing for materials near the ground surface between 1 and 2½ feet yielded PI results ranging from 10 to 11, which indicates low expansion potential.

2.7 GROUNDWATER CONDITIONS

Based on equilibrium pore pressure not being achieved in our CPT pore pressure dissipation tests, we conclude that we did not observe static groundwater in our explorations, which extended to a maximum depth of approximately 58 feet. The Seismic Hazard Zone Report for the Cupertino quadrangle reports a historical high groundwater depth of greater than approximately 50 feet below the ground surface for the site (CGS, 2002).

Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, and changes in irrigation and drainage patterns.

2.8 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project, we performed moisture content, plasticity index, gradation, hydrometer, and soil corrosion potential testing. Moisture content, fines content, and plasticity index results are recorded in the boring logs in Appendix A; other laboratory data is included in Appendix B.

2.9 SLOPE STABILITY ANALYSES

2.9.1 Method of Analysis

Geotechnical evaluation of slope stability involves assessment of the input parameters along with the selected method of analysis. While there are many different methods of stability analysis and numerous available computer programs, we selected the program SLIDE2, by Rocscience. SLIDE2 is a two-dimensional limit equilibrium program that is widely used in slope stability evaluations. We used Spencer's Method, which is an iterative solution that satisfies both force and moment equilibrium and assumes all slice side forces have the same inclination. This method is appropriate for both circular and non-circular failure surfaces; for this evaluation, we considered only circular failure surfaces.

In evaluating the stability of slopes under seismic conditions, we used a "pseudostatic" method of analysis. This has been a common method of evaluating the stability of engineered slopes during earthquakes. The pseudostatic method models the effects of transient or pulsating earthquake loading on a potential slide mass by using an equivalent sustained horizontal force that is the product of a seismic coefficient and the weight of the potential slide mass. The slope is first analyzed to establish the minimum factor of safety under static conditions. Once this critical failure surface is located, an additional horizontal force acting in the direction of potential failure is imposed on the sliding mass. This additional force is equal to the soil mass multiplied by a seismic coefficient of horizontal acceleration.

For most pseudostatic stability analyses in the United States, a design seismic coefficient on the order of 0.05 g to 0.15 g has typically been used, where "g" is the ground acceleration expressed in terms of the equivalent acceleration due to Earth's gravity. Using the procedure outlined in Special Publication 117A (California Geological Survey, Guidelines for Evaluating and Mitigation Seismic Hazards in California, 2008) and based on the maximum considered earthquake geometric mean peak ground acceleration (PGA_m) from the 2022 California Building Code (CBC), a moment magnitude of 7.9 based on an earthquake on the San Andreas Fault, we obtained and used a seismic coefficient (K_h) of 0.34 g for our analysis using the 15-centimeter threshold.

2.9.2 Acceptable Factors of Safety

Based on local geotechnical practice, in our opinion, we recommend that a static factor of safety of 1.5 and a pseudostatic factor of safety of 1.0 be considered adequate for the proposed slopes. We considered the various levels of conservatism involved in determining the engineering properties of the soil (density, shear strength, unit weight, etc.), the assumptions made in the method of analysis, and potential variations in field conditions.

2.9.3 Geometry and Idealized Soil Profiles

We reviewed the proposed preliminary grading plans to identify critical cross-sections to analyze for slope stability. We selected one cross section (A-A') at the steepest grade of the adjacent slope. The location of the analyzed Cross Section A-A' is shown in the Site Plan, Figure 2.

We developed an idealized profile representative of the anticipated subsurface conditions and proposed slope configurations at the cross-section location. For modeling the planned improvements, we referenced the proposed development plan for retaining wall location and building footprints. We applied surcharge loads at the proposed building footing print and roadways based on the development plan. Cross Section A-A' is shown in Appendix D.

To develop our shear strength parameters, we used the subsurface information obtained from our field exploration, laboratory test results, and engineering judgement. We also referred to shear strength parameters for geologic units presented in the Seismic Hazard Zone report for the Cupertino quadrangle (CGS, 2002). Based on our review of the data and our experience, we selected the following parameters to represent lower-bound shear strengths for the native geologic units used in our slope stability models.

TABLE 2.9.3-1: Strength Parameters for Slope Stability Analysis

MATERIAL	UNIT WEIGHT (pcf)	COHESION (psf)	PHI (degrees)
Surficial deposits – Lean Clay and Clayey Sand	120	150	28
Pleistocene Deposits – Poorly Graded Sand with Low Fines Content	120	100	35
Pleistocene Deposits – Clayey Gravel and Clayey Sand	120	200	38
Pleistocene Deposits – Silt and Lean Clay	120	200	28

2.9.4 Results of Analyses

We performed slope stability analyses for both static and pseudostatic conditions. The results of our analyses are summarized in the table below and are included in Appendix D.

TABLE 2.9.4-1: Summary of Slope Stability Analyses

FACTOR OF SAFETY		
LOCATION	STATIC	PSEUDOSTATIC (0.31g)
Cross-Section A-A’	1.5	1.0

2.10 LIQUEFACTION ANALYSIS

According to the Seismic Hazard Zones Map for the Cupertino Quadrangle (CGS, 2002) the site is not located within a liquefaction hazard zone. However, the CGS (2002) and Santa Clara County (SCC, 2012) Santa Clara County Hazard Zones (SCC, 2012) (Figure 6), identify the property immediately west of the project site as an area that lies within a liquefaction hazard zone, as shown in Figure 5. Note that the Seismic Hazard Zone Maps provide potential liquefaction risks based on general geologic conditions.

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, and uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, it is said to have liquefied. If the sand consolidates or vents to the surface during and following liquefaction, ground settlement, and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also undergo “cyclic softening” or strength loss as a result of cyclic loading.

We encountered medium dense to very dense sand in our CPTs and borings. Laboratory testing on samples collected from the borings revealed appreciable fines content in the sand. We did not encounter groundwater in our subsurface explorations, and the historic depth to groundwater at the site is over 50 feet below ground surface (CGS 2002). Therefore, we conclude the risk of liquefaction at the site is low.

2.11 DYNAMIC DENSIFICATION SETTLEMENT

Dynamic densification settlement of loose granular soil above the groundwater table, also called dry-sand settlement, can cause settlement of the ground surface due to earthquake-induced vibrations. Dynamic densification is generally a higher risk for sand or gravel with fines contents less than 15 percent. Our previous CPT-based analysis of potential dynamic densification settlement for sand layers above the groundwater table indicated up to 1 inch of settlement at the site, with most densification settlement occurring within a fine-grained to silty sand lens at depths of approximately 24 to 28 feet below ground surface. However, our boring explorations and laboratory testing revealed the subsurface conditions at these depths at the site have higher fines contents than previously shown in the CPTs. We evaluated the dynamic-densification potential of sandy layers with fines content less than 15 percent using the procedures outlined in Tokimatsu and Seed (1987) and Robertson and Shao (2010). Based on our analysis, we estimate up to ½ inch of dry-sand densification is possible under the maximum considered earthquake (MCE_G) seismic event.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications, and finalized with additional explorations.

The primary geotechnical concerns that could affect development on the site are existing fill, expansive soil, and seismically induced settlement (i.e., dry sand settlement). We summarize our conclusions below.

3.1 EXISTING FILL

Our borings encountered approximately 1½ feet of fill across the site. This depth of fill is consistent with typical depth of disturbance associated with orchards, which were present at the site. Non-engineered fill is also possible to exist around existing structures and in-place improvements, such as abandoned septic systems.

Existing, non-engineered fill may undergo excessive settlement, especially under new fill or building loads. Without proper documentation of the existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We anticipate manmade debris within the existing fill can be selectively removed from the material before reuse as engineered fill. We present fill removal recommendations in Section 5.2.

3.2 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and liquefaction. Based on site observations, topographic and lithologic data, subsurface data, and regional geology, the risk of regional subsidence or uplift, landslides, tsunamis, or seiches are considered low to negligible at the site.

The following sections present a discussion of select seismic hazards as they apply to the site.

3.2.1 Ground Rupture

The site is located within a Santa Clara County Fault Rupture Hazard Zone, as shown in Figure 6. According to USGS (2022), the Monte Vista-Shannon fault zone is located less than ¼ mile east and west of the project site. We performed a fault trench study (ENGEO, 2024b) to evaluate possible ground rupture. Based on our findings, the risk of ground rupture within the site is low.

3.2.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the region could cause considerable ground shaking at the site, similar to that which has occurred in past major events. To mitigate the effects of ground shaking, all structures should be designed using sound engineering judgment and in accordance with the latest edition of the California Building Code (CBC), as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some non-structural damage, and (3) resist major earthquakes without collapse, but with some structural, as well as non-structural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake. Section 3.4 presents the 2022 California Building Code design parameters.

3.2.3 Liquefaction

As summarized in Section 2.10, the site does not lie within a liquefaction hazard zone, though the area immediate to the west of our site does.

Considering the granular deposits encountered in our explorations were medium dense to very dense sand and we did not encounter groundwater in our subsurface explorations, the potential for liquefaction at the site is low.

3.2.4 Dynamic Densification

As summarized in Section 2.11, medium dense to very dense granular material was encountered above the design groundwater level. Based on our analyses described in Section 2.11, we estimate potential dynamic densification settlements of up to ½ inch. This densification settlement should be considered in the foundation design provided in Section 6.0.

3.2.5 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move towards a free face or down a gentle slope. Generally, the effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope.

The western boundary of the site sits atop an approximately 2½:1 (horizontal:vertical) slope that is approximately 50 feet in height. However, the risk of liquefaction occurring and acting as the mechanism for lateral spread is low. Additionally, our pseudostatic slope stability analysis presented in Section 2.9 meets recommended factors of safety considering approximately 15 cm of deflection. Based on these factors, we conclude the risk of lateral spreading at the project site is low.

3.3 2022 CBC SEISMIC DESIGN

The 2022 CBC utilizes seismic design criteria established in the ASCE/SEI Standard "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16). Based on the subsurface conditions encountered and shear-wave velocity measurements at 1-CPT1, we characterized the site as Site Class C. In Table 3.3-1 below, we provide the CBC seismic parameters based on the United States Geological Survey's (USGS') Seismic Design Maps for your use.

TABLE 3.3-1: 2022 CBC Seismic Design Parameters, Latitude: 37.31014 Longitude: -122.06033

PARAMETER	VALUE
Site Class	C
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.31
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.83
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.77
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.16
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.85
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.78
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.96
Site Coefficient, F_{PGA}	1.2
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	1.15
Long-period transition-period, T_L	12 sec

3.4 SOIL CORROSION POTENTIAL

As part of this study, we obtained representative soil samples and submitted to Cerco Analytical, a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride. The results are included in Appendix B and summarized in the table below.

TABLE 3.4-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (feet)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)	REDOX (mV)
2-B1	1	7.28	4,800	< 15	< 15	240
2-B2	1.5	7.97	5,900	< 15	<15	240

The 2022 CBC references the 2019 American Concrete Institute Manual, ACI 318-19, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

In accordance with the criteria presented in ACI Table 19.3.1.1, this soil is categorized as within the F0 freeze-thaw class, S0 sulfate exposure class, W0 moisture/water exposure class, and C0 corrosion class.

Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 pounds per square inch (psi) is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

As part of their analyses, Cerco Analytical provided brief commentary based on the test results. The full results are presented in Appendix B. We provide a summary of the commentary below.

- Based on the resistivity measurements, the soil from both samples is considered "moderately corrosive" to buried metal piping. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping, such as ductile iron firewater, should be protected against corrosion.
- The reported values of chloride, sulfate, and pH concentrations do not pose a significant impact to metal or concrete.
- The reported value for redox potential is indicative of "slightly corrosive" soil resulting from anaerobic soil conditions.

The recommendations provided are for general reference. If it is desired to investigate this further, we recommend a corrosion consultant be retained to evaluate whether specific corrosion recommendations are advised for the project.

3.5 STATIC SETTLEMENT

Soil may settle in response to new loads induced by new fill, structures, or equipment. This settlement, if it occurs, may occur as immediate (elastic) or long-term consolidation settlement. The site is primarily underlain by medium dense to very dense sandy soil. We anticipate building loads will apply to an average bearing pressure of up to 1,000 pounds per square foot (psf). Considering the subsurface conditions and that site grading is performed in accordance with the recommendations in Section 5, we estimate less than ½ inch of elastic settlement due to static loading, with settlement occurring almost immediately upon construction of the building foundations.

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.
2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fill have been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define "structural areas" as any area sensitive to settlement of compacted soil. These areas include, but are not limited to, building pads, sidewalks, pavement areas, and retaining walls.

5.1 DEMOLITION AND SITE PREPARATION

Site development should commence with the demolition and removal of surface and subsurface improvements that are planned for removal. Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing buildings and their associated foundations, slabs, buried utility and irrigation lines, pavements, debris, shrubs, and associated roots. Any existing features extending below the planned finished site grades should be demolished and backfilled with suitable material compacted to the recommendations presented in Section 5.5.

Existing vegetation and their roots should be removed from areas to receive fill or improvements. Tree roots should be removed down to a depth of at least 3 feet below existing grade. Large vegetation and debris should be separately stockpiled from soil fill. Subject to approval by the landscape architect, stripping's and organically contaminated soil can also be used in landscape areas. Organically contaminated soil that will be retained for future use in landscape areas should be stockpiled in areas where they will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface, as determined by our representative.

5.2 EXISTING FILL REMOVAL

As discussed in previous sections, existing disturbed soil is present at the site from previous site development. At this time, we consider this fill to be undocumented, non-engineered fill.

We recommend existing fill material and any other loose/soft material beneath the proposed improvements be removed to expose native soil conditions and replaced as engineered fill under the observation of our representative. Removal of potential existing fill surrounding below-grade elements, such as existing footings and underground utilities, can likely be achieved during excavation for demolition and removal of the below-grade structures. Our representative should confirm that excavation bottoms within existing fill expose material that is stiff and free of deleterious matter. The base of the subexcavation should be processed, moisture conditioned, and compacted as engineered fill.

5.3 SOIL MOISTURE CONDITIONS

The contractor may encounter excessively over-optimum (wet) soil moisture conditions during winter or spring grading, during or following periods of rain. Wet soil can make proper compaction difficult or impossible.

Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather,
2. Mixing with drier materials,
3. Mixing with a cement product, or
4. Stabilizing with aggregate or geotextile stabilization fabric, or both.

We should be allowed to evaluate options 3 and 4 and approve the approach prior to implementation.

5.4 ACCEPTABLE FILL

On-site soil is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and rock particles greater than 6 inches in maximum dimension. Oversized particles should be either removed from the fill or broken down to meet the requirement. Imported fill material should meet the above requirements and have a PI less than 12 with approximately 20 percent passing the No. 200 sieve. We should be contacted to sample and test the proposed imported fill material at least 10 days prior to delivery to the site. Environmental sampling and testing of potential import soil sources should also be submitted to us for review.

5.5 FILL PLACEMENT

5.5.1 Structural Areas

After removal and recompaction of non-engineered fill, the exposed non-yielding surface of areas to receive fill or to be left at grade should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. The following compaction control requirements should be applied to all fill, including backfill, except for landscape areas.

TABLE 5.5.1-1: Compaction Control Requirements

FILL LOCATION	REQUIRED RELATIVE COMPACTION* (%)	MINIMUM MOISTURE CONTENT (percentage points above optimum)
General Fill	90	2
Pavement and Flatwork Aggregate Base	95	0

5.5.2 Landscape Areas

The contractor should compact the finished subgrade in landscape areas in accordance with Section 5.5.1, with a minimum relative compaction of 85 percent and at a moisture content of at least 2 percentage points above optimum.

5.6 SITE DRAINAGE

5.6.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where development conditions restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Do not allow water to pond near foundations, pavements, exterior flatwork, or site walls.

5.6.2 Stormwater Bioretention Areas

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by baserock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

6.0 FOUNDATION RECOMMENDATIONS – RESIDENTIAL STRUCTURES

We developed structural improvement recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis.

Based on our slope stability analysis in Section 2.9, we recommend a 25-foot setback from the top of slope for any residential dwelling units. This setback is visually presented at its approximate location in Figure 2.

6.1 POST-TENSIONED MAT FOUNDATIONS

We recommend that the proposed residential structures be supported on post-tensioned (PT) mat foundations bearing on prepared native soil or engineered fill.

The structural engineer should determine the actual PT mat thickness using the geotechnical recommendations in this report; we defer to the professional judgment of the structural engineer on the necessary mat thickness. We recommend that PT mats have a thickened edge at least 2 inches greater than the mat thickness and that the thickened edge be at least 12 inches wide. ENGEO should be retained to review the PT mat foundation design.

PT mats may be designed for an average allowable bearing pressure of up to 1,000 psf for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for wind or seismic loads. Design PT mats using the criteria presented in Table 6.1-1 below.

TABLE 6.1-1: Post-Tensioned Mat Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	9.0	5.0
Differential Soil Movement, y_m (inches)	0.5	0.7

The above values are based on the procedure presented by the Post-Tensioning Institute "Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive and Stable Soils" (PTI DC10.5-19).

Underlay PT mats with a moisture reduction system as recommended below.

6.1.1 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, such as post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."
2. Concrete shall have a concrete water-cement ratio of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

7.0 FOUNDATION RECOMMENDATIONS – ANCILLARY STRUCTURES

We developed structural improvement recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis.

In general, ancillary structures can be supported on conventional footings or helical piles. The following recommendations should be incorporated into the foundation design.

7.1 CONVENTIONAL FOOTINGS

7.1.1 Footing Dimensions and Allowable Bearing Capacity

Provide minimum footing dimensions as follows in Table 7.1.1-1 below.

TABLE 7.1.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (inches)	MINIMUM WIDTH (inches)
Continuous	18	18
Isolated	18	24

*Below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. Any cold joints between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade.

Design foundations recommended above for a maximum allowable bearing pressure of 1,500 psf for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

7.1.2 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design.

- Passive Lateral Pressure: 300 pcf
- Coefficient of Friction: 0.35

The values above are unfactored and an appropriate factor of safety should be included, as necessary, based on the method of analysis and design combination. We recommend half of the passive resistance to be used if both passive pressure and friction are used in combination. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or a concrete slab. Additionally, passive pressure should be ignored where lateral confinement is less than 10 feet to daylight. Based on the proposed wall layout and existing grades of the slope, passive resistance should be ignored at elevations above 389 feet (NAVD88). Foundations may require deepening to meet these conditions.

7.2 HELICAL PILES

It is our understanding from you that the retaining walls may be supported utilizing helical piles. However, vertical helical piles will not provide significant lateral load resistance so grade beams or battered piles may be necessary.

Helical piles should be designed according to the soil parameters listed below.

Table 7.2-1: Helical Pier Design Soil Parameters

SOIL MATERIALS	UNIT WEIGHT (pcf)	FRICTION ANGLE (degrees)	COHESION (psf)
Foundation Soils	125	26	0

A minimum length should be determined to ensure helical piles achieve adequate lateral confinement with the adjacent slope to the west. Lateral and axial capacity of the piles should not be considered until 10 feet of lateral confinement from the adjacent slope is achieved. Based on the proposed wall layout and existing grades of the slope, pile capacity should be ignored at elevations above 389 feet (NAVD88).

To achieve lateral resistance within a helical pile system, we recommend that the helical piles are connected to a pile cap. The pile cap may provide an unfactored passive resistance of 300 pcf (as discussed in Section 7.1.2). If additional lateral resistance is necessary, the helical piles may be battered. Lateral resistance from battered piles should be ignored within the active soil pressure zone of the retained soil, which can be considered as an imaginary line starting 5 feet inward from the base of the back of the wall and extending upward into the retained soil at an angle of ($45 + \phi/2$).

7.2.1 Construction Considerations

A contractor experienced in the construction of helical piles should perform the construction. The successful performance of helical wall systems is dependent on proper installation methods. The geotechnical engineer should perform full-time monitoring of helical pile installation and testing.

Localized areas of dense soil should be anticipated during installation, which may make installation difficult or infeasible. Therefore, it may be necessary to predrill helical piles prior to installation. Helical piles must achieve the design embedment length and torque requirements regardless of drilling conditions.

An appropriate testing program should be developed for our approval. The load-testing program should consider alternative soil conditions and methods, such as predrilling, employed during construction.

8.0 SLABS-ON-GRADE

8.1 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 4 inches of concrete over 4 inches of aggregate base. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557). Thicken flatwork edges to at least 8 inches to help control moisture variations in the subgrade and place wire mesh or rebar within the middle third of the slab to help control the width and offset of cracks. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

8.2 TRENCH BACKFILL

Backfill and compact all trenches below building slabs-on-grade and to 5 feet laterally beyond any edge in accordance with Section 5.5.

9.0 RETAINING WALLS

9.1 LATERAL SOIL PRESSURES

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 60 pcf. In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 40 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

9.2 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, non-woven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.
2. Extend rock drains from the wall base to within 12 inches of the top of the wall.
3. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

9.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 5.5. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

9.4 FOUNDATIONS

Site walls may be supported on continuous footings designed in accordance with recommendations presented in Section 7.1, except the minimum embedment depth should be increased to 24 inches below lowest adjacent soil grade, or helical piers design in accordance with recommendations presented in Section 7.2. Walls along the western site boundary at the top of the slope may be supported on helical piers only.

10.0 PAVEMENT DESIGN

10.1 PRELIMINARY FLEXIBLE PAVEMENTS

Because of the variable near-surface soil, it is our opinion that a resistance value (R-value) of 5 is applicable for preliminary design. The R-value of the subgrade should be confirmed during site grading. Using estimated traffic indexes (TI) for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 10.1-1: Recommended Hot Mix Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)
5	3	10
6	3½	13
7	4	16
8	5	18

The civil engineer should determine the appropriate traffic indexes based on the estimated traffic loads and frequencies. The minimum pavement section(s) should be confirmed by the civil engineer and local jurisdiction (such as City or County).

Pavement materials and construction should comply with the specifications and requirements of the Standard Specifications by Caltrans, the local jurisdiction, and our compaction recommendations in Section 9.3.

10.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. We recommend the following minimum design sections for rigid pavements.

- Use a minimum section of 6 inches of Portland Cement concrete over 4 inches of Caltrans Class 2 Aggregate Base
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi
- Provide minimum control joint spacing in accordance with Portland Cement Association Guidelines

Final design of rigid pavement sections and accompanying reinforcement should be performed based on estimated traffic loads and frequencies.

10.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 5.5. Aggregate base should meet the requirements for ¾-inch maximum Class 2 Aggregate Base in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

10.4 CUTOFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 10857 Linda Vista Drive Project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strive to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies, or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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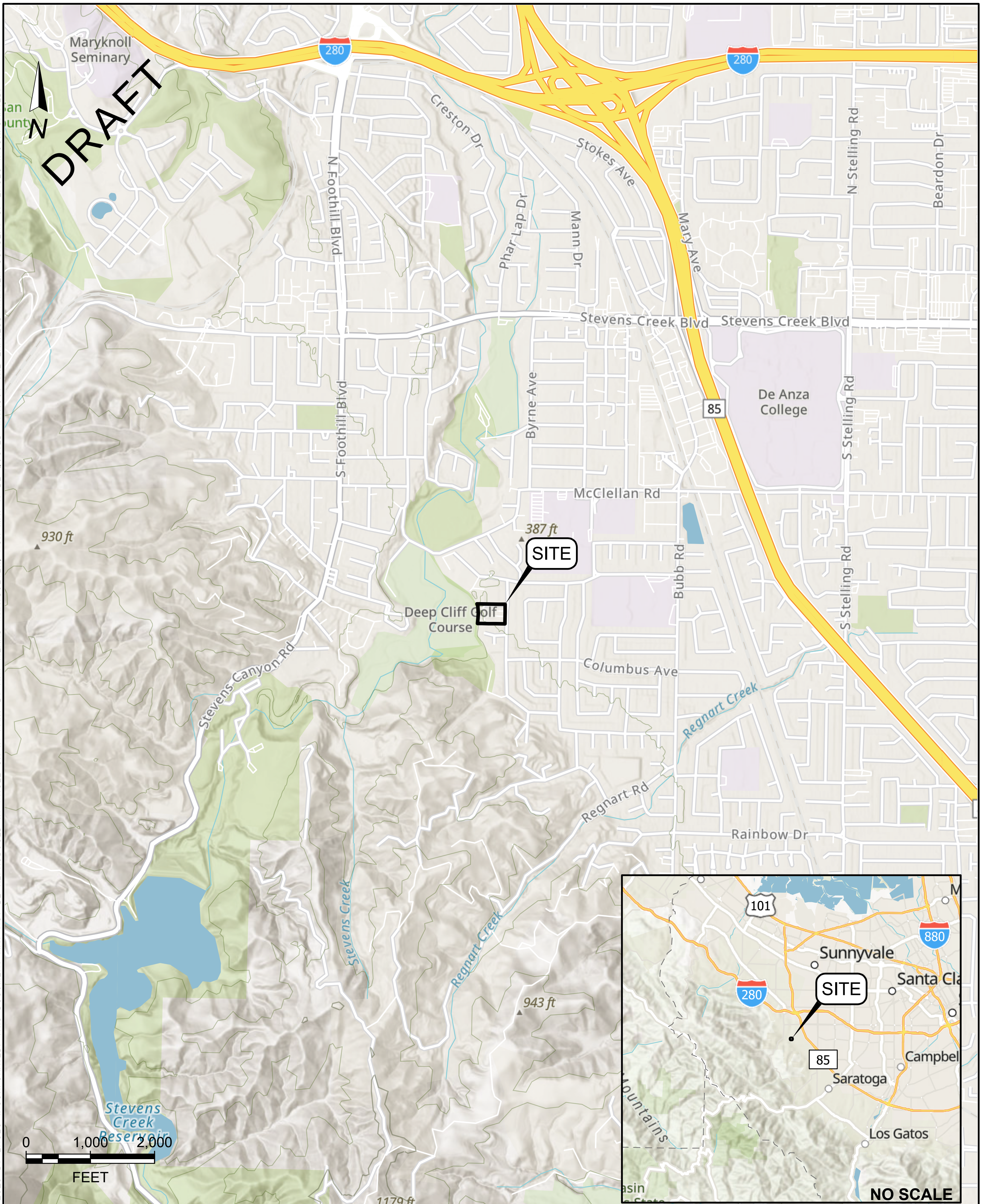
FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Plan
FIGURE 3: Regional Geologic Map
FIGURE 4: Regional Faulting and Seismicity Map
FIGURE 5: Seismic Hazards Zone Report
FIGURE 6: Santa Clara County Geologic Hazards Zone
FIGURE 7: City of Cupertino Geologic Hazards Zone



BASEMAP SOURCE: ENGEO INC, ESRI WORLD TOPO MAP

ENGEO Expect Excellence	VICINITY MAP 10857 LINDA VISTA DRIVE CUPERTINO, CALIFORNIA	PROJECT NO. : 25712.000.001 SCALE: AS SHOWN DRAWN BY: NWC / CHECKED BY: JTR	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

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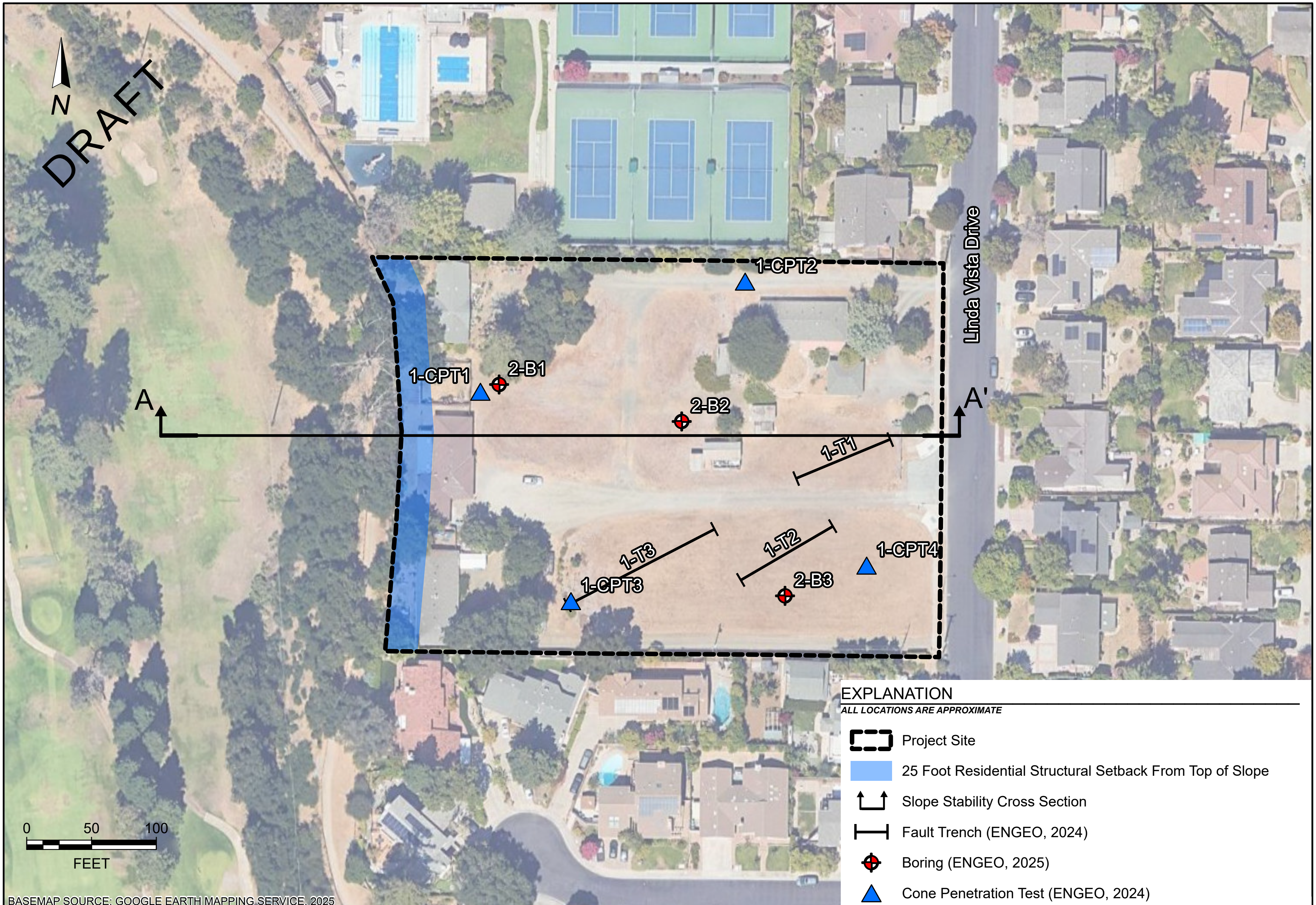
LAYOUT: 01. VICINITY 8.5 X 11 PORTRAIT USER: NCLOUGH



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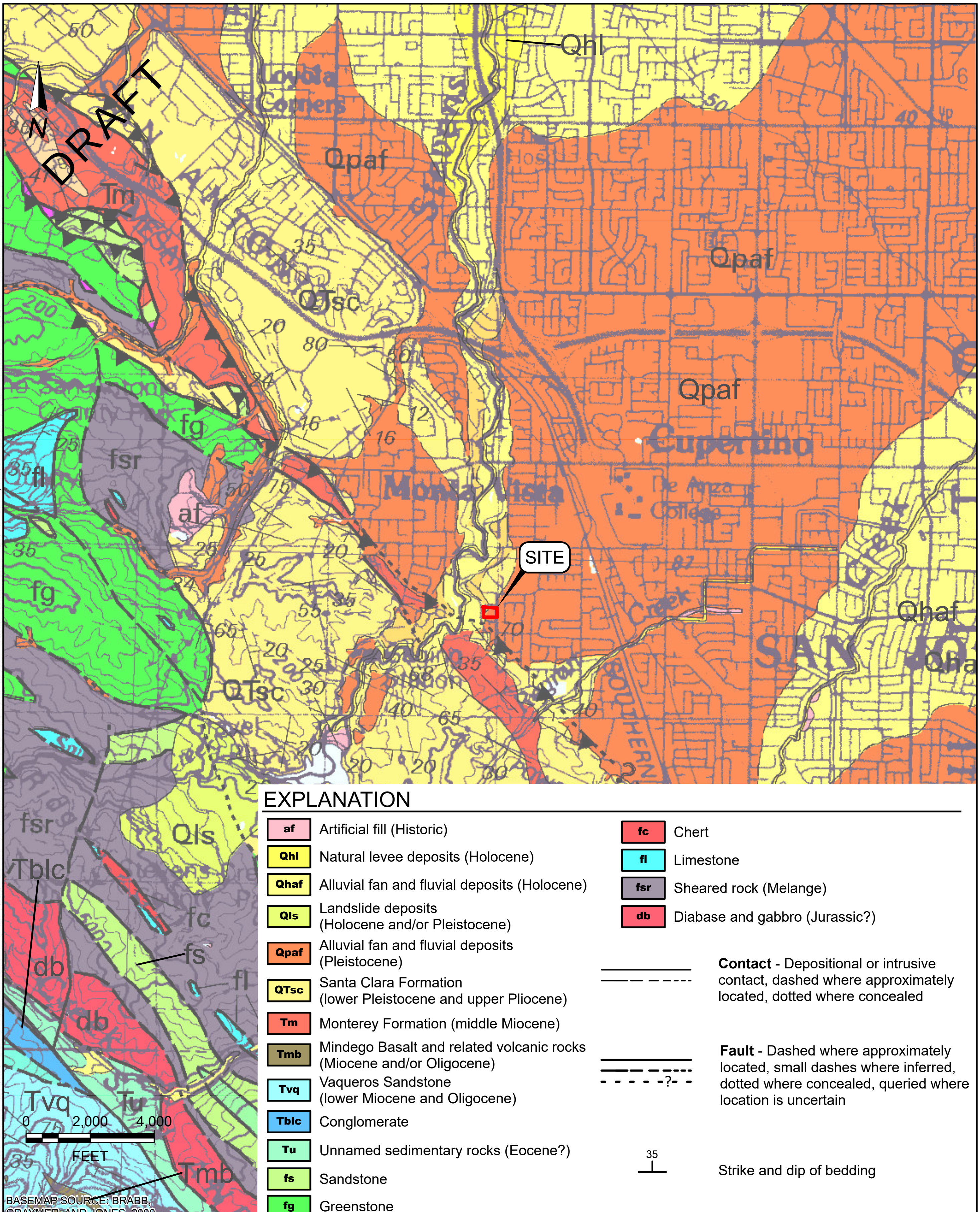
SITE PLAN
10857 LINDA VISTA DRIVE
CUPERTINO, CALIFORNIA

PROJECT NO. : 25712.000.001	FIGURE NO. 2
SCALE: AS SHOWN	
DRAWN BY: NWC	CHECKED BY: JTR

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25712\25712000001\GEOTECH\GEX\10857 LINDA VISTA DRIVE.APRX
LAYOUT: 02. SITE PLAN 8.5 X11 LANDSCAPE USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR



ENGEO
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GEOLOGIC MAP
10857 LINDA VISTA DRIVE
CUPERTINO, CALIFORNIA

PROJECT NO.: 25712.000.001	
SCALE: AS SHOWN	
DRAWN BY: NWC	CHECKED BY: JTR

FIGURE NO. 3

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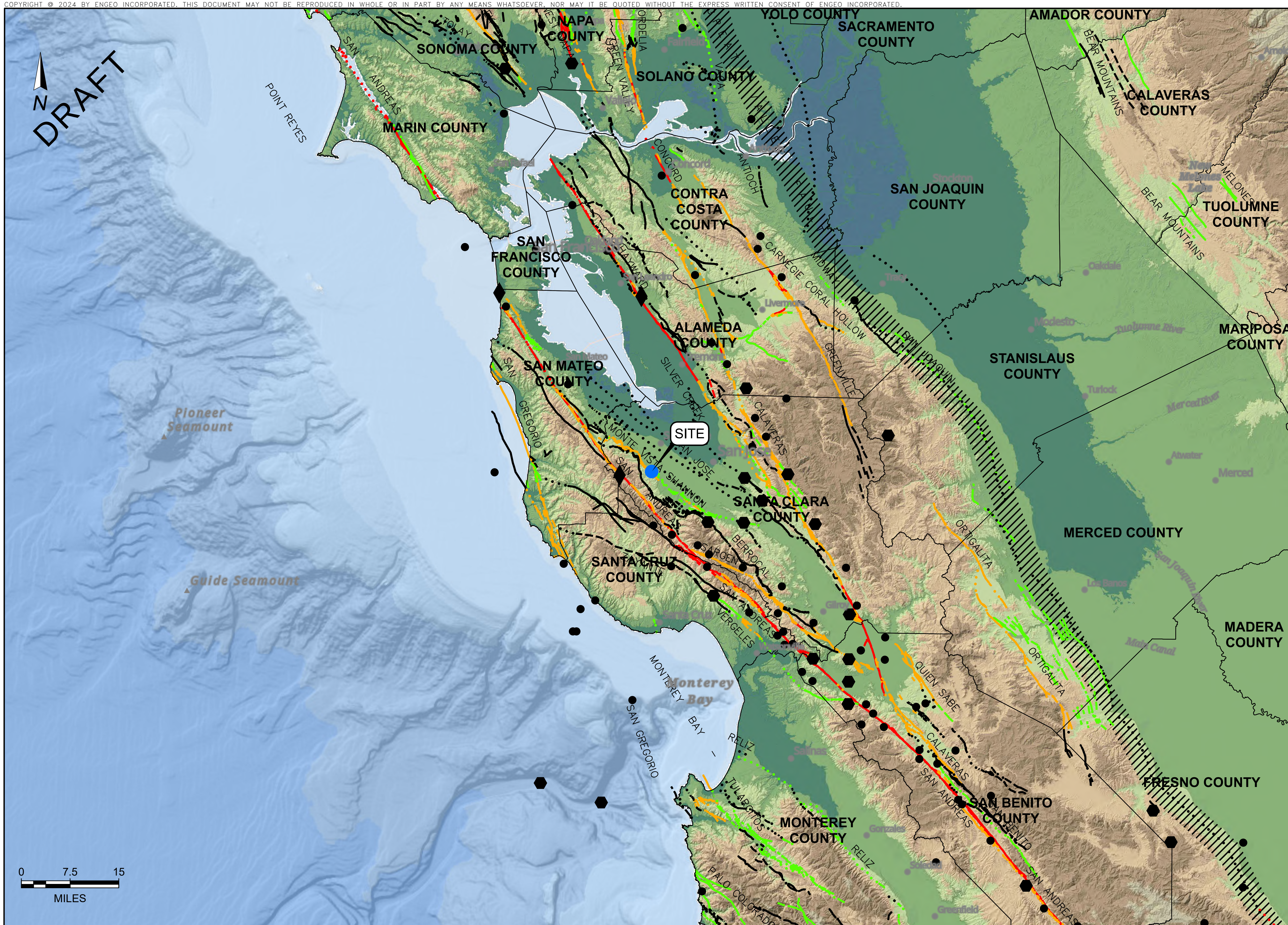
LAYOUT: 03. GEOLOGIC 8.5 X 11 PORTRAIT USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- Project Site

HISTORIC EARTHQUAKE EPICENTERS

- Magnitude 5-6
- Magnitude 6-7
- Magnitude 7+
- Historic Blind Thrust Fault Zone

QUATERNARY FAULTS 2020

Based on time of most recent surface deformation

- Historical (<150 Years), Well Constrained Location
- Historical (<150 Years), Moderately Constrained Location
- Historical (<150 Years), Inferred Location
- Latest Quaternary (<15,000 Years), Well Constrained Location
- Latest Quaternary (<15,000 Years), Moderately Constrained Location
- Latest Quaternary (<15,000 Years), Inferred Location
- Latest Quaternary (<15,000 Years), Inferred Location
- Late Quaternary (<130,000 Years), Moderately Constrained Location
- Late Quaternary (<130,000 Years), Inferred Location
- Middle And Late Quaternary (<750,000 Years), Well Constrained Location
- Middle And Late Quaternary (<750,000 Years), Moderately Constrained Location
- Middle And Late Quaternary (<750,000 Years), Inferred Location
- Undifferentiated Quaternary(<1.6 Million Years), Well Constrained Location
- Undifferentiated Quaternary(<1.6 Million Years), Moderately Constrained Location
- Undifferentiated Quaternary(<1.6 Million Years), Inferred Location
- Class B (Various Age), Well Constrained Location
- Class B (Various Age), Moderately Constrained Location
- Class B (Various Age), Inferred Location

BASE MAP SOURCE:
CITY OF CUPERTINO, COUNTY OF SANTA CLARA, CALIFORNIA STATE PARKS, ESRI, TOMTOM, GARMIN, SAFEGRAPH, GEOTECHNOLOGIES, INC, METI/NASA, USGS, BUREAU OF LAND MANAGEMENT, EPA, NPS, USDA, USFWS, CSUMB, ESRI, GARMIN, NATURALVUE, ESRI, GEBCO, GARMIN, NATURALVUE, COUNTY OF SANTA CLARA, CALIFORNIA STATE PARKS, ESRI, TOMTOM, GARMIN, SAFEGRAPH, FAO, METI/NASA, USGS, BUREAU OF LAND MANAGEMENT, EPA, NPS, USFWS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
C.G.S. HISTORIC EARTHQUAKE DATABASE

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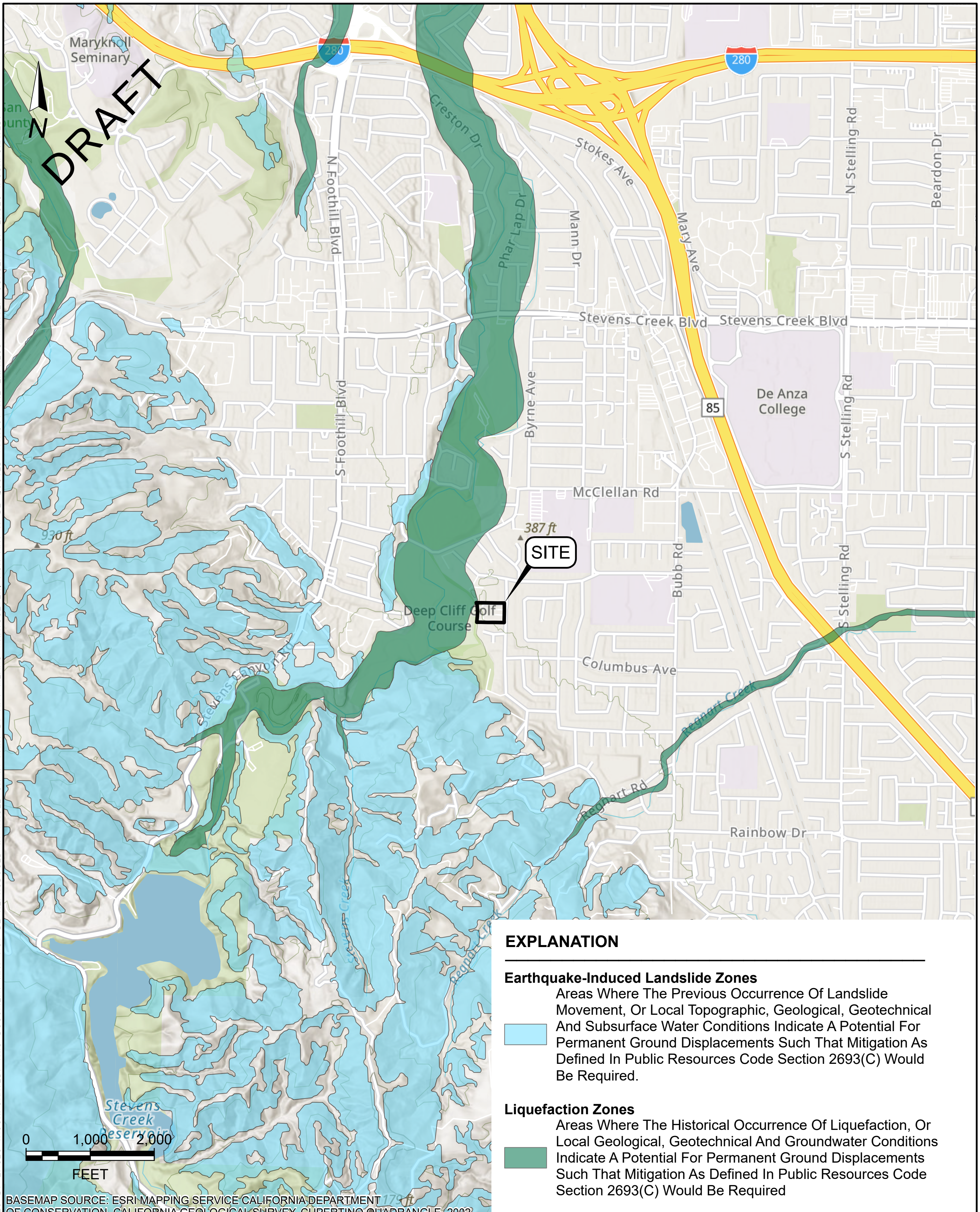
REGIONAL FAULTING AND SEISMICITY MAP
10857 LINDA VISTA DRIVE
CUPERTINO, CALIFORNIA

PROJECT NO.: 25712.000.001	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: NWC	CHECKED BY: JTR

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25712\25712000001\GEOTECH\GEX\10857 LINDA VISTA DRIVE.APRX
LAYOUT: 04. FAULT&SEISMICITY 11 X 17 LANDSCAPE USER: MHAWKINS

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

Earthquake-Induced Landslide Zones

Areas Where The Previous Occurrence Of Landslide Movement, Or Local Topographic, Geological, Geotechnical And Subsurface Water Conditions Indicate A Potential For Permanent Ground Displacements Such That Mitigation As Defined In Public Resources Code Section 2693(C) Would Be Required.

Liquefaction Zones

Areas Where The Historical Occurrence Of Liquefaction, Or Local Geological, Geotechnical And Groundwater Conditions Indicate A Potential For Permanent Ground Displacements Such That Mitigation As Defined In Public Resources Code Section 2693(C) Would Be Required

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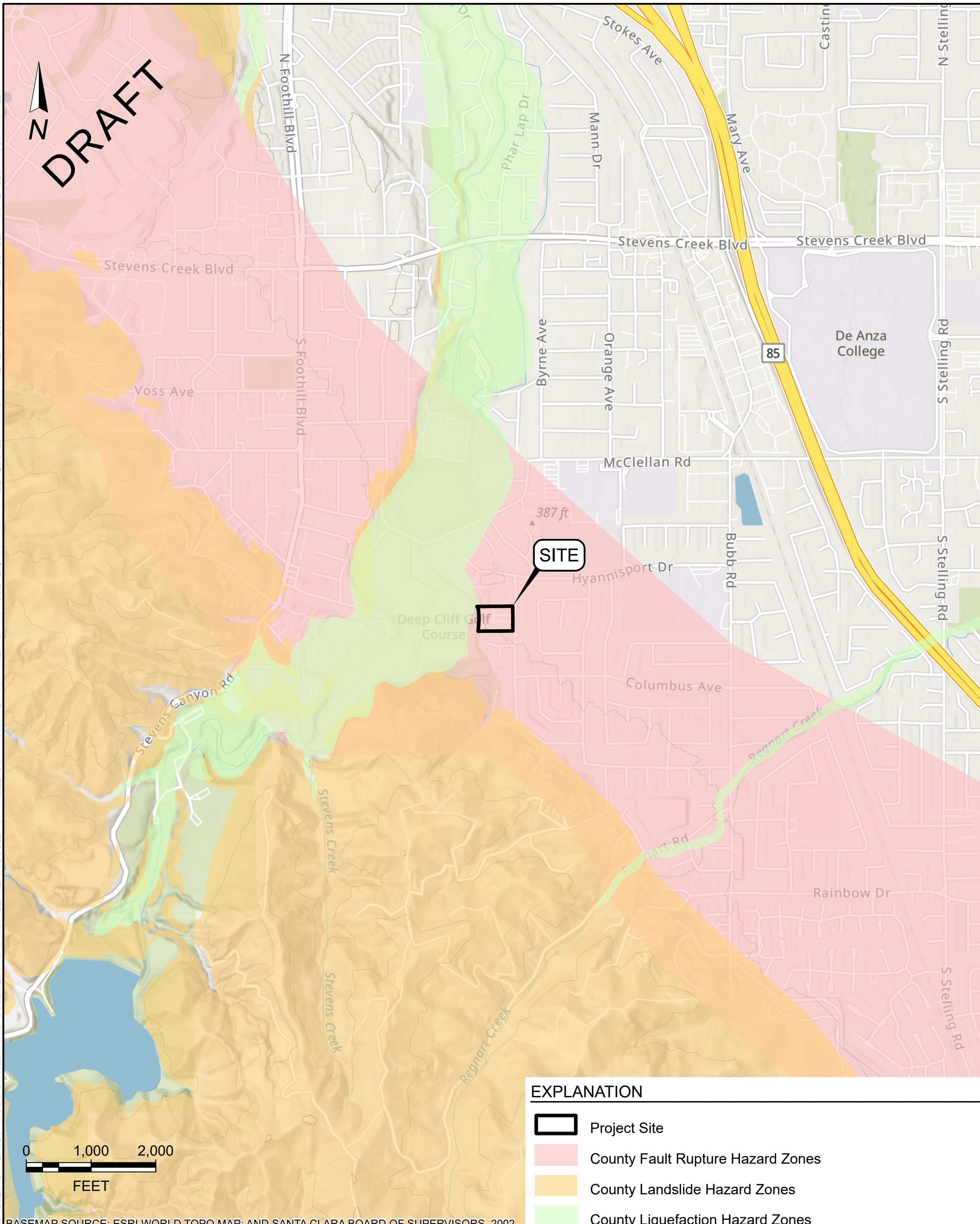
SEISMIC HAZARD ZONES MAP
10857 LINDA VISTA DRIVE
CUPERTINO, CALIFORNIA

PROJECT NO. : 25712.000.001		FIGURE NO. 5
SCALE: AS SHOWN		
DRAWN BY: NWC	CHECKED BY: JTR	

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25712\25712000001\GEOTECH\GEX\10857 LINDA VISTA DRIVE.APRX
LAYOUT: 05. SEISMIC HAZARD ZONE MAP 8.5 X 11 PORTRAIT USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR

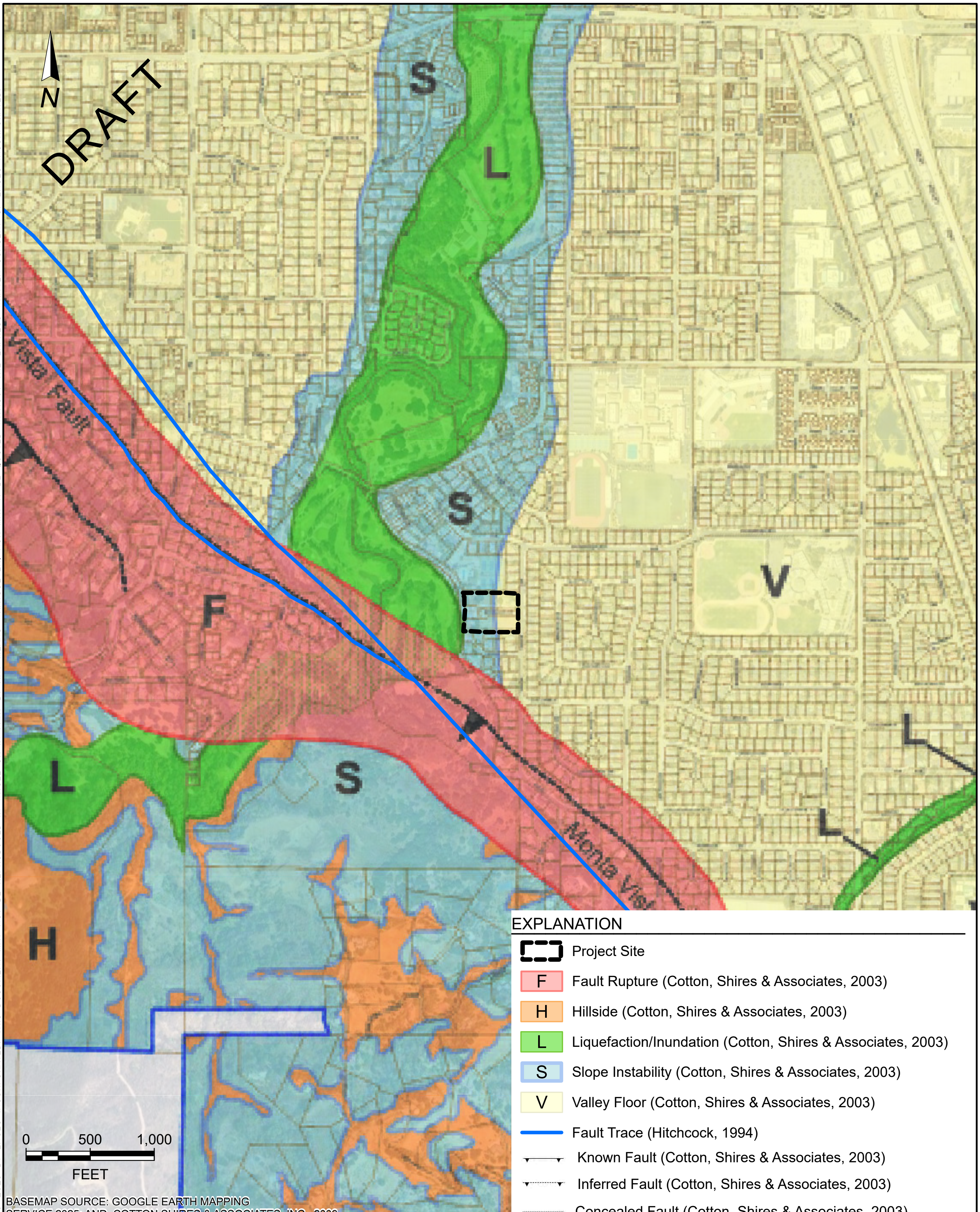


ENGEO Expect Excellence

SANTA CLARA COUNTY GEOLOGIC HAZARD ZONES 10857 LINDA VISTA DRIVE CUPERTINO, CALIFORNIA	PROJECT NO.: 25712.000.001		FIGURE NO. 6
	SCALE: AS SHOWN		
	DRAWN BY: NWC	CHECKED BY: JTR	

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25712\25712000001\GEOTECH\LINDA VISTA FAULT EVALUATION\25712000001_LINDAVISTAFAULTEVALUATION.APRX
LAYOUT: 05. SANTA CLARA COUNTY GEOLOGIC HAZARD ZONES 8.5 X 11 PORTRAIT USER: NCLOUGH

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- Project Site
- F Fault Rupture (Cotton, Shires & Associates, 2003)
- H Hillside (Cotton, Shires & Associates, 2003)
- L Liquefaction/Inundation (Cotton, Shires & Associates, 2003)
- S Slope Instability (Cotton, Shires & Associates, 2003)
- V Valley Floor (Cotton, Shires & Associates, 2003)
- Fault Trace (Hitchcock, 1994)
- Known Fault (Cotton, Shires & Associates, 2003)
- Inferred Fault (Cotton, Shires & Associates, 2003)
- Concealed Fault (Cotton, Shires & Associates, 2003)

ENGEO Expect Excellence	CITY OF CUPERTINO GEOLOGIC AND SEISMIC HAZARDS MAP 10857 LINDA VISTA DRIVE CUPERTINO, CALIFORNIA	PROJECT NO.: 25712.000.001 SCALE: AS SHOWN DRAWN BY: NWC CHECKED BY: JTR	FIGURE NO. 7

ORIGINAL FIGURE PRINTED IN COLOR

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25712\25712000001\GEOTECH\GEX\10857 LINDA VISTA DRIVE.APRX
LAYOUT: 07. CITY OF CUPERTINO GEOLOGIC AND SEISMIC HAZARDS MAP 8.5 X 11 PORTRAIT USER: NCLOUGH

APPENDIX A

KEY TO BORING LOGS
BORING LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMITS 50% OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH - Inorganic silt with high plasticity
			CH - Inorganic clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)[1]
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	APPROXIMATE SHEAR STRENGTH (PSF)[2]
VERY SOFT	0-250
SOFT	250-500
MEDIUM STIFF	500-1,000
STIFF	1,000-2,000
VERY STIFF	2,000-4,000
HARD	> 4,000

MOISTURE CONDITION

DRY	Absence of moisture, dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

Solid - Layer Break

Dashed - Gradational or approximate layer break

SAMPLER SYMBOLS

- Modified California (3-inch O.D.) Sampler
- California (2.5-inch O.D.) Sampler
- S.P.T. Split Spoon (2-inch O.D.) Sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- No Recovery

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

NOTES

1. Standard Penetration Tests (S.P.T.) number of blows for a 140-pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8-inch I.D.) sampler, assuming 60% hammer efficiency
2. Approximate shear strength measured in field at time of drilling in units of pounds per square foot

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FILE PATH: C:\Users\imccreery\OneDrive - ENGEO_Shared Files\BoreDM\Key to Borelogs\KeyToBoringLogs_BoreDM_rev1.dwg SAVE DATE: 6/12/2024 12:56:04 PM SAVED BY: imccreery

SOIL BORING 2-B2

ENGEO
Expect Excellence

LATITUDE: 37.31026 LONGITUDE: -122.06034

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED:	04/03/2025	LOGGED BY / REVIEWED BY:	Q. Parker / ZAC
HOLE DEPTH:	30 ft	DRILLING CONTRACTOR:	Hanlon Drilling
HOLE DIAMETER:	4 in	DRILLING METHOD:	Solid Flight Auger
SURFACE ELEV.:	393.5 ft (NAVD88)	HAMMER TYPE:	140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SANDY LEAN CLAY (**CL**), reddish brown, stiff to hard, moist, low plasticity, fine- to coarse-grained sand, subrounded fine gravel [Qpaf]											
			approximately upper 18 inches disturbed			11	25	11	58			2*		PP
												2*		PP
												4.5*		PP
	390													
						31						>4.5*		PP
5			**SANDY LEAN CLAY WITH GRAVEL** (**CL**), reddish brown, hard, moist, approximately 30% fine- to coarse-grained sand, approximately 15 to 20% subangular fine gravel									>4.5*		PP
			CLAYEY SAND WITH GRAVEL (**SC**), reddish brown, dense, moist, fine- to coarse-grained sand, approximately 30 to 40% subangular to subrounded fine gravel, approximately 15 to 20% fines			37								
	385													
10			becomes yellowish brown to reddish brown, grades to fine to coarse gravel, contains interbedded fine-grained sand lenses			32								
	380													
15			**SILTY SAND** (**SM**), yellowish red, loose, moist, fine-grained sand, low plasticity fines			11			28					
						10								
	375		**CLAYEY GRAVEL WITH SAND** (**GC**), reddish brown, very dense, moist, subrounded fine to coarse gravel, approximately 20 to 30% fine- to coarse-grained sand, approximately 15 to 20% fines			90								

SOIL BORING 2-B2

ENGEO
Expect Excellence

LATITUDE: 37.31026 LONGITUDE: -122.06034

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED: 04/03/2025
HOLE DEPTH: 30 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 393.5 ft (NAVD88)

LOGGED BY / REVIEWED BY: Q. Parker / ZAC
DRILLING CONTRACTOR: Hanlon Drilling
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						90								
	370													
25			coarse gravel content decreases			61								
	365													
			grades to dry to moist			50/4"								
30														

End of boring at approximately 30 feet below the existing ground surface. Groundwater not encountered during drilling.

SOIL BORING 2-B3

ENGEO — Expect Excellence

LATITUDE: 37.30989 LONGITUDE: -122.06006

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED:	04/03/2025	LOGGED BY / REVIEWED BY:	Q. Parker / ZAC
HOLE DEPTH:	32 ft	DRILLING CONTRACTOR:	Hanlon Drilling
HOLE DIAMETER:	4 in	DRILLING METHOD:	Mud Rotary Wash
SURFACE ELEV.:	393.5 ft (NAVD88)	HAMMER TYPE:	140 lb. Auto Trip

Depth (ft)	Elevation (ft)	MATERIAL DESCRIPTION	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
		SANDY LEAN CLAY (**CL**), reddish brown, medium stiff to very stiff, moist, low plasticity, fine- to coarse-grained sand, subangular to subrounded fine gravel [Qpaf]									
		approximately upper 18 inches disturbed	12	23	10	52					
									3*		PP
									3*		PP
	390										
							13	119			
			33								
5									4*		PP
		CLAYEY SAND WITH GRAVEL (**SC**), reddish brown, medium dense, moist, approximately 15 to 20% subangular fine gravel, approximately 15 to 20% fines							>4.5*		PP
		LEAN CLAY WITH SAND (**CL**), reddish brown, stiff, low plasticity, approximately 20 to 30% fine- to coarse-grained sand	29								
	385	**CLAYEY SAND WITH GRAVEL** (**SC**), reddish brown, medium dense to very dense, moist, approximately 15 to 20% subangular fine gravel, approximately 15 to 20% fines									
10											
		becomes dry to moist, dense, grades to approximately 20 to 30% subangular to subrounded fine to coarse gravel	40								
	380	switch from solid flight auger to mud rotary drilling									
15											
		grades to subangular fine gravel	53								
	375										

ENGEO
Expect Excellence

SOIL BORING 2-B3

LATITUDE: 37.30989 LONGITUDE: -122.06006

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED:	04/03/2025	LOGGED BY / REVIEWED BY:	Q. Parker / ZAC
HOLE DEPTH:	32 ft	DRILLING CONTRACTOR:	Hanlon Drilling
HOLE DIAMETER:	4 in	DRILLING METHOD:	Mud Rotary Wash
SURFACE ELEV.:	393.5 ft (NAVD88)	HAMMER TYPE:	140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			becomes dense			37								
			grades to approximately 30 to 40% gravel											
	370		**SILT** (**ML**), reddish brown to dark brown, stiff to very stiff, moist, low plasticity, <10% fine- to coarse-grained sand			36								
25			**SILT WITH GRAVEL** (**ML**), reddish brown to dark brown, hard, moist, low plasticity, approximately 15 to 25% subrounded fine gravel											
			SILTY GRAVEL WITH SAND (**GM**), reddish brown, very dense, moist, subangular to subrounded fine to coarse gravel, approximately 20 to 30% fine- to coarse-grained sand, approximately 15 to 25% fines			50/6"								
	365		**SILT** (**ML**), reddish brown, stiff to very stiff, moist, low plasticity, <10% fine- to coarse-grained sand			31								
30										20	110	2.5*		PP
						22								

End of boring at approximately 32 feet below the existing ground surface. Groundwater not measured due to drilling method.

ENGEO
Expect Excellence

SOIL BORING 2-B1

LATITUDE: 37.31033 LONGITUDE: -122.06083

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED: 04/03/2025
HOLE DEPTH: 48 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 396 ft (NAVD88)

LOGGED BY / REVIEWED BY: Q. Parker / ZAC
DRILLING CONTRACTOR: Hanlon Drilling
DRILLING METHOD: Mud Rotary Wash
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	395		**CLAYEY SAND** (**SC**), reddish brown, medium dense, moist, low plasticity, fine- to coarse-grained sand, approximately 30 to 40% fines, approximately 5 to 10% subrounded to subangular fine gravel [Qpaf] approximately upper 18 inches disturbed			16								
5	390		**CLAYEY GRAVEL WITH SAND** (**GC**), reddish brown to light gray, dense to very dense, moist, subrounded fine gravel, approximately 20 to 30% fines, approximately 20 to 30% fine- to coarse-grained sand, iron oxide staining present			72								
			thin interbeds of fine- to coarse-grained sand											
			CLAYEY SAND (**SC**), yellowish brown to yellowish red, dense, moist, fine- to coarse-grained sand, <15% subangular fine gravel			53								
10	385					41			14					
15	380		**POORLY GRADED SAND WITH CLAY AND GRAVEL** (**SP-SC**), yellowish brown to yellowish red, very dense, moist, medium- to coarse-grained sand, approximately 15 to 25% subangular fine to coarse gravel, iron oxide staining present			55								

ENGEO
Expect Excellence

SOIL BORING 2-B1

LATITUDE: 37.31033 LONGITUDE: -122.06083

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED: 04/03/2025
HOLE DEPTH: 48 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 396 ft (NAVD88)

LOGGED BY / REVIEWED BY: Q. Parker / ZAC
DRILLING CONTRACTOR: Hanlon Drilling
DRILLING METHOD: Mud Rotary Wash
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	375					57								
			increase in clay content											
			CLAYEY SAND WITH GRAVEL (SC), yellowish brown to yellowish red, dense to very dense, moist, fine- to coarse-grained sand, subangular to subrounded fine to coarse gravel			95								
25														
	370								15					
						73								
30			grades to approximately 20 to 30% subangular to subrounded fine to coarse gravel											
	365					75								
35			iron oxide staining present											
	360					95								

SOIL BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 37.31033 LONGITUDE: -122.06083

10857 Linda Vista Drive
Cupertino, CA
25712.000.001

DATE DRILLED: 04/03/2025
HOLE DEPTH: 48 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 396 ft (NAVD88)

LOGGED BY / REVIEWED BY: Q. Parker / ZAC
DRILLING CONTRACTOR: Hanlon Drilling
DRILLING METHOD: Mud Rotary Wash
HAMMER TYPE: 140 lb. Auto Trip

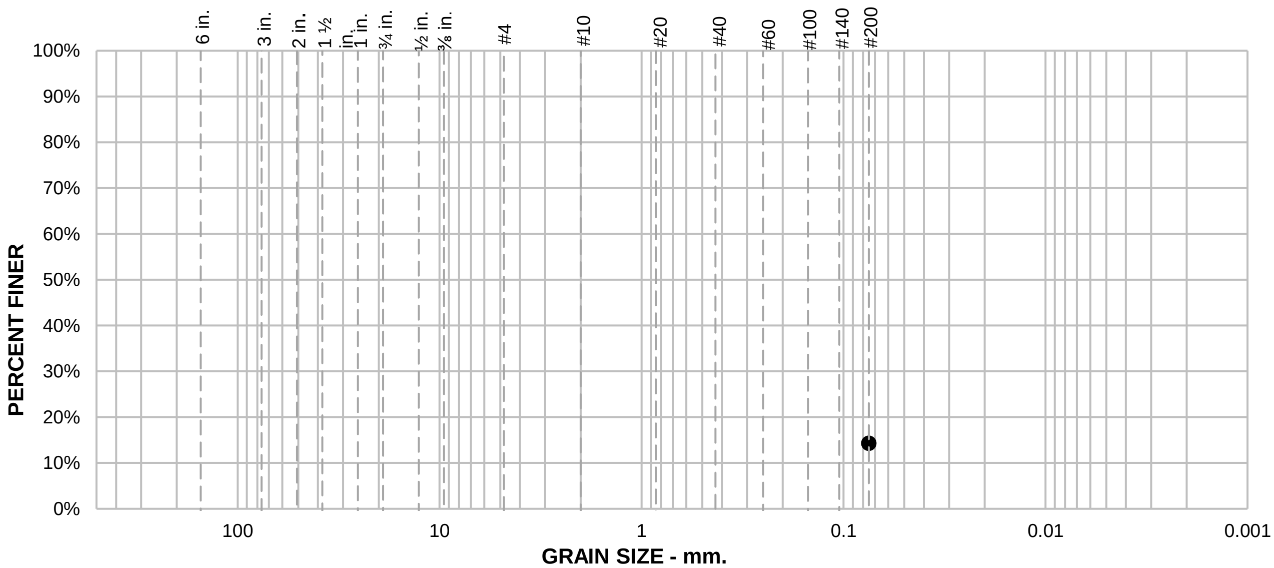
Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			grades to subangular fine gravel			90/11"								
	355													
45														
	350					92								
			contains coarse gravel			64								

End of boring at approximately 48 feet below the existing ground surface. Groundwater not measured due to drilling method.

APPENDIX B

LABORATORY TEST DATA

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 2-B1@10'

DEPTH (ft): 10

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						14	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	14		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 725.82 g

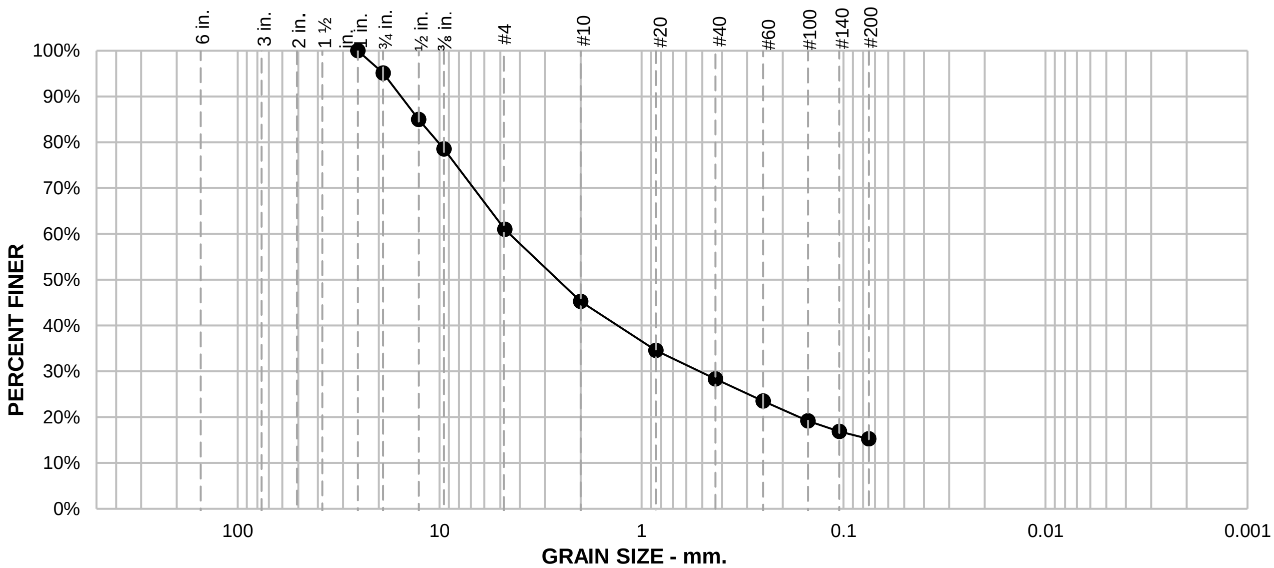
* (no specification provided)

CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

Expect Excellence

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 2-B1@26'

DEPTH (ft): 26

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	5	34	16	17	13	15	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
¾ in.	95		
½ in.	85		
⅜ in.	79		
#4	61		
#10	45		
#20	35		
#40	28		
#60	23		
#100	19		
#140	17		
#200	15		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 15.5543 mm	D_{85} = 12.7000 mm	D_{60} = 4.5000 mm
D_{50} = 2.6207 mm	D_{30} = 0.5224 mm	D_{15} = 0.0750 mm
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

ENGEO Expect Excellence

CLIENT: SummerHill Homes

PROJECT NAME: 10857 Linda Vista Drive

PROJECT NO: 25712.000.001 PH003

PROJECT LOCATION: Cupertino, CA

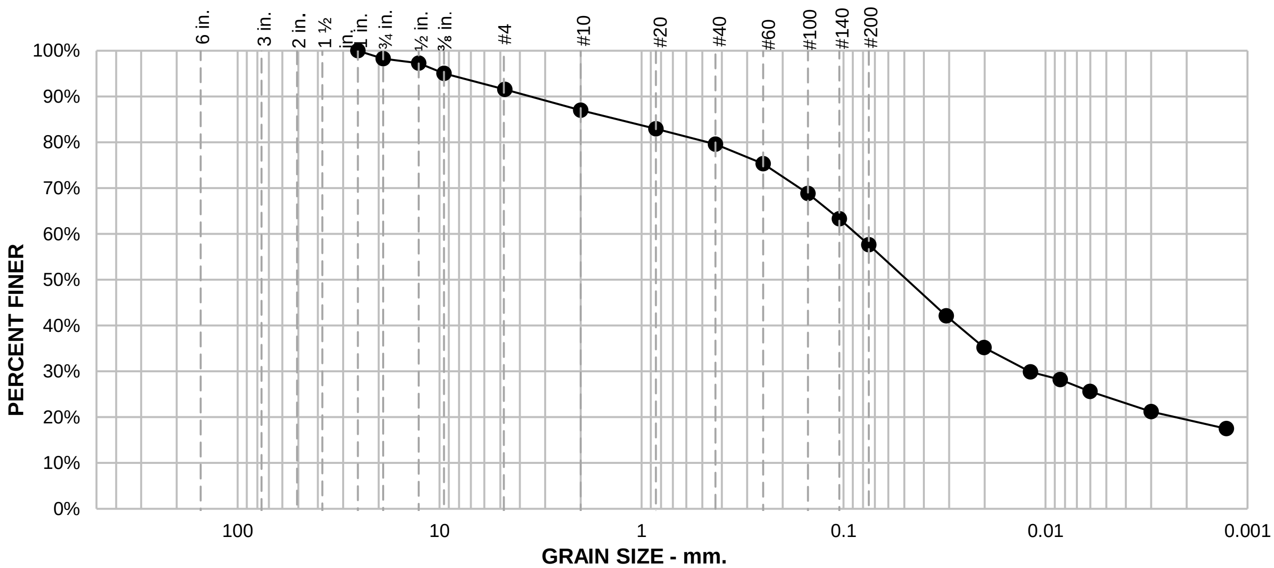
REPORT DATE: 4/25/2025

TESTED BY: Y. Cabrales

REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 2-B2@2

DEPTH (ft): 2

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	1.8	6.6	4.6	7.4	22.0	38.1	19.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
¾ in.	98.2		
½ in.	97.3		
⅜ in.	95.1		
#4	91.6		
#10	87.0		
#20	83.0		
#40	79.6		
#60	75.4		
#100	68.8		
#140	63.3		
#200	57.6		
0.0310 mm.	42.1		
0.0201 mm.	35.2		
0.0119 mm.	29.9		
0.0085 mm.	28.2		
0.0060 mm.	25.6		
0.0030 mm.	21.2		
0.0013 mm.	17.5		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 14	LL = 25	PI = 11

COEFFICIENTS

D_{90} = 3.5158 mm	D_{85} = 1.3038 mm	D_{60} = 0.0864 mm
D_{50} = 0.0486 mm	D_{30} = 0.0120 mm	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS = CL

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

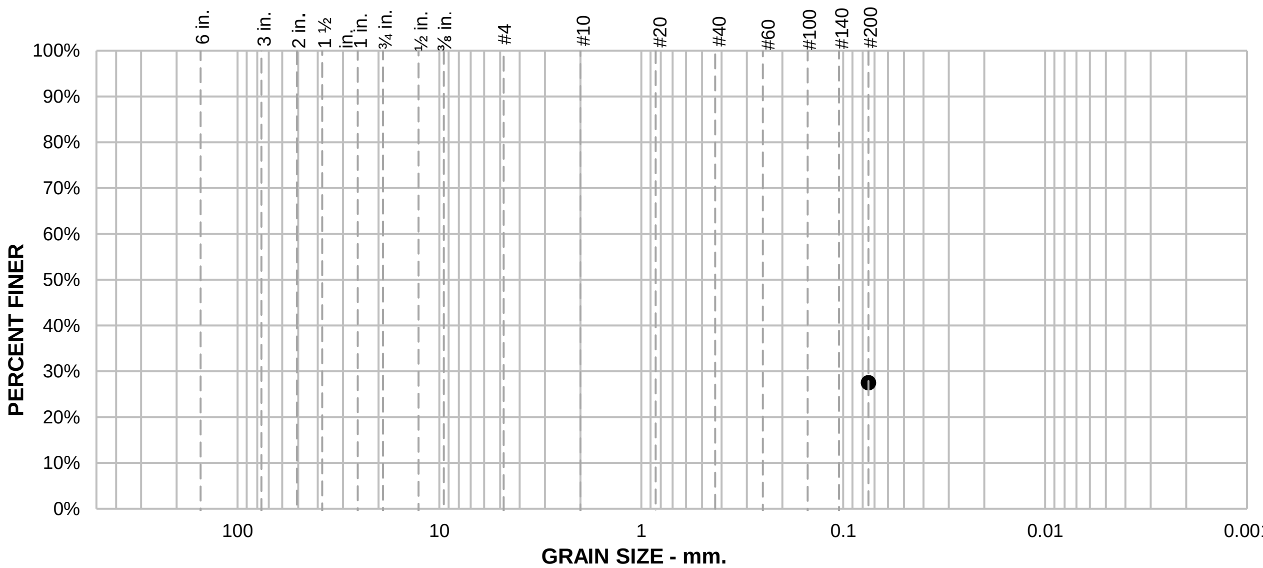
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 2-B2@15.5-16'

DEPTH (ft): 15.5-16

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						27.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	27.5		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

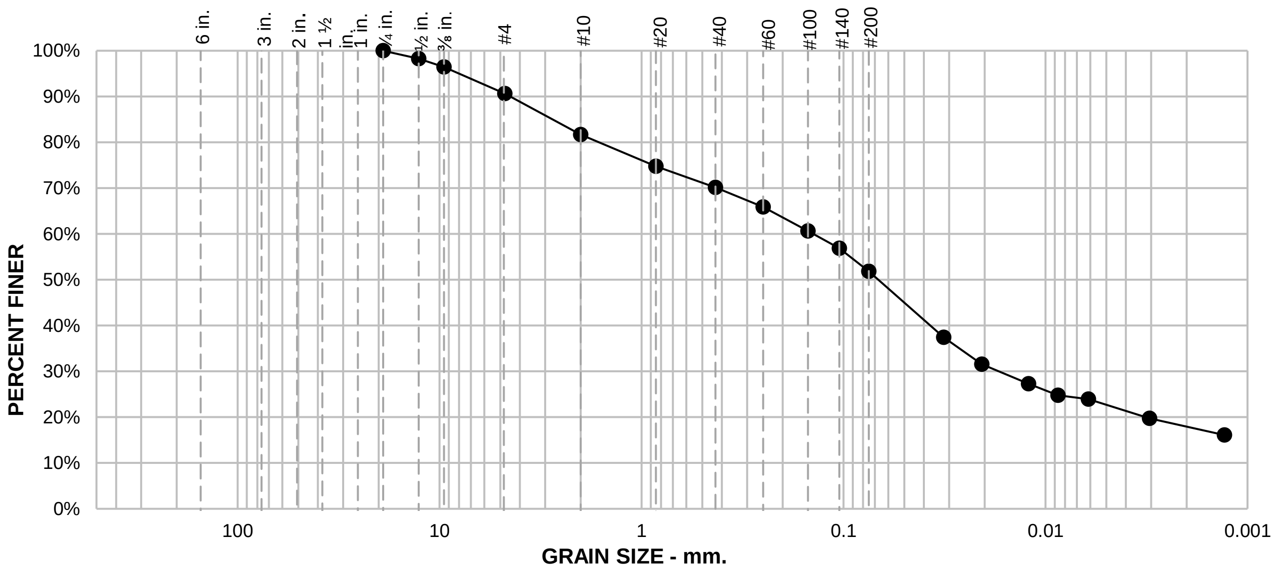
REMARKS

Soak time = 180 min
Dry sample weight = 98.61 g

* (no specification provided)

CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D422



SAMPLE ID: 2-B3@1.5-2'

DEPTH (ft): 1.5-2

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		9.3	9.0	11.5	18.4	33.9	17.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100.0		
½ in.	98.2		
⅜ in.	96.5		
#4	90.7		
#10	81.7		
#20	74.7		
#40	70.2		
#60	65.9		
#100	60.6		
#140	56.9		
#200	51.8		
0.0319 mm.	37.4		
0.0207 mm.	31.5		
0.0121 mm.	27.3		
0.0087 mm.	24.8		
0.0061 mm.	24.0		
0.0031 mm.	19.7		
0.0013 mm.	16.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 13 LL = 23 PI = 10

COEFFICIENTS

D_{90} = 4.4409 mm D_{85} = 2.7465 mm D_{60} = 0.1416 mm
D_{50} = 0.0674 mm D_{30} = 0.0171 mm D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS = CL

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

* (no specification provided)

ENGEO
Expect Excellence

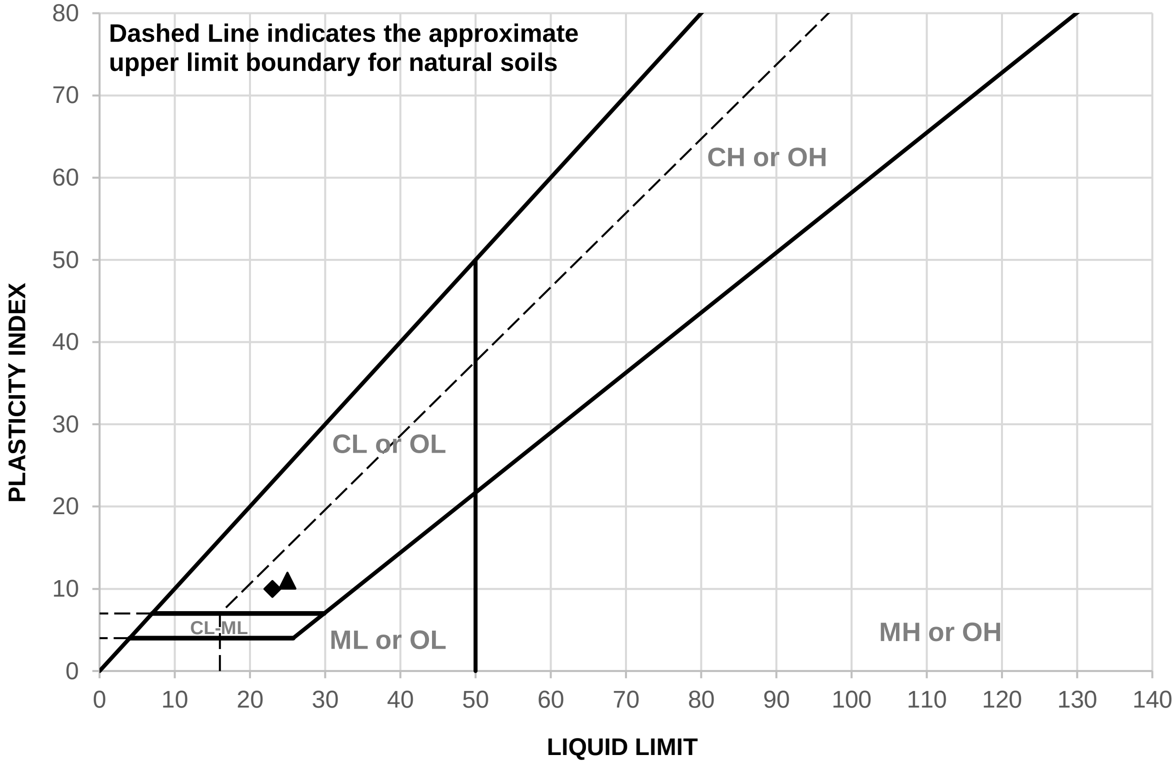
CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	2-B3 @30'							
DEPTH (ft.)	30							
METHOD A OR B	B							
MOISTURE CONTENT (%)	19.5							
DRY DENSITY (pcf)	109.9							

CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	2-B2@2'	2	See exploration logs	25	14	11
◆	2-B3@1.5-2'	1.5-2	See exploration logs	23	13	10

	SAMPLE ID	TEST METHOD	REMARKS
▲	2-B2@2'	PI: ASTM D4318, Wet Method	
◆	2-B3@1.5-2'	PI: ASTM D4318, Wet Method	

ENGEO
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CLIENT: SummerHill Homes
PROJECT NAME: 10857 Linda Vista Drive
PROJECT NO: 25712.000.001 PH003
PROJECT LOCATION: Cupertino, CA
REPORT DATE: 4/25/2025
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

24 April 2025

Job No. 2504054
Cust. No. 12963

Ms. Lauren Loey
ENGEO Incorporated
17278 Golden Valley Parkway
Lathrop, CA 95330

Subject: Project No.: 25712.000.001
Project Name: 10857 Linda Vista Drive, Cupertino, CA
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Ms. Loey:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 22, 2025. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The pH of the soil is 7.28 and 7.97, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are both 240-mV and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 25712.000.001
Client's Project Name: 10857 Linda Vista Drive, Cupertino, CA
Date Sampled: 3-Apr-25
Date Received: 22-Apr-25
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 24-Apr-2025

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2504054-001	2-B1 @ 1 ft [(1)]	240	7.28	-	4,800	-	N.D.	N.D.
2504054-002	2-B2 @ 1.5 ft [(2)]	240	7.97	-	5,900	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G187	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	22-Apr-2025	23-Apr-2025	-	22-Apr-2025	-	23-Apr-2025	23-Apr-2025

Julia Clauson
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected (<15 mg/kg)
(1) bag shows 2-B1 @ 1-2.5'
(2) bag shows 2-B2 @ 1-1.5'

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Chain of Custody

Page of

1100 Willow Pass Court
Concord, CA 94520-1005
925 462-2771
Fax: 925 462-2775
www.cercoanalytical.com
CERCO analytical

Job No.	CU#		Client Project I.D.
2504054	129663		25712.000.001

Schedule
Analyte

Date Sampled | Date Due

Full Name
10857 Linda Vista Drive
25712.000.001

Phone X
Fax

Company and/or Mailing Address
ENGEO Incorporated, 17278 Golden Valley Parkway,
Lathrop, CA 95330

Cell 925-364-4098 ☒

Sample Source
10857 Linda Vista Drive, Cupertino, CA

ANALYSIS — ASTM w/Brief Evaluation

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation							
1	2-B1 at 1 ft	4/3	10am	Soil	ziptop bag	quart	n/a	1	x	x	x	x	x		x		*bag shows @ 1-2.5					
2	2-B2 at 1.5 ft	4/3	8 am	Soil	2.5x6" liner bag	2.5x6"	n/a	1	x	x	x	x	x		x		" " @ 1-1.5					

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. a t Lab -°C
Sampler

	Date	Time
Relinquished By:		
Received By: Julie Ch	4/22/25	1005
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:
THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM TUBES

Email Address: Lauren Loey, lloey@engeo.com

APPENDIX C

PREVIOUS EXPLORATIONS (ENGEO)

Cone Penetration Test Report
Trench Logs

PRESENTATION OF SITE INVESTIGATION RESULTS

10857 Linda Vista Drive

Prepared for:

ENGEO, Inc.

ConeTec Job No: **24-56-27871**

Project Start Date:	2024-06-20
Project End Date:	2024-06-20
Release Date:	2024-06-25

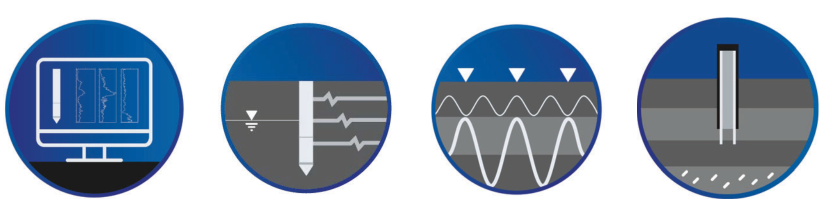
Report Prepared by:

ConeTec, Inc.

506 De Carlo Ave, Richmond, CA 94801

Tel: (510) 357-3677

ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com



CONETEC

ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. for ENGEO, Inc..

Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report. Please refer to the list of attached documents following the text of this report. A site map, test summaries, and test plots are all included in the body of the report.

Project	
Client	ENGEO, Inc.
Project	10857 Linda Vista Drive
ConeTec Project Number	24-56-27871
Test Types	CPTu/SCPTu
Additional Comments	None

Contents

The following listed below are included in the body of this report:

- **Site Map**
- **Limitations and Closure**
- **Project Information**
- **Report Appendices**
- **Supporting Documents and Materials**

SITE MAP

All locations are approximate unless otherwise stated in the body of the report.

ConeTec Job Number: 24-56-27871

Client: ENGEO, Inc.

Project: 10857 Linda Vista Drive

Date: 2024-06-25

LIMITATIONS

3rd Party Disclaimer

The "Report" refers to this report titled: 10857 Linda Vista Drive

The Report was prepared by ConeTec for: ENGEO, Inc.

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by: ENGEO, Inc.

The "Report" refers to this report titled: 10857 Linda Vista Drive

ConeTec was retained to collect and provide the raw data ("Data") which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

Closure

Thank you for the opportunity to work on this project. The equipment used as well the field procedures followed, all complied with current accepted best practice standards.

Report prepared by: Alan Sweeney

Anthony Rasmussen

PROJECT INFORMATION

Rig		
Description	**Deployment System**	**Test Type**
C02-015 CPT Truck Rig	Twin mounted cylinders	CPTu/SCPTu

Coordinates		
Test Type	**Collection Method**	**EPSG Number**
CPTu/SCPTu	Consumer Grade GPS	32610 (WGS84 / UTM Zone 10 North)

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC1061:T1500F15U35	1061	15	225	1500	15	35
The CPTu summary indicates which cone was used for each sounding.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.
Additional Comments	None

Calculated Geotechnical Parameters	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

REPORT APPENDICES

The appendices listed below are included in the report:

- **Cone Penetration Test (CPTu) Summary and Standard CPTu Plots**
- **Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic**
- **Soil Behavior Type (SBT) Scatter Plots**
- **Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots**
- **Seismic Cone Penetration Test (SCPTu) Tabular Results**
- **SCPTu Test Plots**
- **SCPTu Velocity Wave Traces**
- **Supplementary Documents and Materials**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No:	24-56-27871
Client:	ENGEO, Inc.
Project:	10857 Linda Vista Drive
Start Date:	20-Jun-2024
End Date:	20-Jun-2024

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Seismic Intervals	Northing[2] (m)	Easting[2] (m)	Surface Elevation[3] (ft)	Refer to Notation Number
1-CPT1	24-56-27871_SP01	2024-06-20	1061:T1500F15U35	15	>58	58.40	18	4129708	583221	397	
1-CPT2	24-56-27871_CP02	2024-06-20	1061:T1500F15U35	15	>9	9.27		4129728	583270	394	
1-CPT2B	24-56-27871_CP02B	2024-06-20	1061:T1500F15U35	15	>36	36.50		4129727	583269	394	
1-CPT3	24-56-27871_CP03	2024-06-20	1061:T1500F15U35	15	>41	41.34		4129668	583245	397	
1-CPT4	24-56-27871_CP04	2024-06-20	1061:T1500F15U35	15	>44	44.46		4129677	583314	394	
Totals	5 Soundings					189.96 ft					

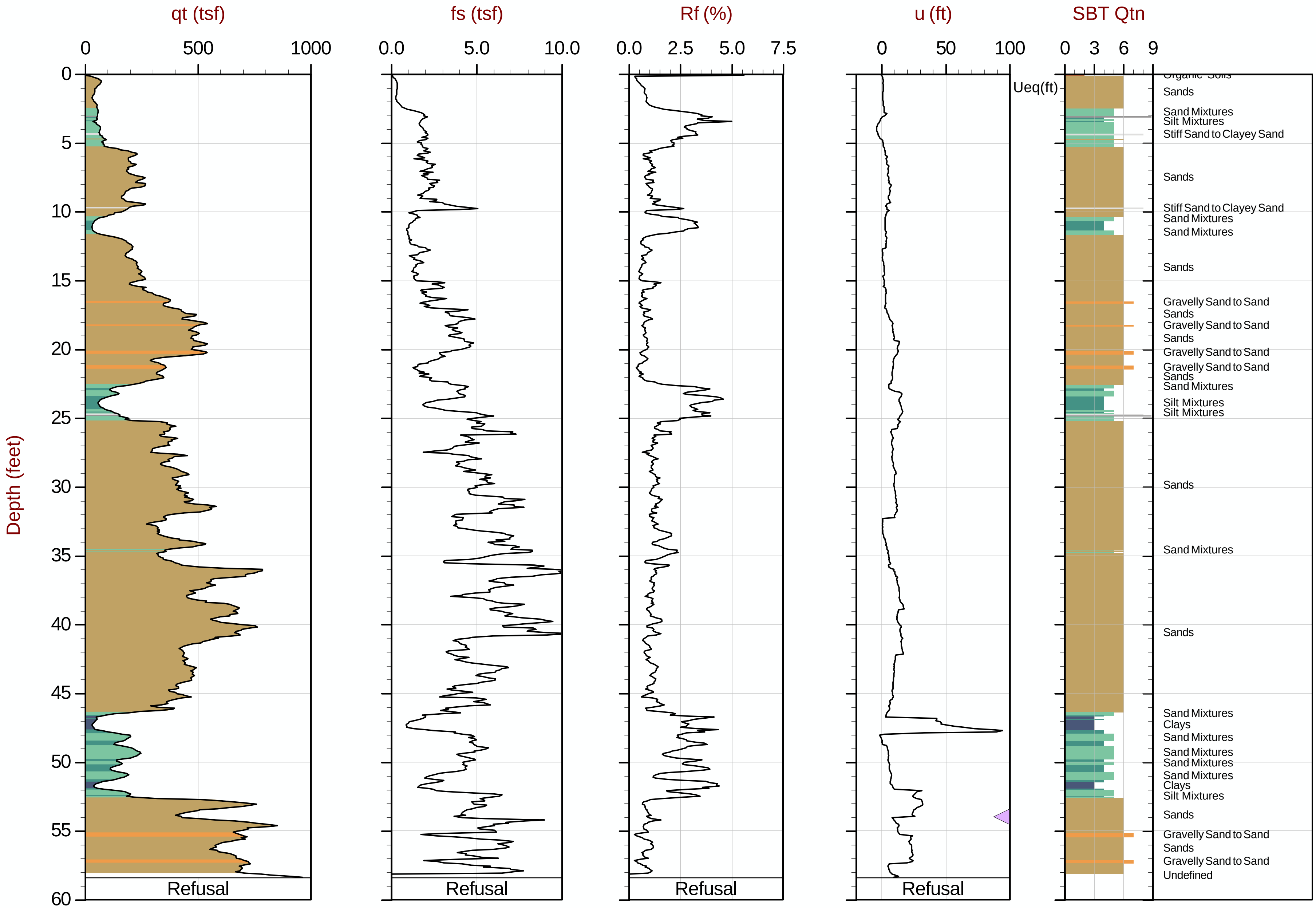
1. The phreatic surface is assumed to be beyond the depth of the sounding for the calculated parameters.
2. The coordinates were collected using a consumer grade GPS receiver. EPSG number: 32610 (WGS84 / UTM Zone 10 North).
3. Elevations taken from Google Earth.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 08:13
Site: 10857 Linda Vista Drive

Sounding: 1-CPT1
Cone: 1061:T1500F15U35



Max Depth: 17.800 m / 58.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129708m E: 583221m

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Hydrostatic Line

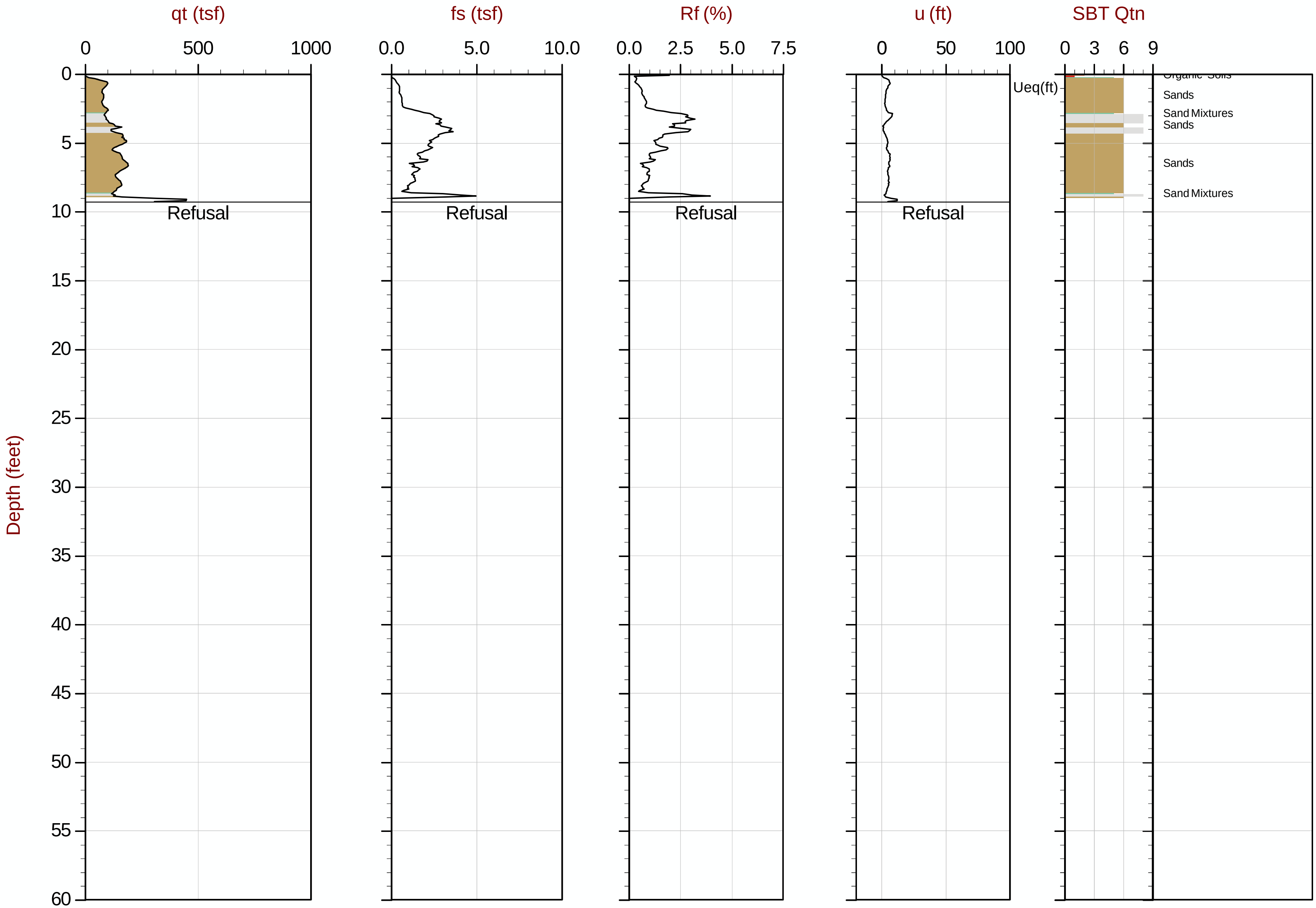
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 11:05
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2
Cone: 1061:T1500F15U35



Max Depth: 2.825 m / 9.27 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129728m E: 583270m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

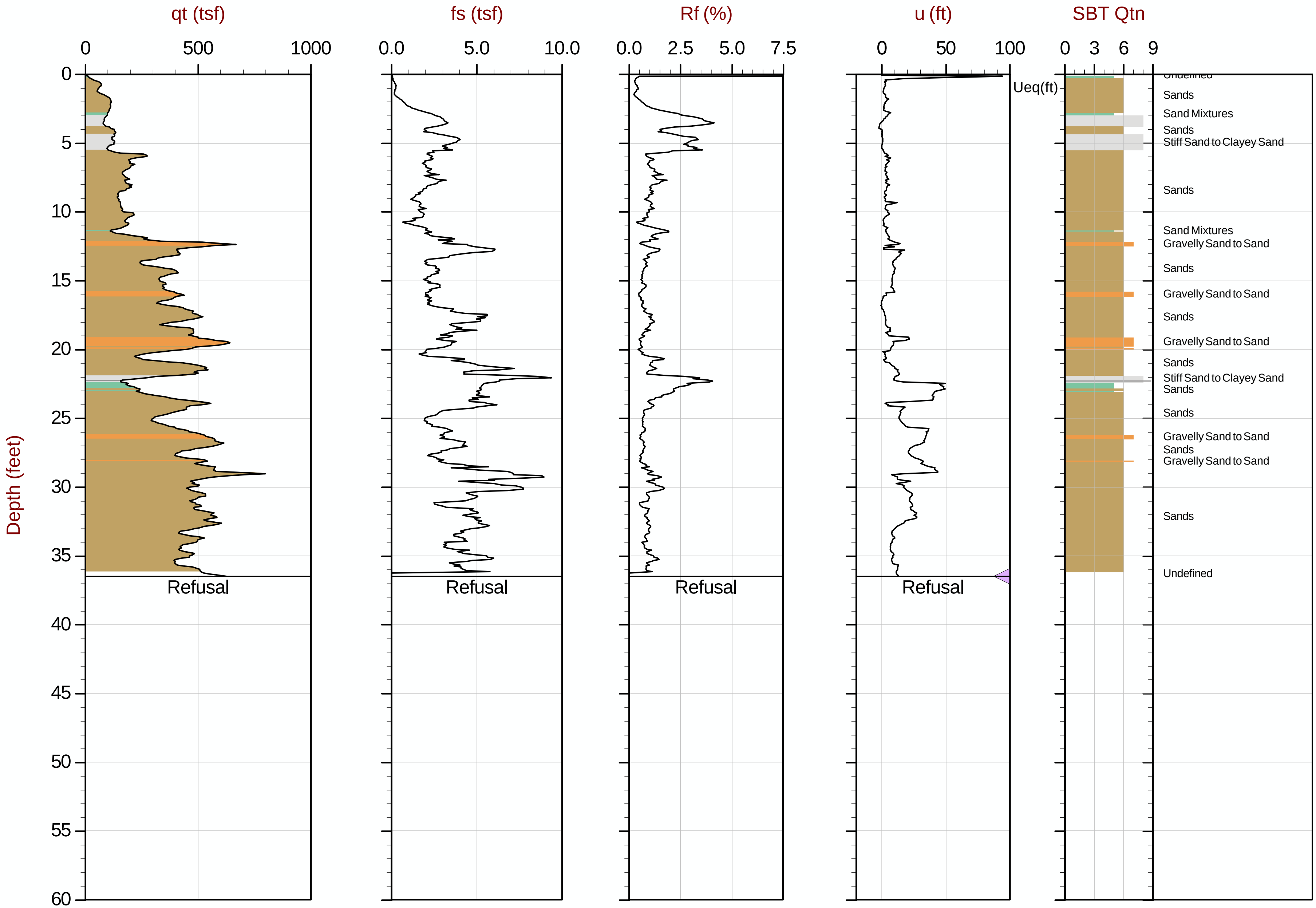
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 11:24
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2B
Cone: 1061:T1500F15U35



Max Depth: 11.125 m / 36.50 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129727m E: 583269m

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Hydrostatic Line

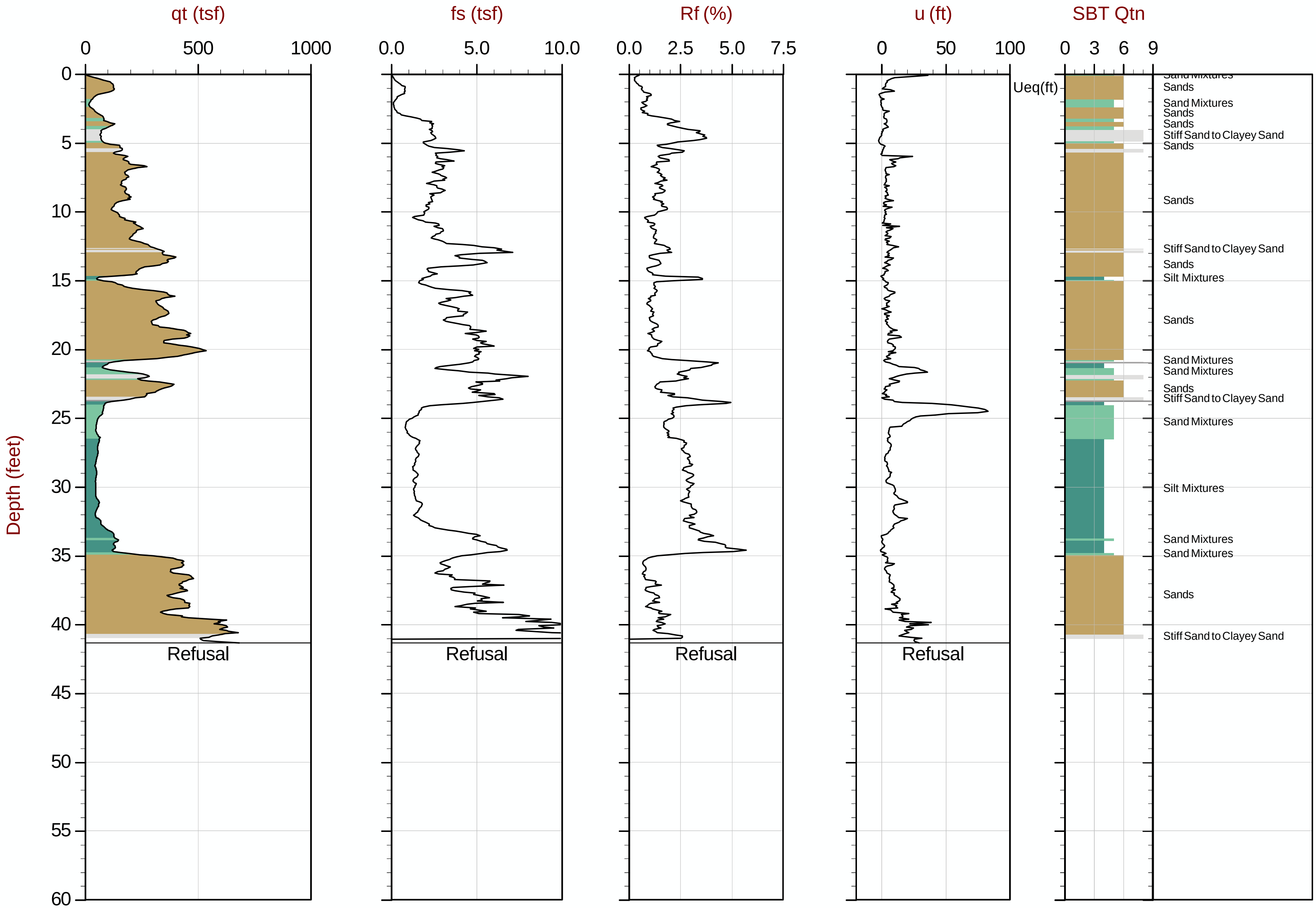
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 13:10
Site: 10857 Linda Vista Drive

Sounding: 1-CPT3
Cone: 1061:T1500F15U35



Max Depth: 12.600 m / 41.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP03.COR
Unit Wt: SBTQtn (PKR2009)

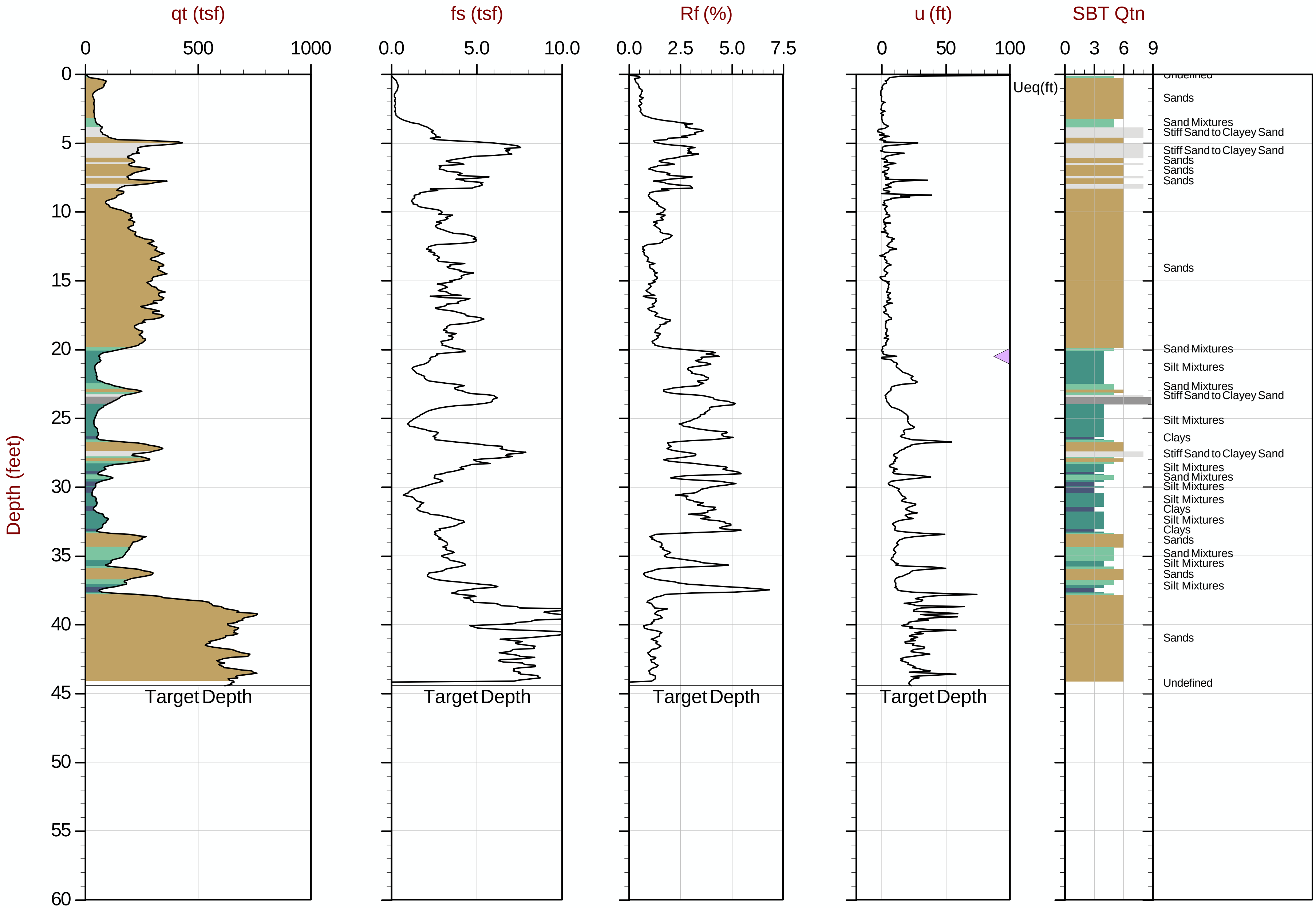
SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129668m E: 583245m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Job No: 24-56-27871
Date: 2024-06-20 12:23
Site: 10857 Linda Vista Drive
Sounding: 1-CPT4
Cone: 1061:T1500F15U35

ENGEO



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129677m E: 583314m

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

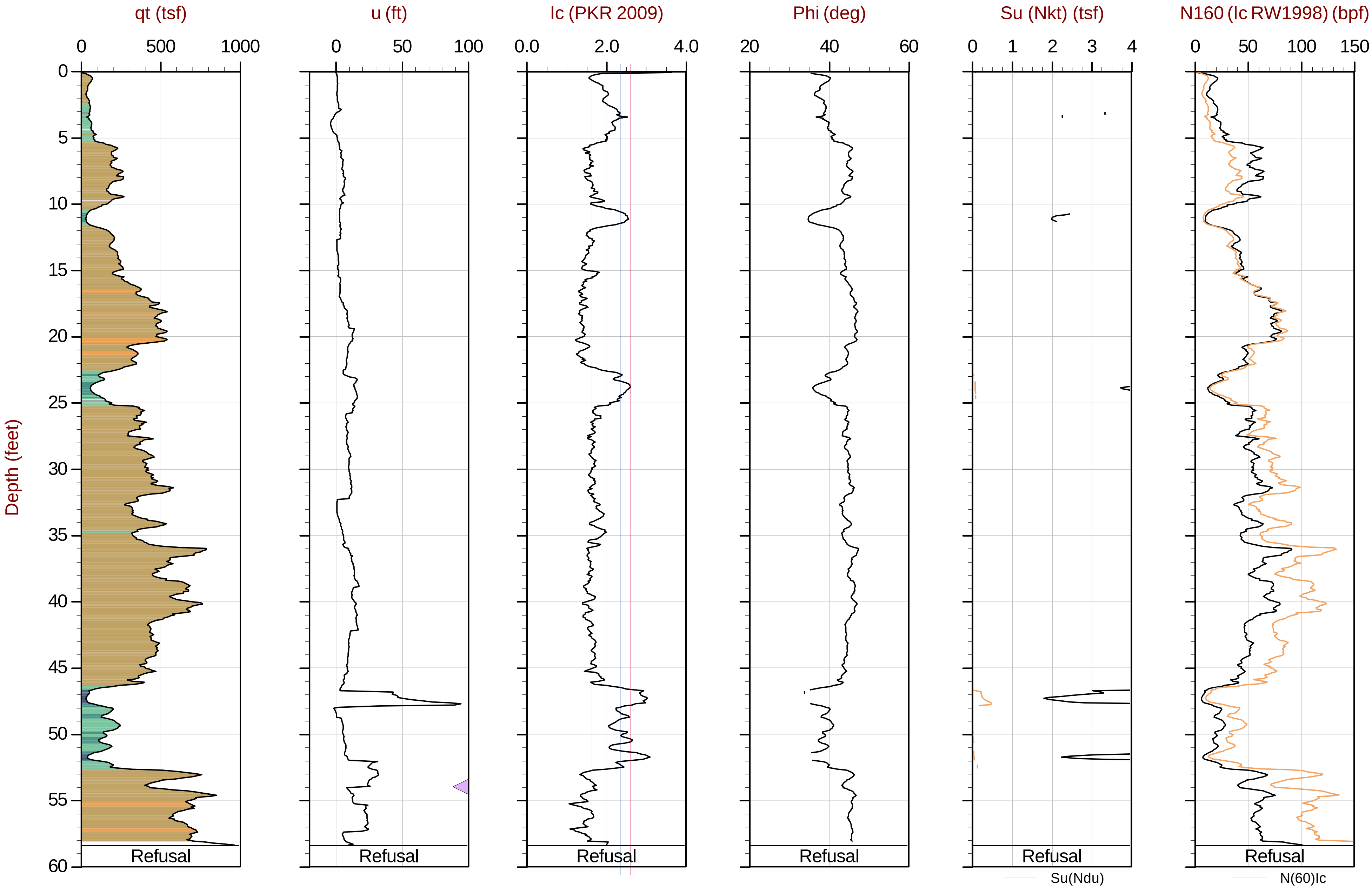
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 08:13
Site: 10857 Linda Vista Drive

Sounding: 1-CPT1
Cone: 1061:T1500F15U35



Max Depth: 17.800 m / 58.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129708m E: 583221m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

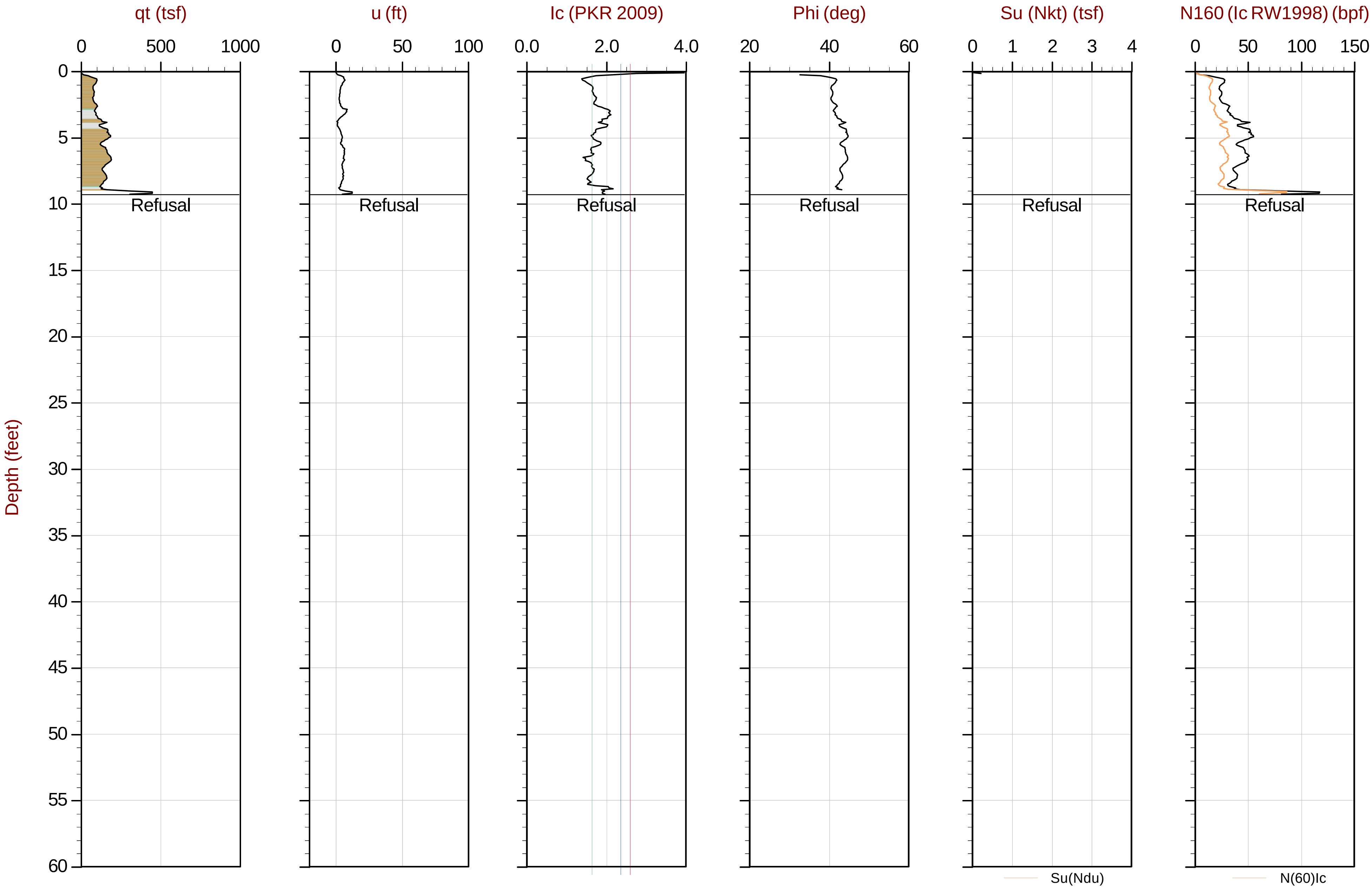
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 11:05
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2
Cone: 1061:T1500F15U35



Max Depth: 2.825 m / 9.27 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129728m E: 583270m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

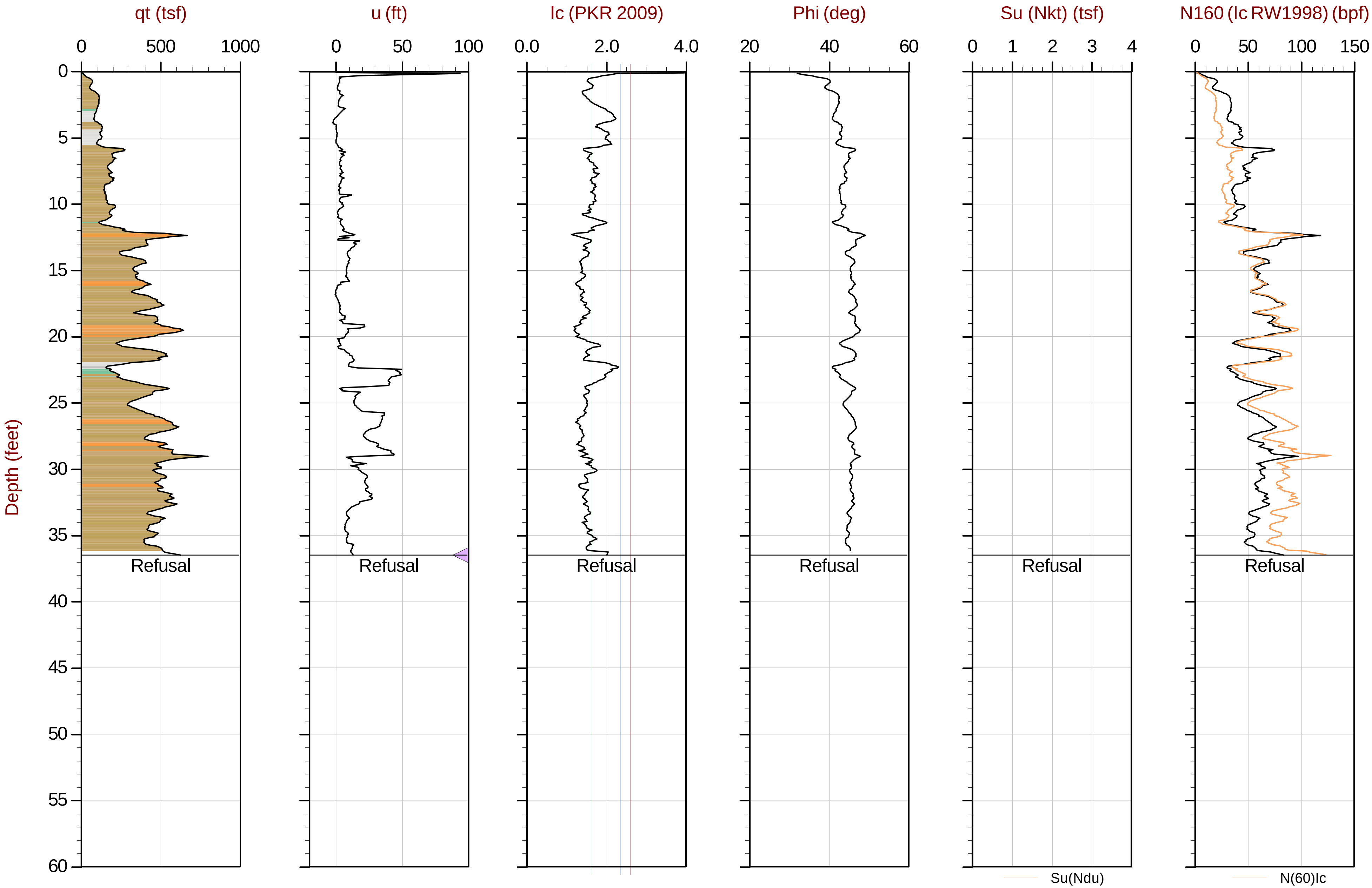
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 11:24
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2B
Cone: 1061:T1500F15U35



Max Depth: 11.125 m / 36.50 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP02B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129727m E: 583269m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

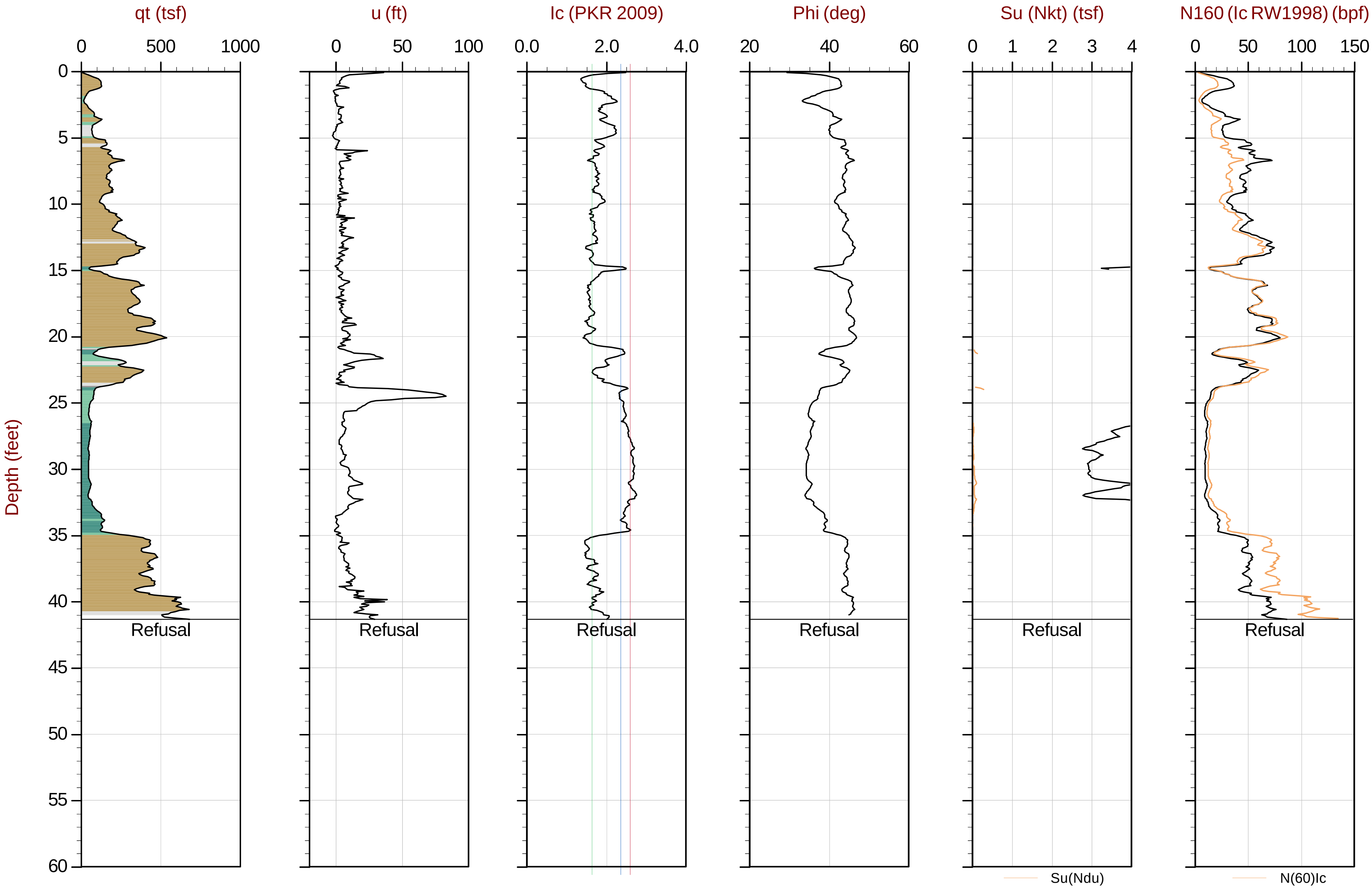
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 13:10
Site: 10857 Linda Vista Drive

Sounding: 1-CPT3
Cone: 1061:T1500F15U35



Max Depth: 12.600 m / 41.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129668m E: 583245m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

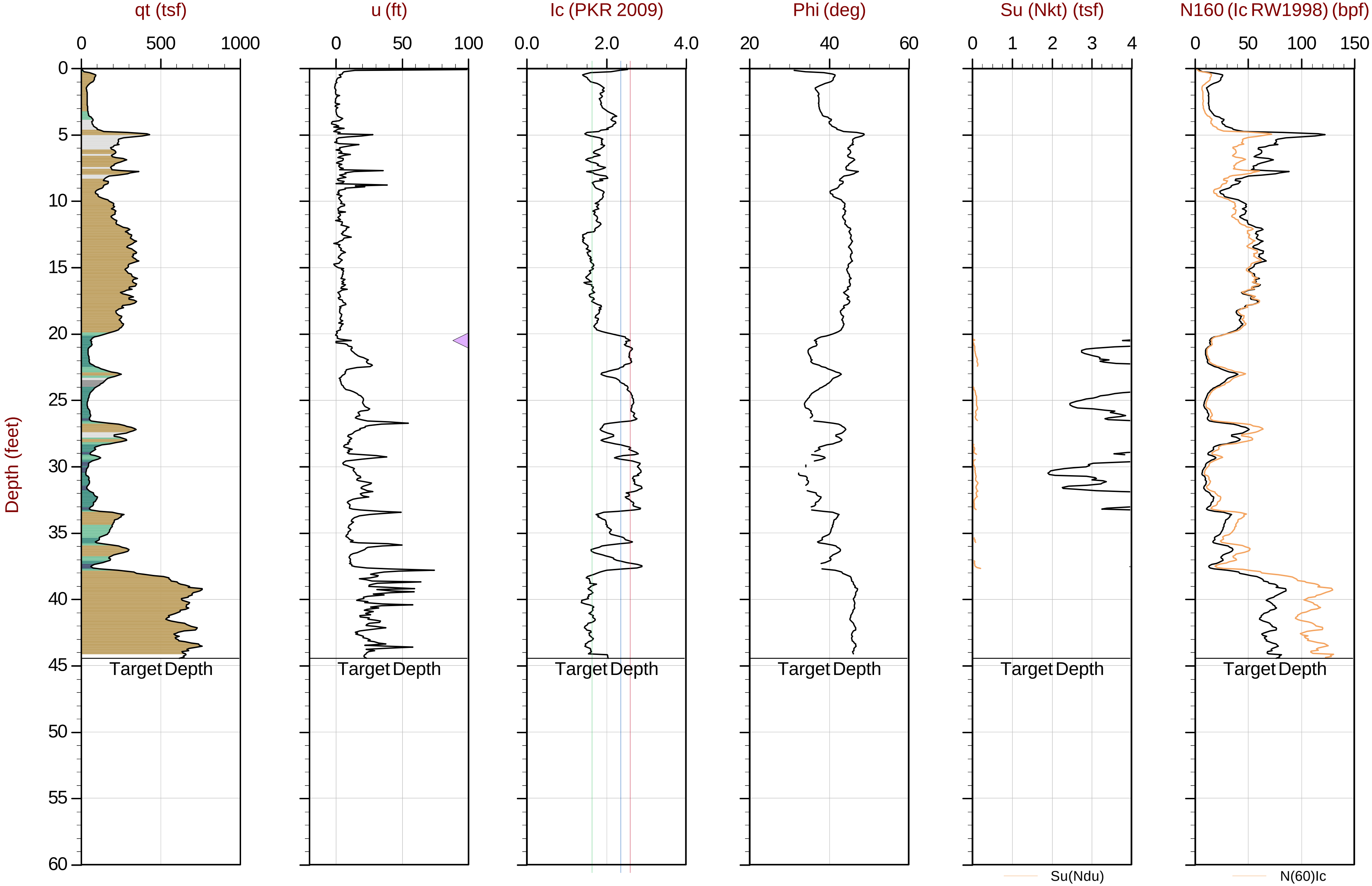
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 12:23
Site: 10857 Linda Vista Drive

Sounding: 1-CPT4
Cone: 1061:T1500F15U35



Max Depth: 13.550 m / 44.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129677m E: 583314m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

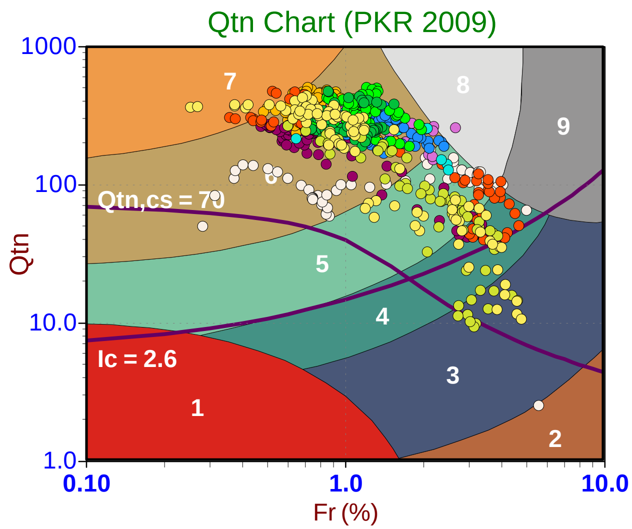
Soil Behavior Type (SBT) Scatter Plots

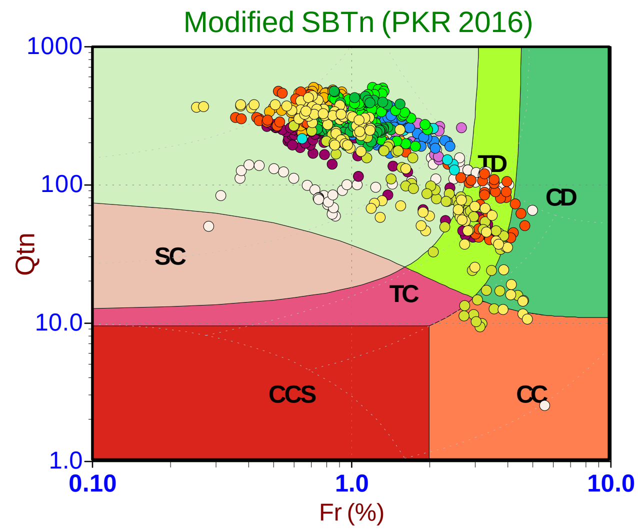
CONETEC

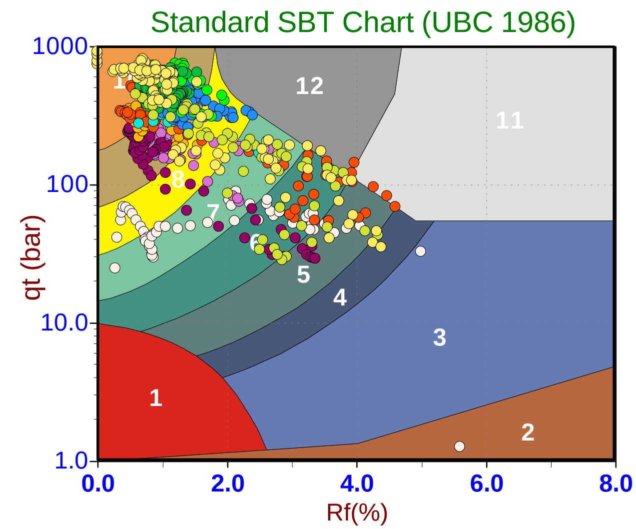
ENGEO

Job No: 24-56-27871
Date: 2024-06-20 08:13
Site: 10857 Linda Vista Drive

Sounding: 1-CPT1
Cone: 1061:T1500F15U35







Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

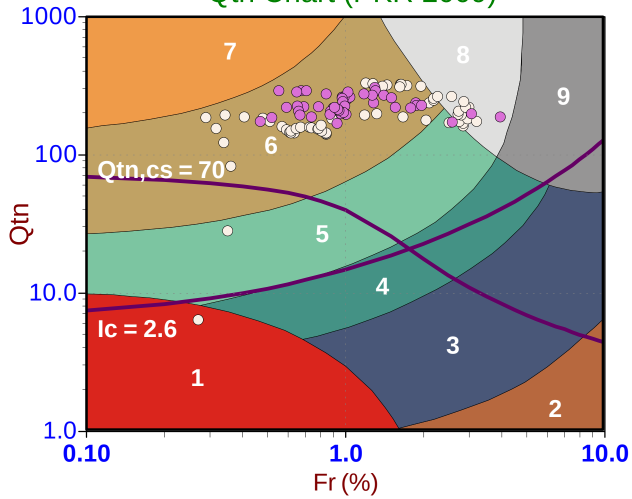
CONETEC

ENGEO

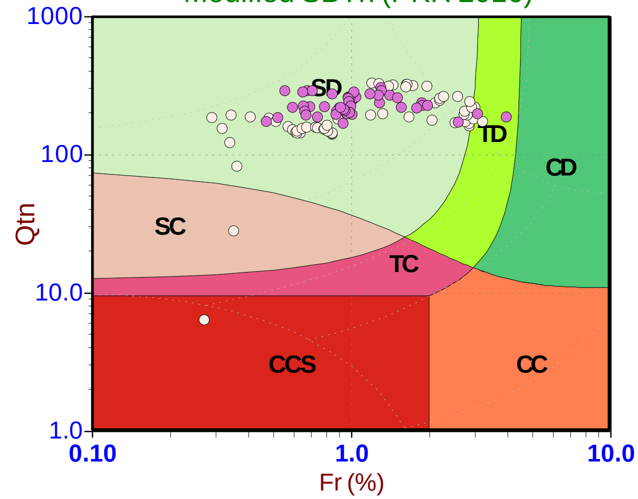
Job No: 24-56-27871
Date: 2024-06-20 11:05
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2
Cone: 1061:T1500F15U35

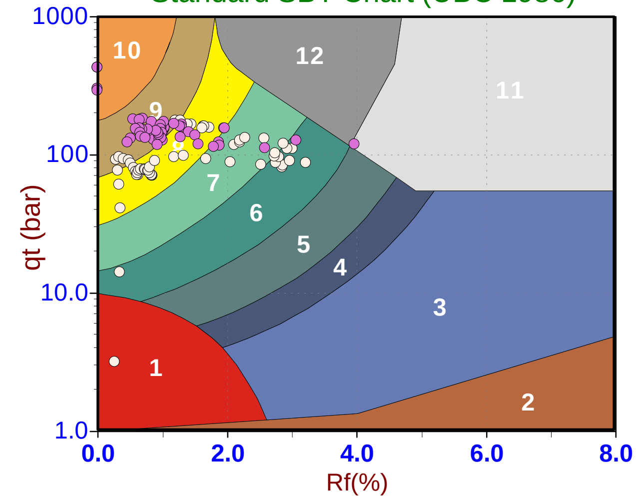
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

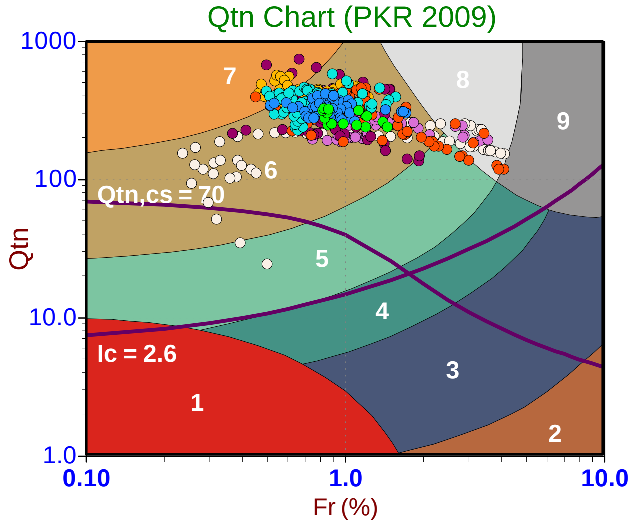
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

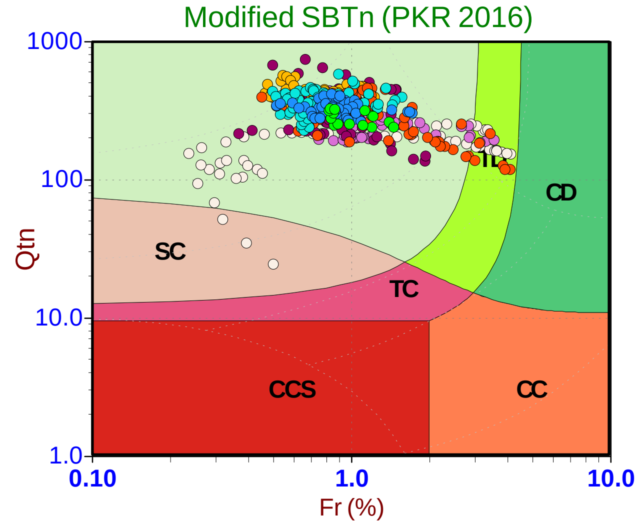
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

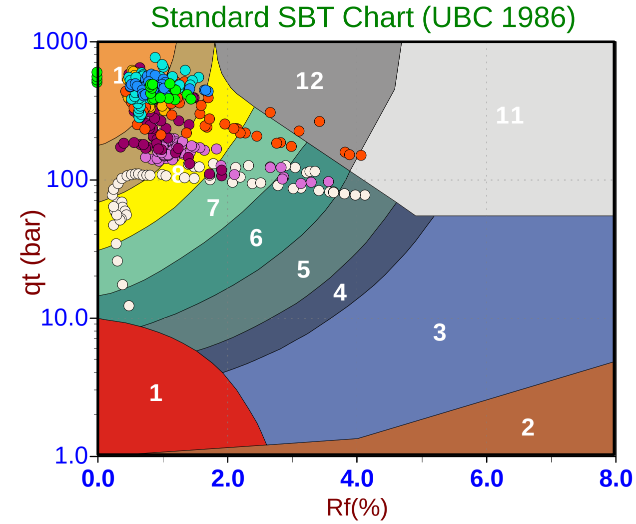
CONETEC

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 11:24
Site: 10857 Linda Vista Drive
Sounding: 1-CPT2B
Cone: 1061:T1500F15U35







Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

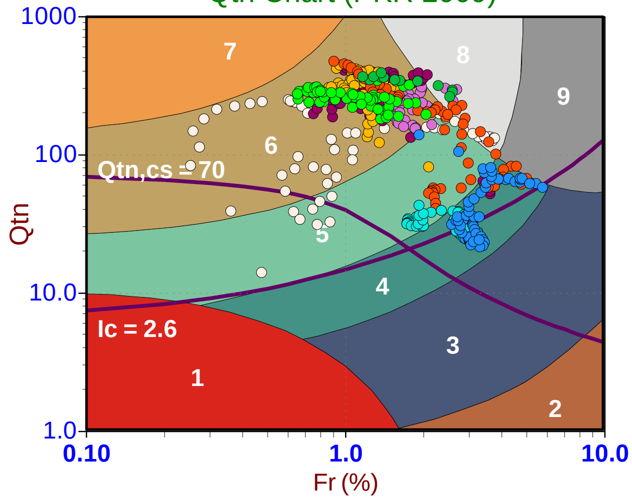
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

ENGEO

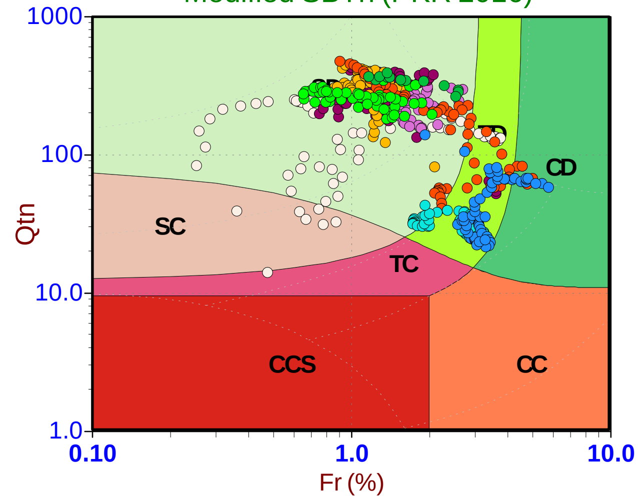
Job No: 24-56-27871
Date: 2024-06-20 13:10
Site: 10857 Linda Vista Drive

Sounding: 1-CPT3
Cone: 1061:T1500F15U35

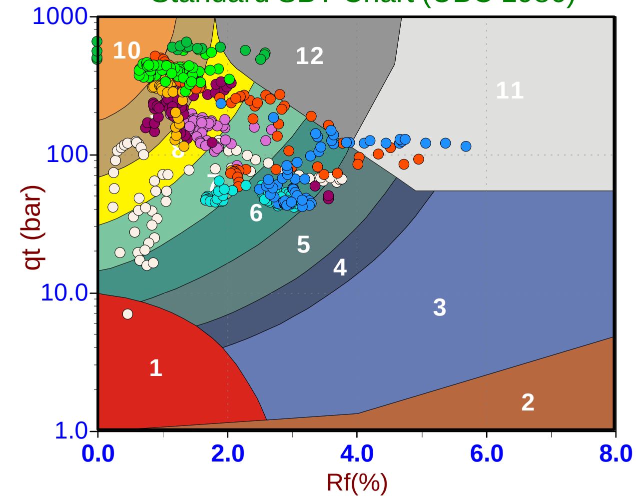
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

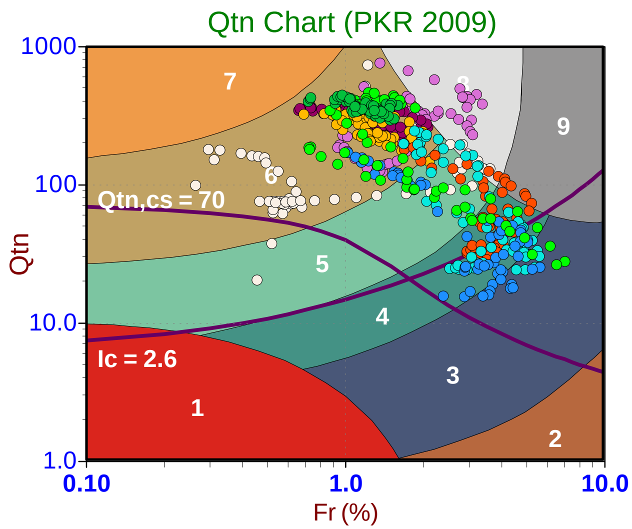
Legend

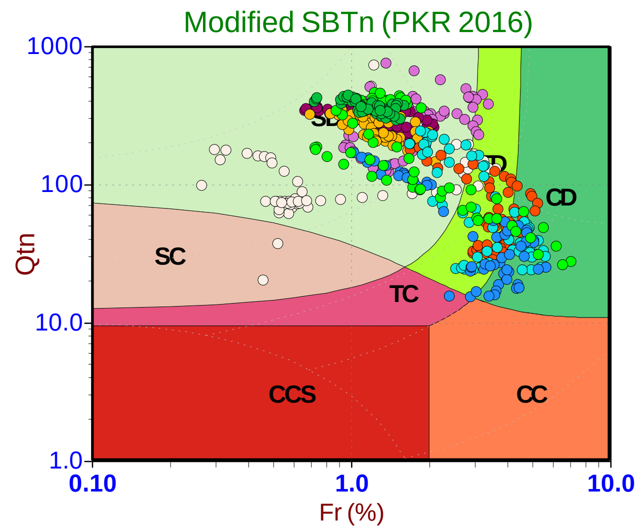
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

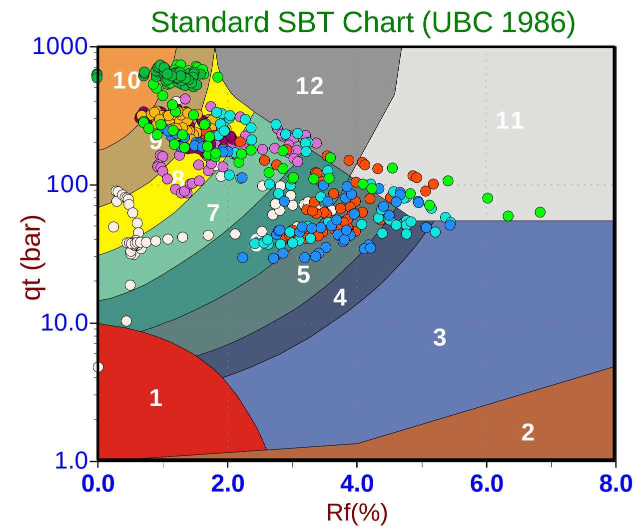
Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

ENGEO	Job No: 24-56-27871 Date: 2024-06-20 12:23 Site: 10857 Linda Vista Drive
	Sounding: 1-CPT4 Cone: 1061:T1500F15U35







Depth Ranges

- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots

Job No:	24-56-27871
Client:	ENGEO, Inc.
Project:	10857 Linda Vista Drive
Start Date:	2024-06-20
End Date:	2024-06-20

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Refer to Notation Number
1-CPT1	24-57-27871_SP01	15	600	53.97			1
1-CPT2B	24-57-27871_CP02B	15	485	36.50			1
1-CPT3	24-57-27871_CP03	15	390	20.51			1
Totals			25 min				

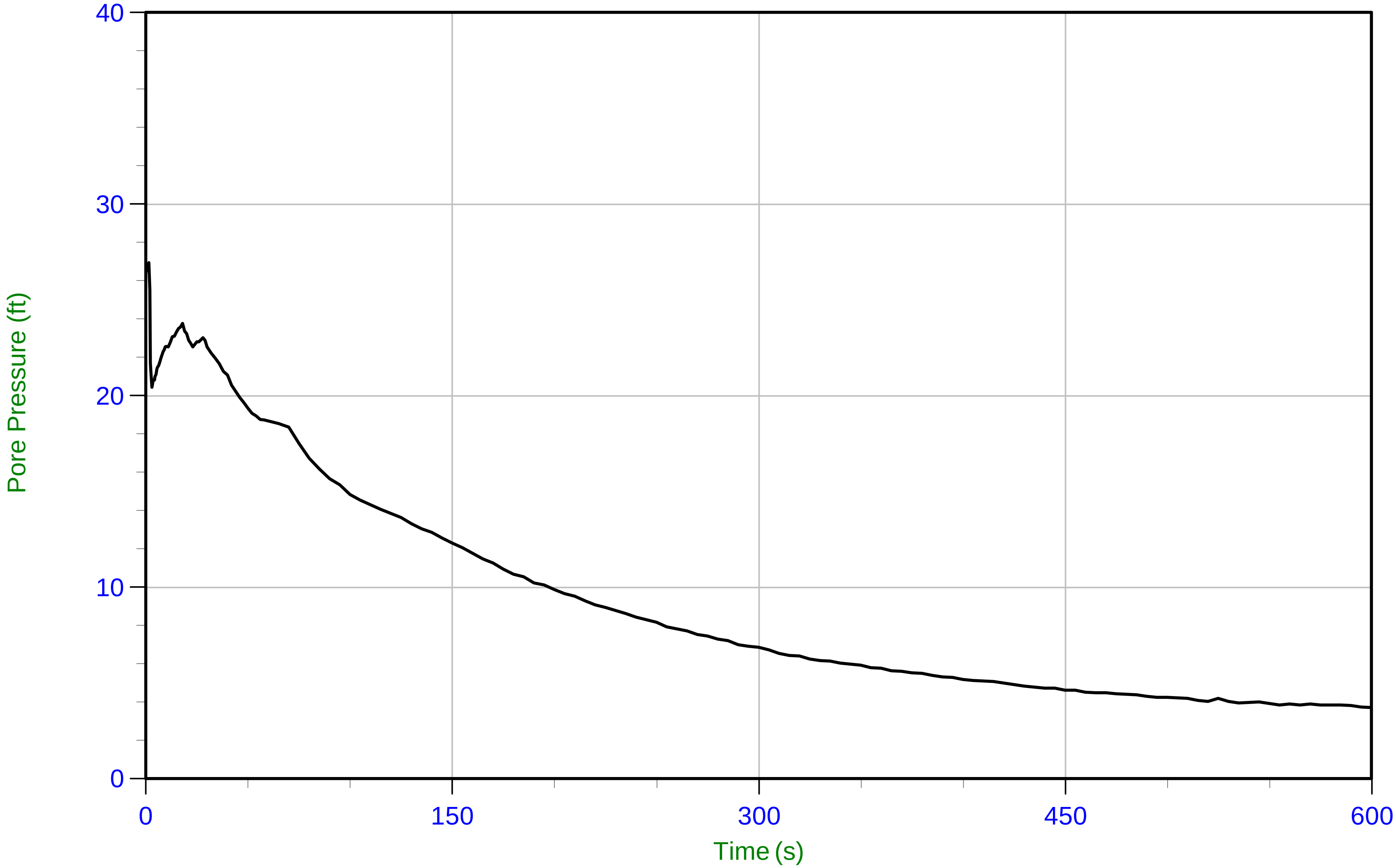
1. Equilibrium pore pressure not achieved.

CONETEC

ENGEO

Job No: 24-57-27871
Date: 06/20/2024 08:13
Site: 10857 Linda Vista Drive

Sounding: 1-CPT1
Cone: 1061:T1500F15U35 Area=15 cm²



Trace Summary:

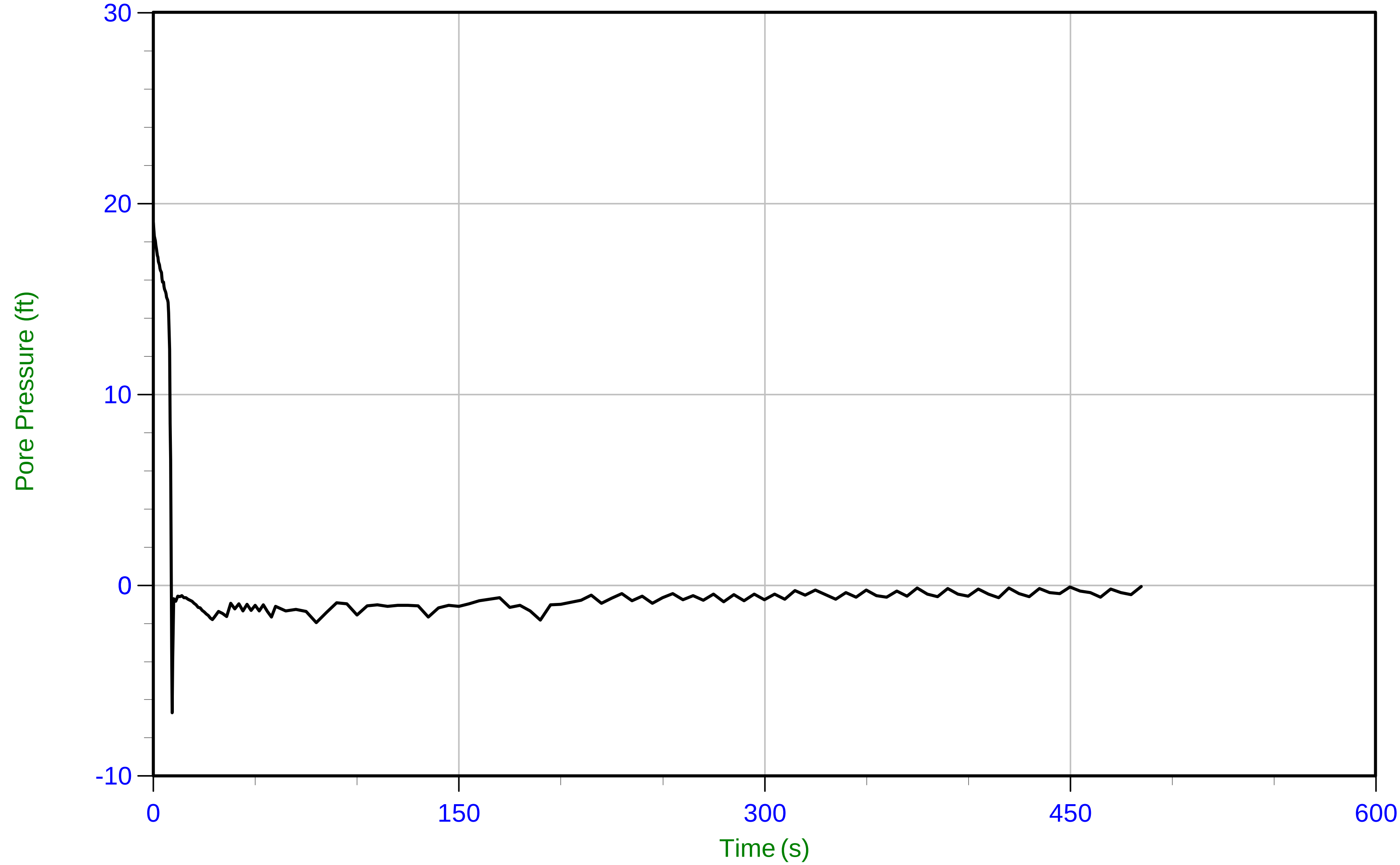
Filename: 24-57-27871_SP01.ppd2
Depth: 16.450 m / 53.969 ft
Duration: 600.0 s

u Min: 3.7 ft
u Max: 26.9 ft
u Final: 3.7 ft

ENGEO

Job No: 24-57-27871
Date: 06/20/2024 11:24
Site: 10857 Linda Vista Drive

Sounding: 1-CPT2B
Cone: 1061:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-57-27871_CP02B.ppd2
Depth: 11.125 m / 36.499 ft
Duration: 485.0 s

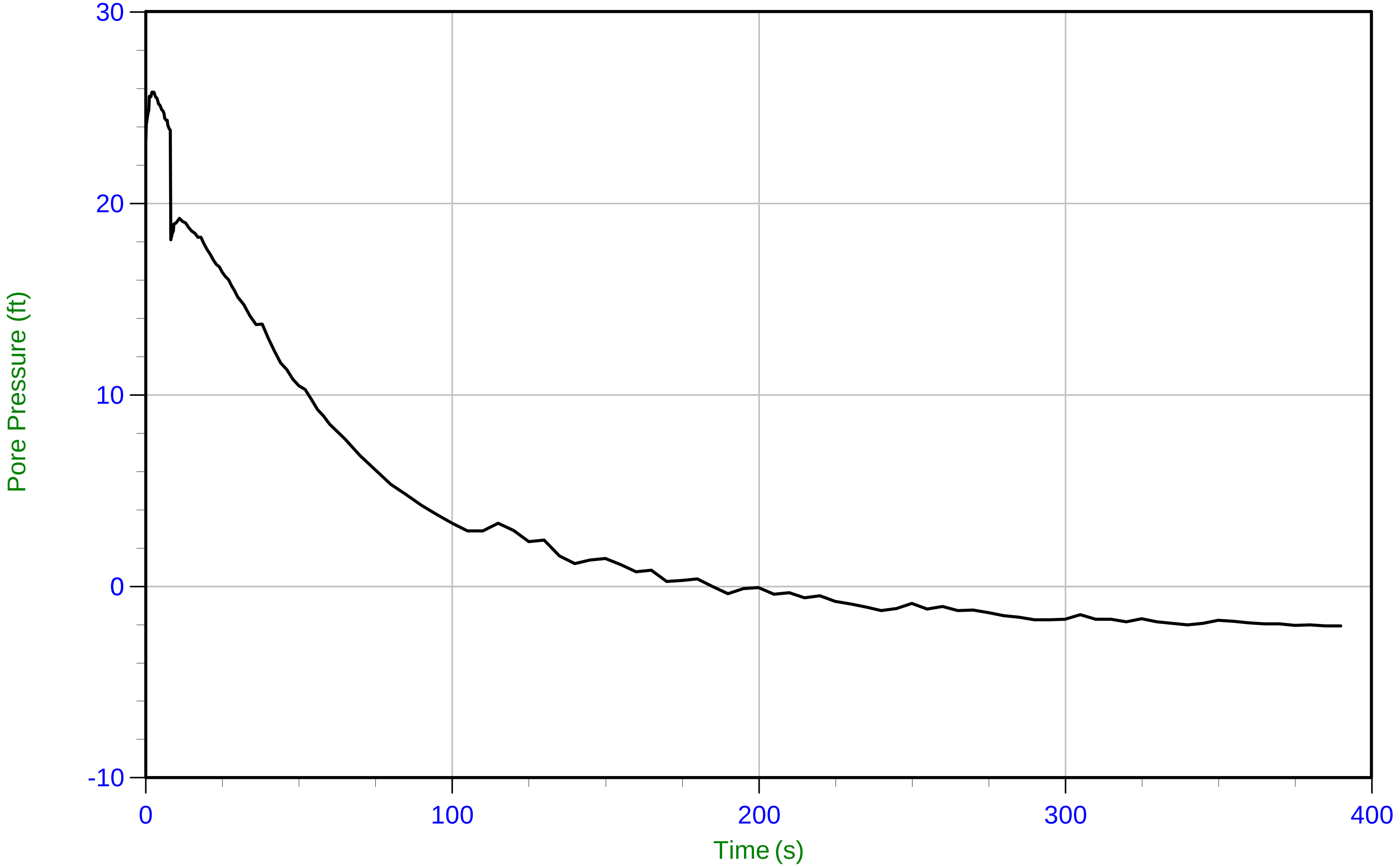
u Min: -6.7 ft
u Max: 19.0 ft
u Final: -0.1 ft

CONETEC

ENGEO

Job No: 24-57-27871
Date: 06/20/2024 12:23
Site: 10857 Linda Vista Drive

Sounding: 1-CPT4
Cone: 1061:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-57-27871_CP04.ppd2
Depth: 6.250 m / 20.505 ft
Duration: 390.0 s

u Min: -2.1 ft
u Max: 25.8 ft
u Final: -2.1 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

ConeTec

Job No:	24-56-27871
Client:	ENGEO
Project:	10857 Linda Vista Drive
Sounding ID:	1-CPT1
Date:	6/20/2024

Seismic Source:	Beam
Seismic Offset (ft):	1.97
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.79	2.13	3.91			
6.07	5.41	6.33	2.42	3.01	804
9.35	8.69	9.29	2.96	3.21	923
12.63	11.97	12.42	3.12	2.73	1143
15.91	15.26	15.60	3.19	2.48	1287
19.19	18.54	18.82	3.22	2.39	1347
22.47	21.82	22.06	3.24	2.10	1543
25.75	25.10	25.31	3.25	1.78	1829
29.04	28.38	28.57	3.26	1.78	1834
32.25	31.59	31.76	3.20	1.64	1949
35.60	34.94	35.09	3.33	1.91	1741
38.88	38.22	38.36	3.27	1.90	1721
42.16	41.50	41.63	3.27	1.72	1900
45.44	44.78	44.90	3.27	1.71	1916
48.72	48.06	48.18	3.27	1.71	1916
52.00	51.35	51.45	3.27	1.76	1863
55.28	54.63	54.72	3.27	2.06	1590
58.40	57.74	57.84	3.11	1.65	1886

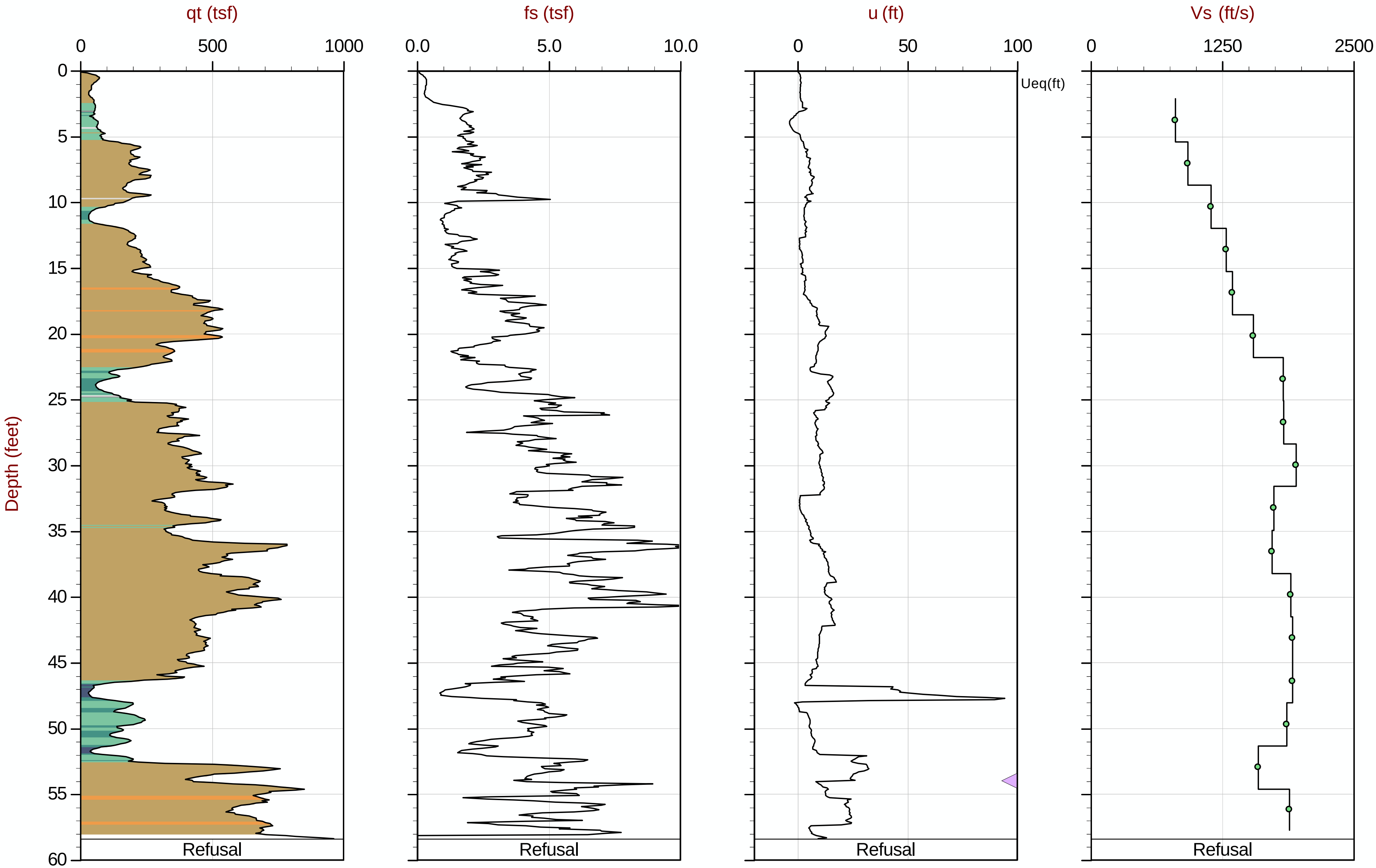
SCPTu Test Plots

ConeTec

ENGEO

Job No: 24-56-27871
Date: 2024-06-20 08:13
Site: 10857 Linda Vista Drive

Sounding: 1-CPT1
Cone: 1061:T1500F15U35



Max Depth: 17.800 m / 58.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27871_SP01.COR
Unit Wt: SBTQtn (PKR2009)

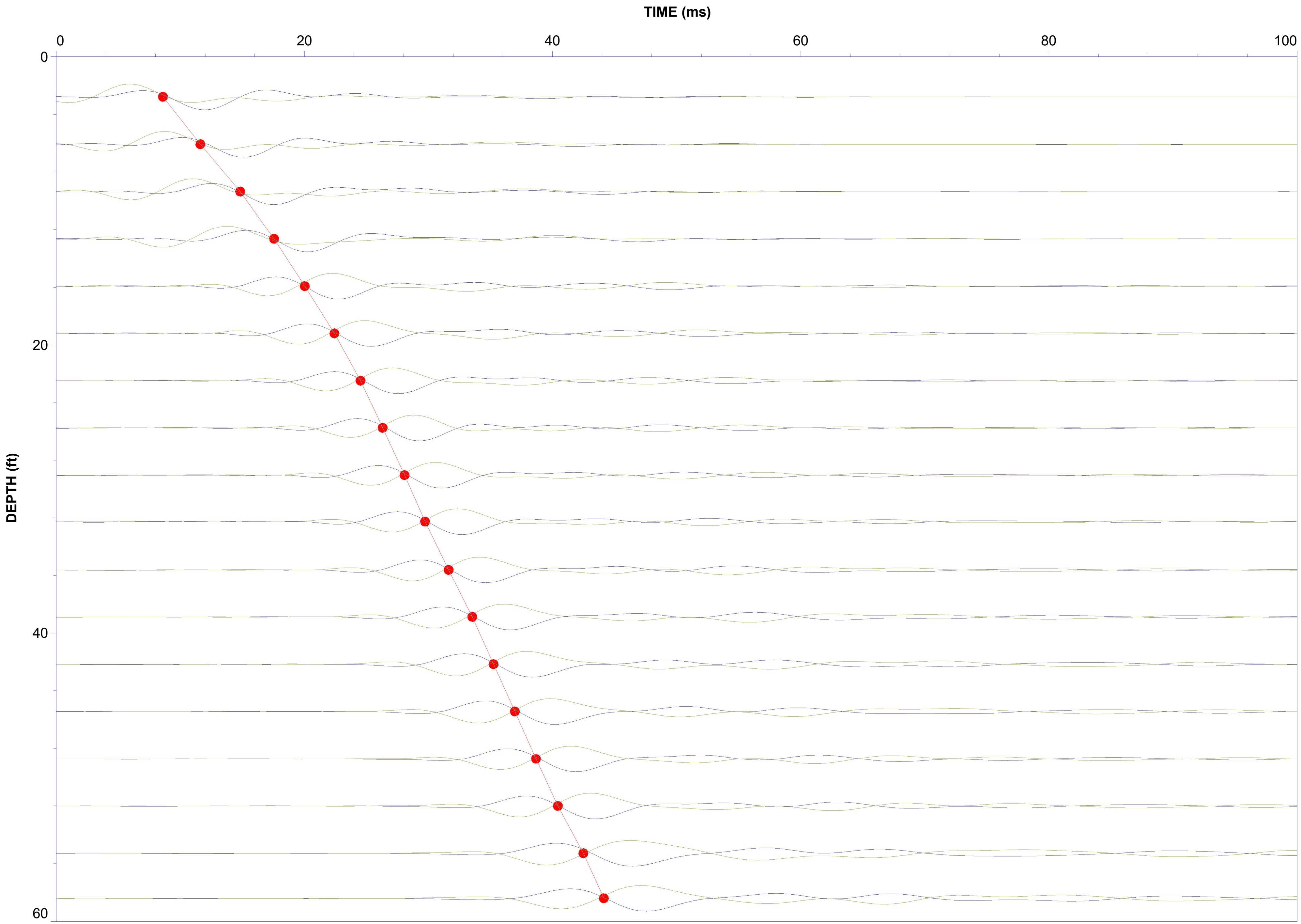
SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4129708m E: 583221m

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

SCPTu Velocity Wave Traces

Job No: 24-56-27871 | Client: ENGEO | Project: 10857 Linda Vista Drive | Analysis: Shear Wave | Sounding: 1-CPT1 | Filter: 10857 Linda Vist
Date: 2024-06-20 | Cone: 1061:T1500F15U35



SUPPORTING DOCUMENTS AND MATERIALS

The documents and materials listed below are included in the report:

- **Methodology Statements**
- **Cone Penetration Digital File Formats**
- **Description of Methods for Calculated CPTu Geotechnical Parameters**
- **Calibration Records**

Methodology Statements

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

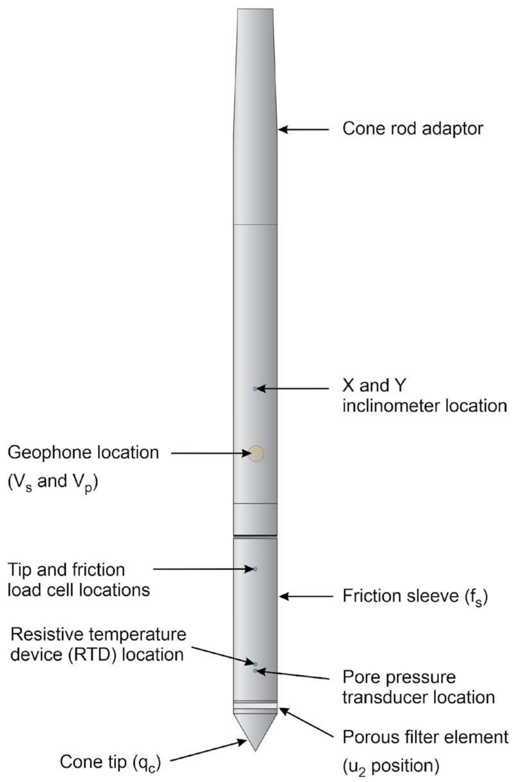


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition system consists of a Windows based computer, signal interface box, and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth encoder that is either portable or integrated into the rig. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson, P.K., 2010. The Soil Behavior Type (SBT) classification chart developed by Robertson, P.K., 2010 is presented in Figure SBT. It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

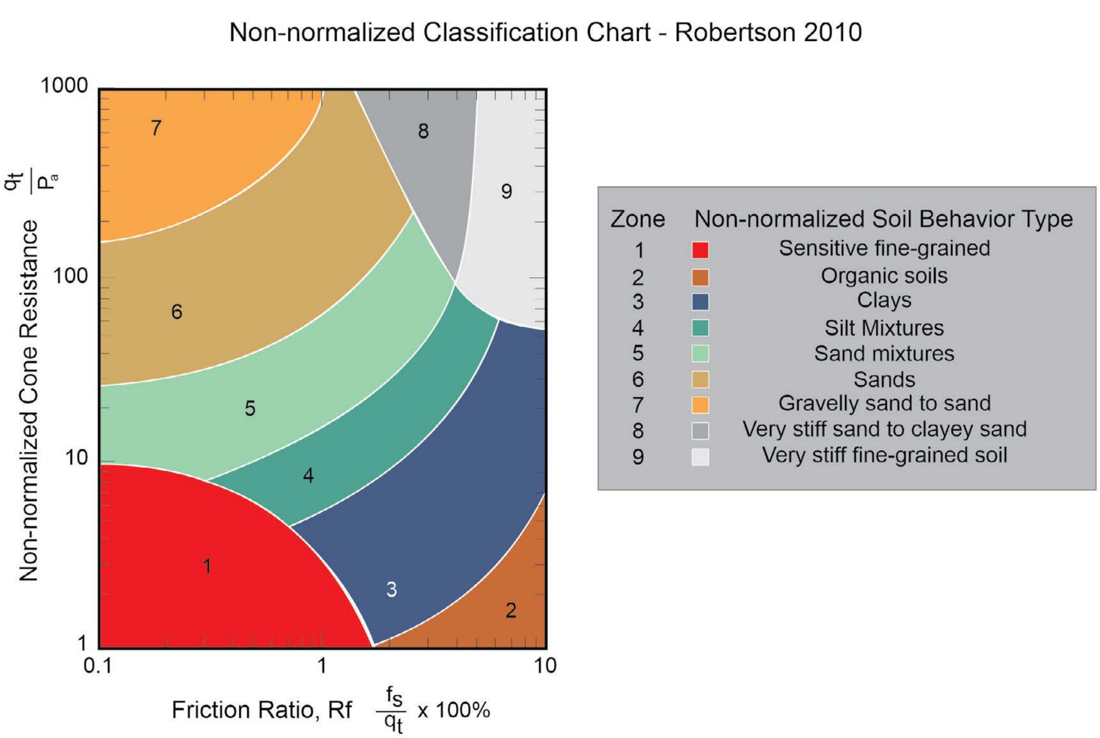


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

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ASTM D5778-20, 2020, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

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Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

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PORE PRESSURE DISSIPATION TEST

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

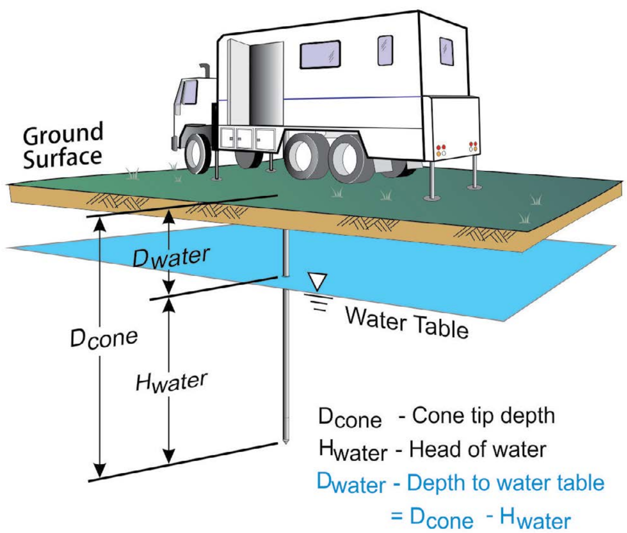

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

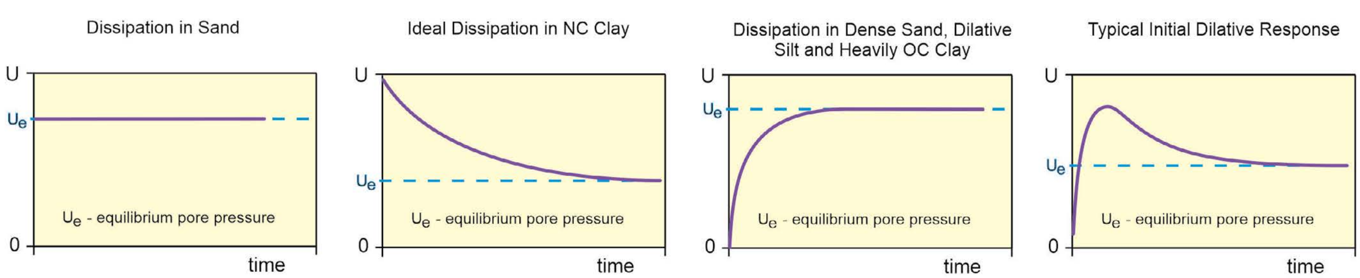

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

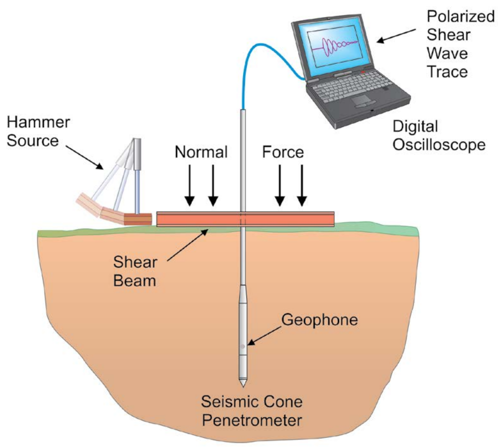

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et al. (1986).

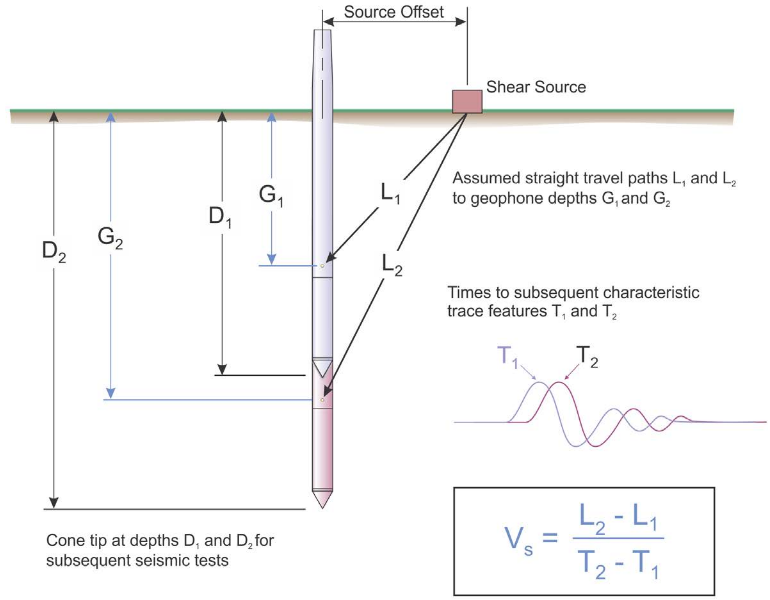

Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

Tabular velocity results and SCPTu plots are presented in the relevant appendix.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet ($\overline{v}_S$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

$$\bar{v}_s = \frac{\sum_{i=1} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_s100 or V_s30.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

REFERENCES

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Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

Cone Penetration Digital File Formats

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 18

Revised February 10, 2023
Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g., 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not performed.

Corrected tip resistance: $q_t = q_c + (1\text{-}a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

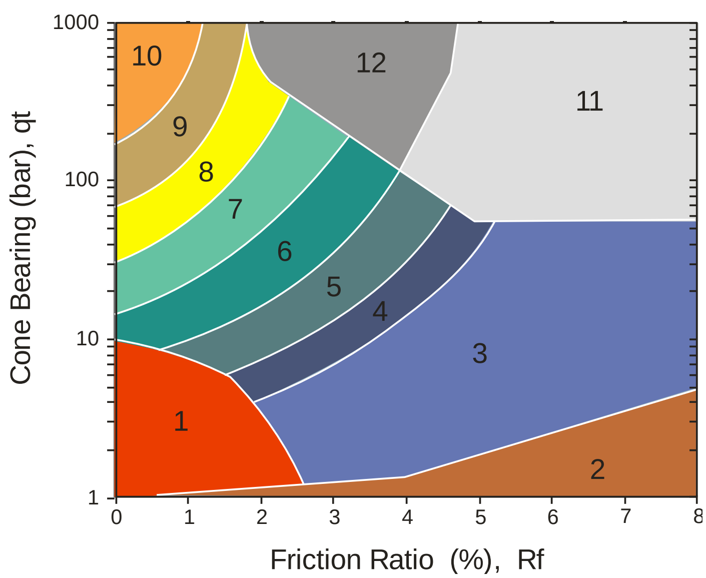
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.



Zone	qt / N	Soil Behavior 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

$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

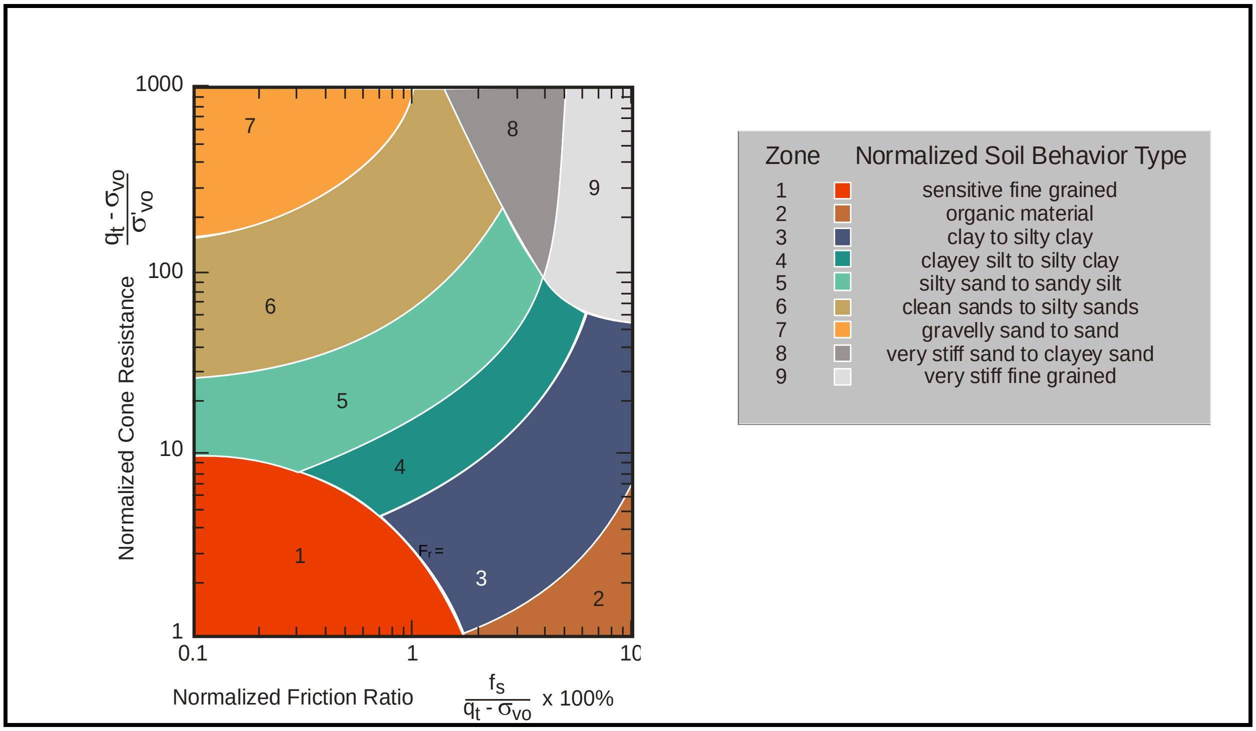


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

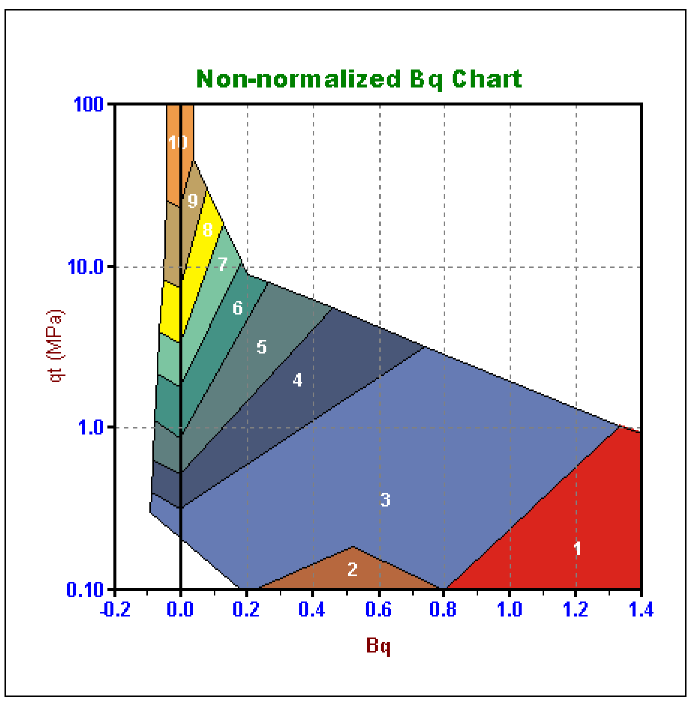


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

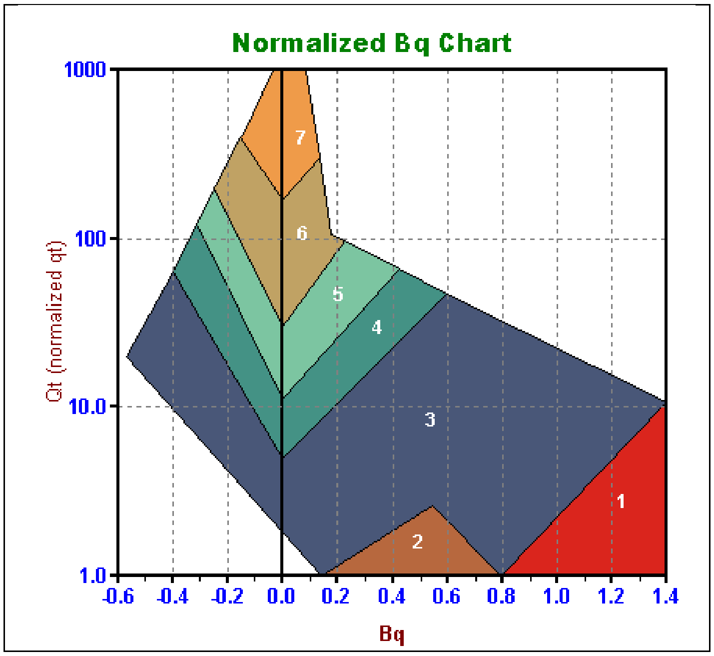


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

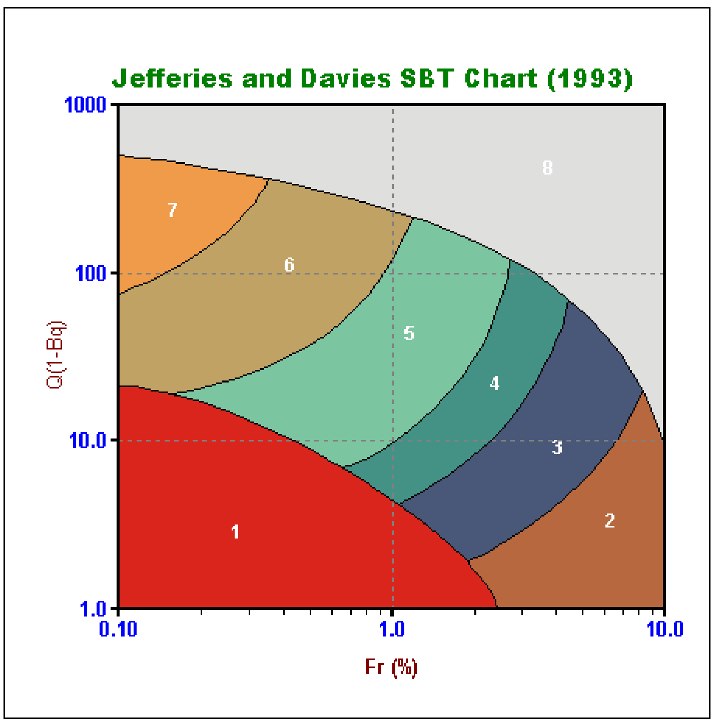


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

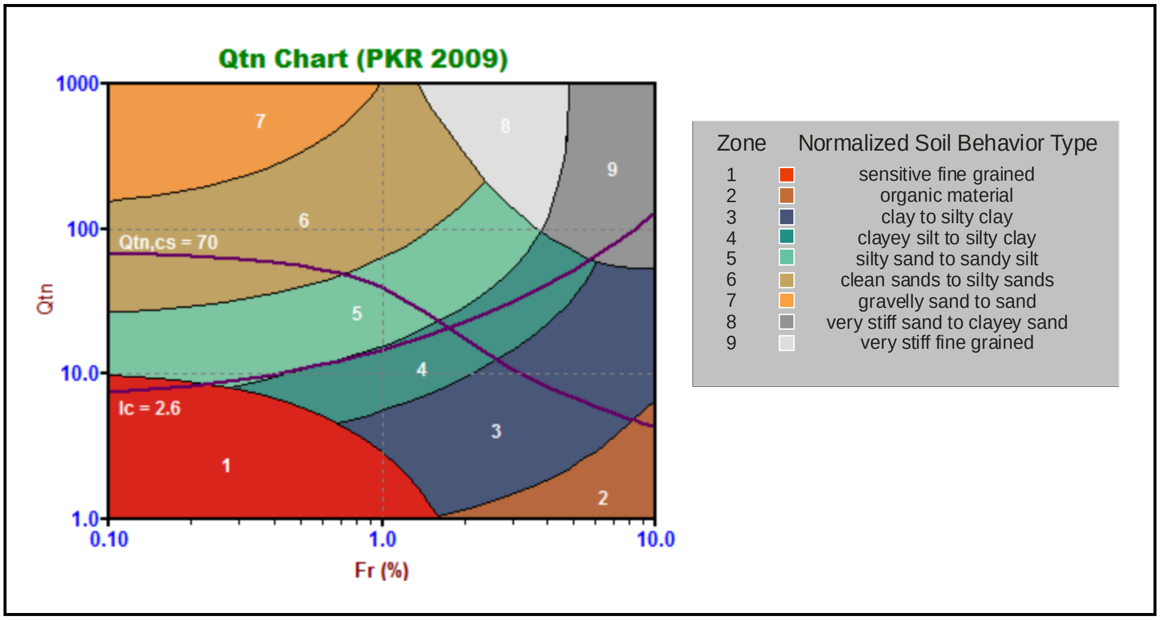


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

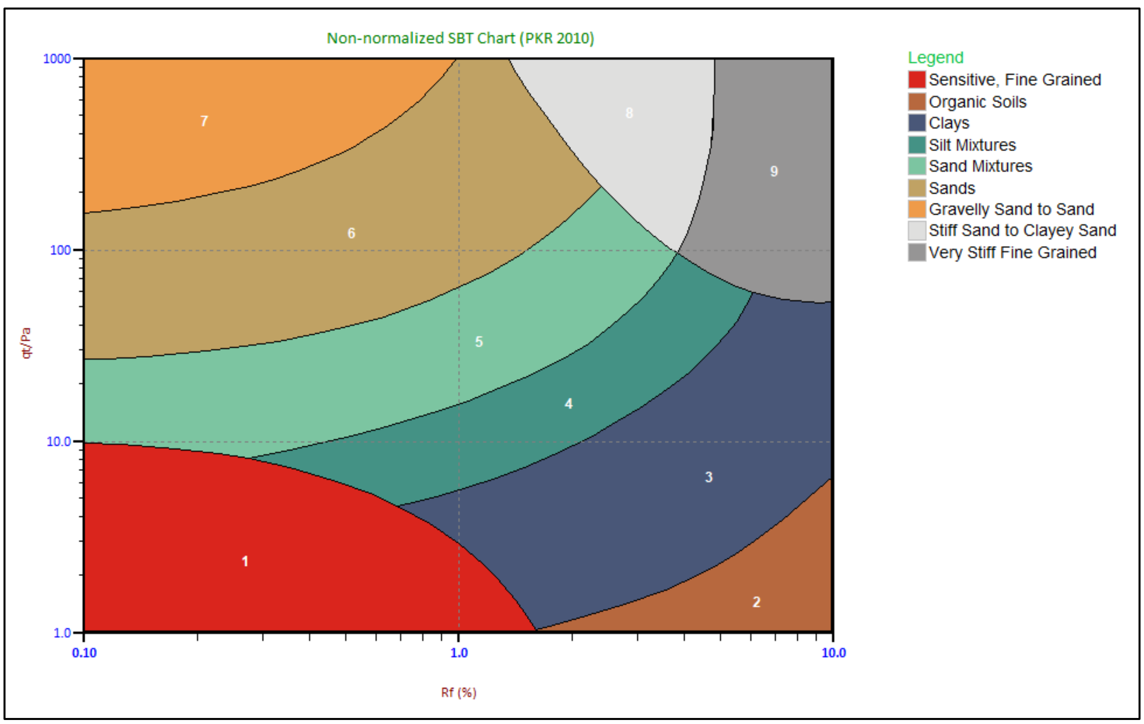


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

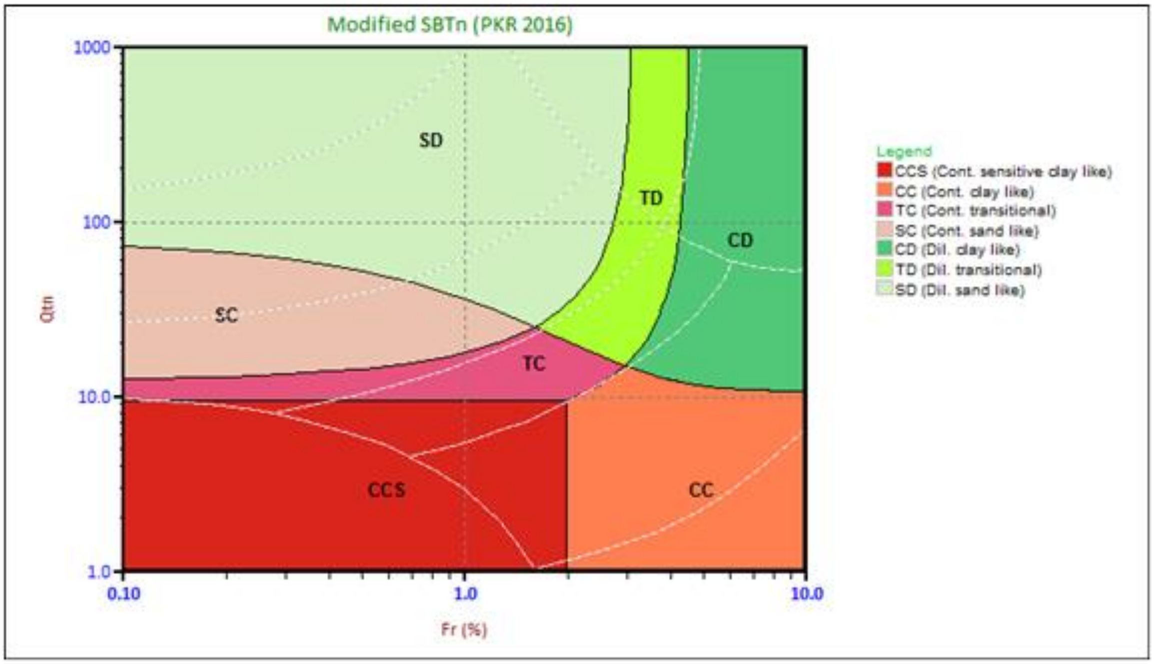


Figure 6. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary, we recommend that the user refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Tables 1and 1a may be included in the output files delivered with this report.

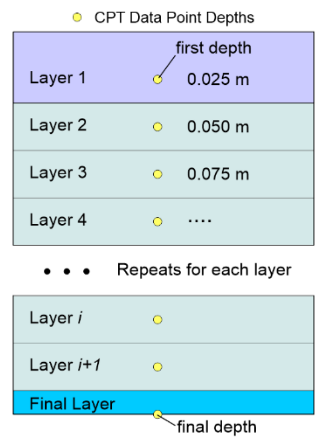
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Reference Notes: CK* - Common Knowledge, U* - Unpublished

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a)\cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s.	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

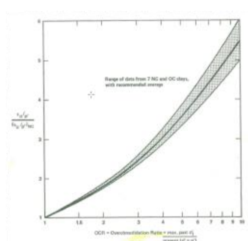
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness 	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0.	$K_o = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically ranging from 1.7 to 2.0)* P_a *is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60} I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5 (1 - I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 - 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ *and* u = *dynamic pore pressure*, u_{eq} = *equilibrium pore pressure*	1, 2, 5
Net q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	36
Q_t or Norm: Qt or Q_{t1}	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5, 15
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-B_q) Q(1-B_q) + 1	Q(1-B_q) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above*	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q_{c1} (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}(0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}(Cq) = C_q * q_t$ (some papers use q_c)	5, 12
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining I_c* *And* *Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting I_c in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c *(PKR 2009) =* $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Q_{tn} (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Q_{tn} (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ *for* $I_c > 3.5$ $FC=0$ *for* $I_c < 1.26$ $FC = 5\%$ *if* $1.64 < I_c < 2.6$ *AND* $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs}$ = 70 line in Figure 4.	$CD = 70 = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5)(1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100\ (Q_{tn} + 10) / (70 + Q_{tn}\ F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 2) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ 3) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60 \cdot (q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25 \cdot (Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma'_v}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $$\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress Δu / σ_v	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su (N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma \left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
$qtNet/G_{max}$	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for "crane walk" accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1,68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young's Modulus, E, at a 25% working load	$E_{25} = \alpha_E\ (qtNet)$ where $\alpha_E = 0.015[10^{(0.55Ic + 1,68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D' Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D'	1) Robertson $M = \alpha_M (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\ [10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D\ (qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}I_c$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\,(1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta = 1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta = 1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\dfrac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\dfrac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\, [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\, [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\, [q_{c1ncs}/1000]^3 + 0.08$	10
Kg or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}{}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
K_g*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g^* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

No.	Reference
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No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
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Calibration Records

Adara Systems Ltd.
201-8327 Eastlake Drive, Burnaby, BC, Canada V5A4W2
Tel: 1-604-273-6013

CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC1061	Model	A15 T1500 F15 U35
Date	2024-05-08		
Technician Performing Calibration	Richard Chen	Signature	
Calibration Approved By	Vishrut Khunt	Signature	

Lab Condition	As Found	As Left		
Lab Temperature	N/A	23˚C		
Lab Humidity	N/A	27%	Reason for Calibration	Repair

Cone Information				
Tip Stress Limit	1500	bar	Tip End Area	15 cm²
Friction Stress Limit	15	bar	Friction Surface Area	225 cm²
Pressure Limit	35	bar	RTD Location	Pressure Carrier
X-Inclinometer Limit	30	degrees	Geophone	X and Z
Y-Inclinometer Limit	30	degrees	Temperature Range	-20°C to 60°C

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	-3.622	-1.064
Sleeve	bar	0.059	-0.023
Pressure	bar	1.235	1.026
X-Inclinometer	degrees	-0.350	-0.013
Y-Inclinometer	degrees	-0.100	0.006
Temperature	˚C	23.630	23.734

Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

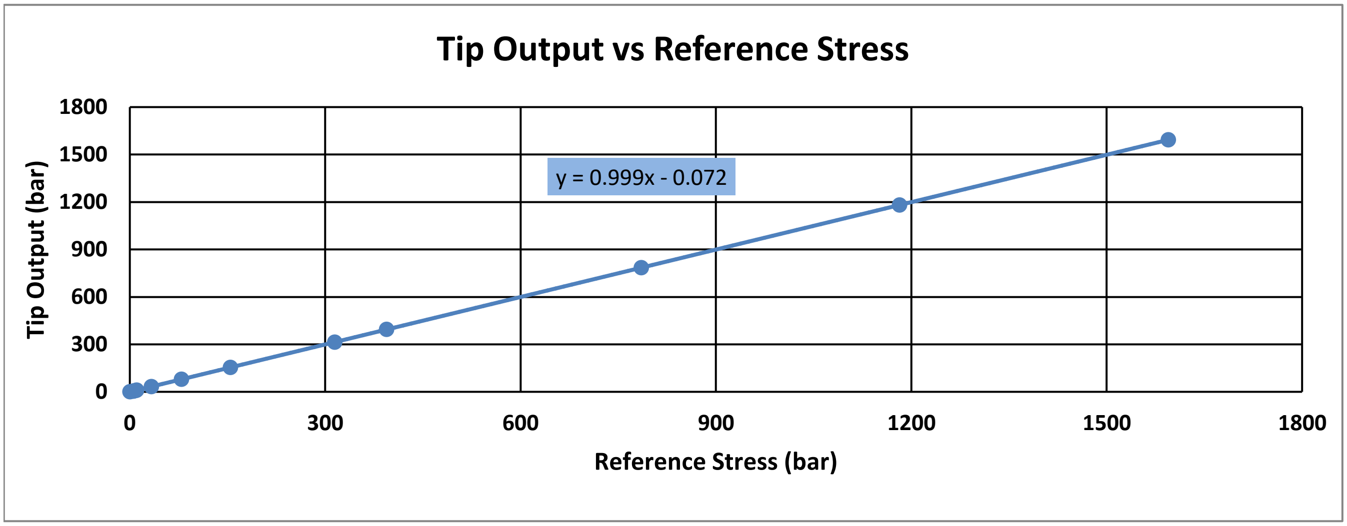
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

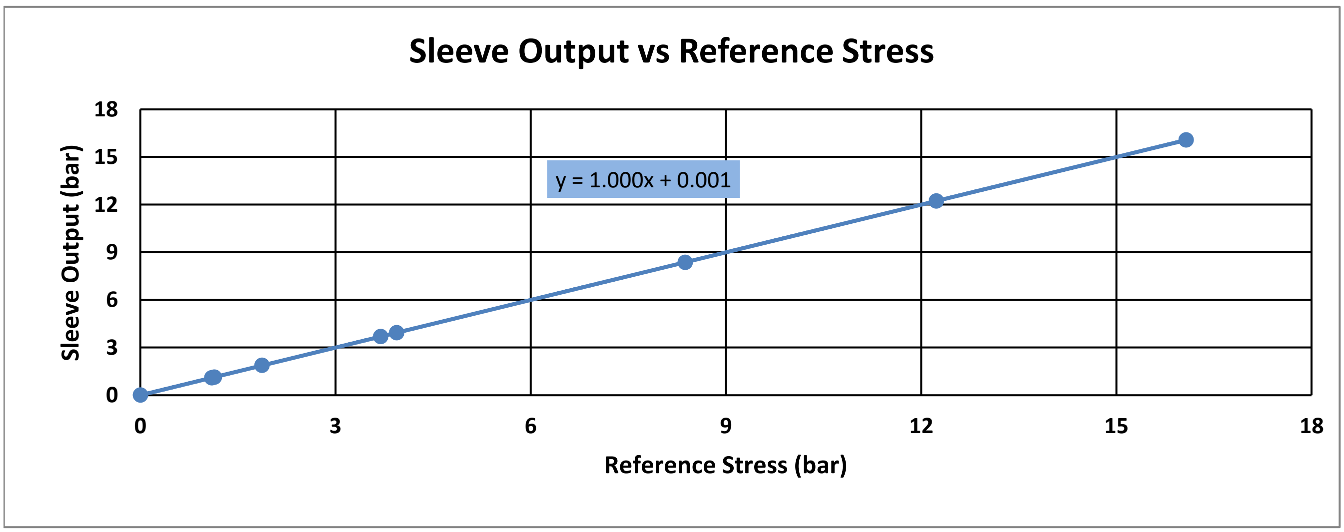
Calibrated with Adara calibration procedure EC_CPTCAL-2.2

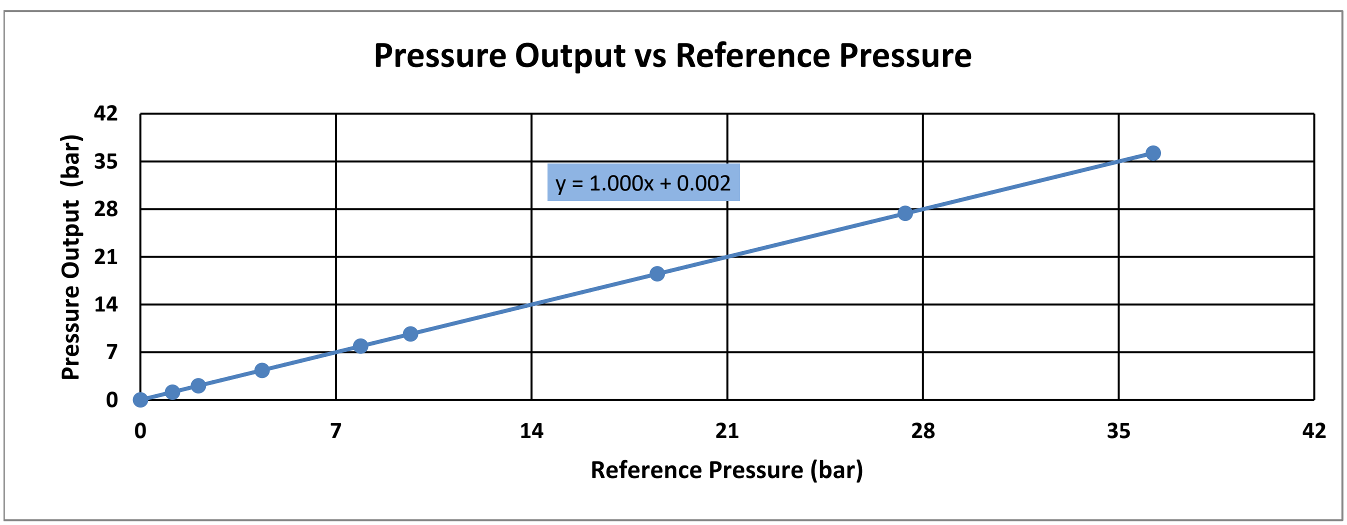
Collective uncertainty of the measurement standards conforms to a test uncertainty ratio (TUR) of 3:1 for tip and sleeve measurement and 4:1 for pressure measurement with a confidence level k=2

Adara Systems Ltd.
201-8327 Eastlake Drive, Burnaby, BC, Canada V5A4W2
Tel: 1-604-273-6013

Cone Output vs Reference Stress/Pressure Plots







Adara Systems Ltd.
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Tel: 1-604-273-6013

Calibration Results

Tip Calibration					
As Found			As Left		
Max. Non Linearity	0.08%	PASS	Max. Non Linearity	0.10%	PASS
Calibration Error	0.07%	PASS	Calibration Error	0.19%	PASS

Sleeve Calibration					
As Found			As Left		
Max. Non Linearity	0.34%	PASS	Max. Non Linearity	0.01%	PASS
Calibration Error	0.35%	PASS	Calibration Error	0.05%	PASS

Pressure Calibration					
As Found			As Left		
Max. Non Linearity	0.03%	PASS	Max. Non Linearity	0.04%	PASS
Calibration Error	0.09%	PASS	Calibration Error	0.06%	PASS

X-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.04%	PASS
Calibration Error	N/A	N/A	Calibration Error	-0.08%	PASS

Y-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.33%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.67%	PASS

Seismic Calibration					
As Found			As Left		
Trigger Delay Error	N/A	N/A	Trigger Delay Error	0.02%	PASS

Temperature Calibration			
Full Scale Error	0.23%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	3.3	22.2	43.4	˚C
Tip	-4.317	-0.915	2.823	bar
Sleeve	-0.061	-0.022	0.021	bar
Pressure	1.048	1.063	1.064	bar
Temperature	3.281	22.037	43.378	˚C

Tip Temperature Coefficient	0.178 bar/˚C	PASS
Sleeve Temperature Coefficient	0.002 bar/˚C	PASS
Pressure Temperature Coefficient	0.000 bar/˚C	PASS

Adara Systems Ltd.
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Tel: 1-604-273-6013

Testing Equipment Details

Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3910	100835	2024-12-12
Tektronix Function Generator	AFG3021B	C030955	100751	2024-10-20
Thermometer	THS-222-555	D23255834	100410	2024-07-11
Thermometer	THS-222-555	D23255829	100410	2024-07-11
Thermometer	THS-222-555	D20345575	100565	2024-07-14

Adara Error Definitions

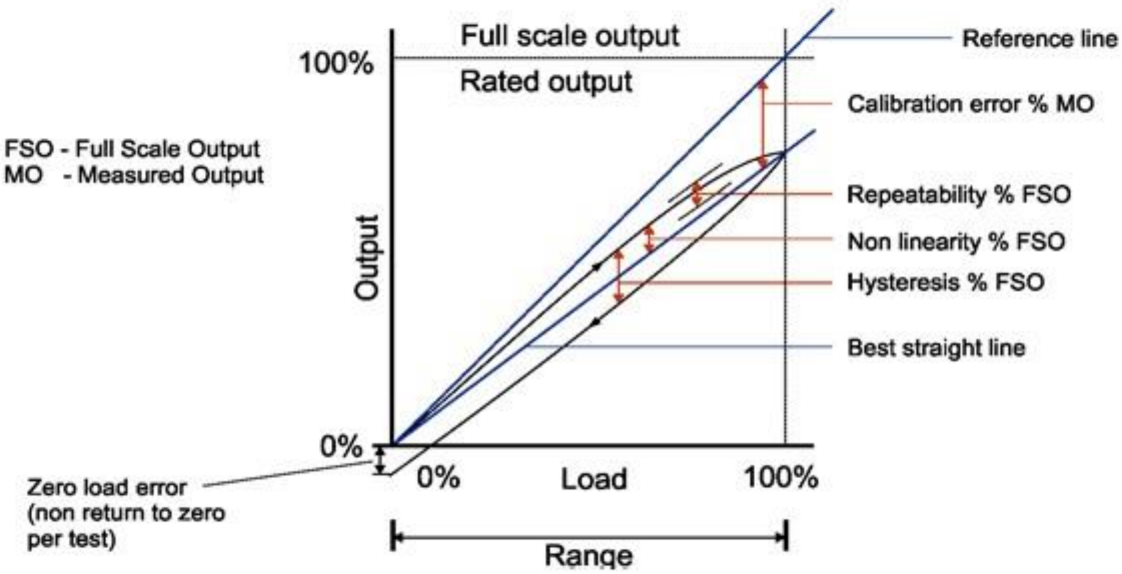


Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

Actual Sensitivity	The slope of the best fit line through all data points starting at zero load.
Slope Error	The error in the best fit line compared to the ideal linear calibration in % . Slope Error = (Best Fit Slope - Ideal Slope) / Ideal Slope
Maximum Non Linearity	This value represents the maximum error (absolute value) relative to the best fit line considering each calibration point starting at loads greater than approximately 10% of FSO. The reported errors are a percent error of FSO. Adara's Pass/Fail criteria is 0.5% of FSO (ASTM is 0.5% of FSO at loads > 20% FSO).
Calibration Error	This value represents the maximum error (absolute value) in the recorded load value as compared to the actual load value for each calibration point for loads greater than approximately 10% of FSO. Adara's Pass/Fail criteria for the tip and sleeve is 0.5% of MO and 1.0% of MO for the pore pressure (ASTM for the tip and sleeve is 1.5% and 1.0% of MO respectively at loads greater than 20% of FSO)

Temperature Check Passing Criteria

Tip Temperature Coefficient	<0.200 bar/˚C
Sleeve Temperature Coefficient	<0.005 bar/˚C
Pressure Temperature Coefficient	<0.0196 bar/˚C

ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

Non Linearity	Tip	≤ +0.5% of FSO
	Sleeve	≤ +1.0% of FSO
Calibration Error	Tip	≤ +1.5% of MO at loads > 20% FSO
	Sleeve	≤ +1.0% of MO at loads > 20% FSO

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

Non Linearity	Pore Pressure	≤ +1.0% of FSO
Calibration Error	Pore Pressure	not specified

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

Class 1	Cone Resistance	35 kPa or 5%
	Sleeve Friction	5 kPa or 10%
	Pore Pressure	10 kPa or 2%
Class 2	Cone Resistance	100 kPa or 5%
	Sleeve Friction	15 kPa or 15%
	Pore Pressure	25 kPa or 3%

Note: ISO Compliance is based on low end calibration only.

References

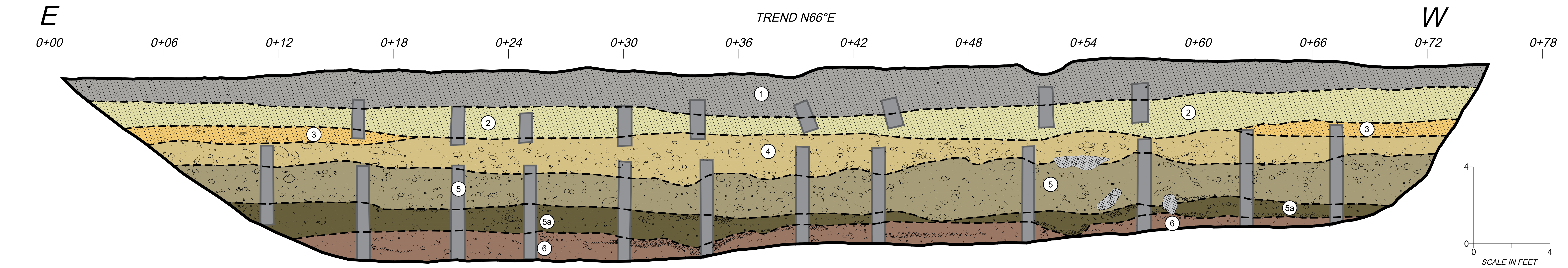
[1] ASTM D5778-20. "Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils". ASTM, West Conshohocken, PA, USA.

[2] ISO 22476-1:2012. "Geotechnical investigation and testing - Field Testing - Part 1: Electrical cone and piezocone penetration test". ISO, Geneva, Switzerland.

ASTM D7400-08. "Standard Test Methods for Downhole Seismic Testing". ASTM, West Conshohocken, PA, USA.

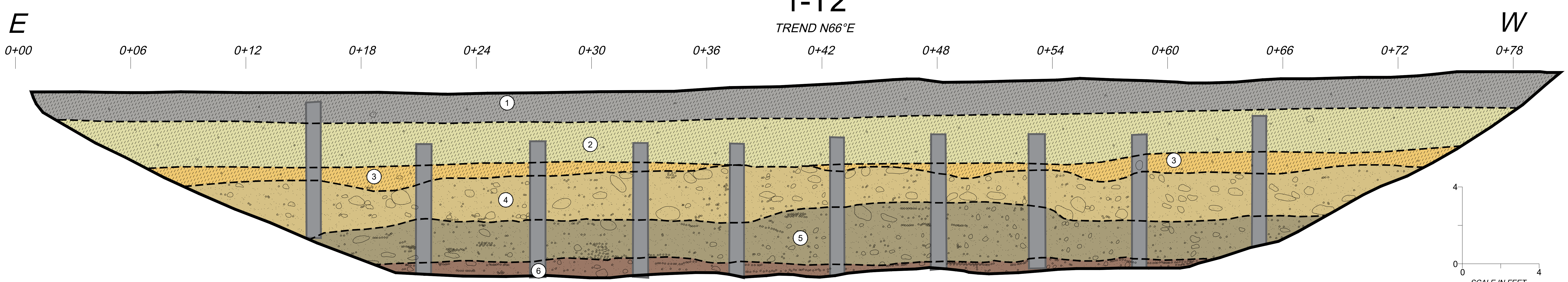
1-T1

TREND N66°E



1-T2

TREND N66°E



1-T3

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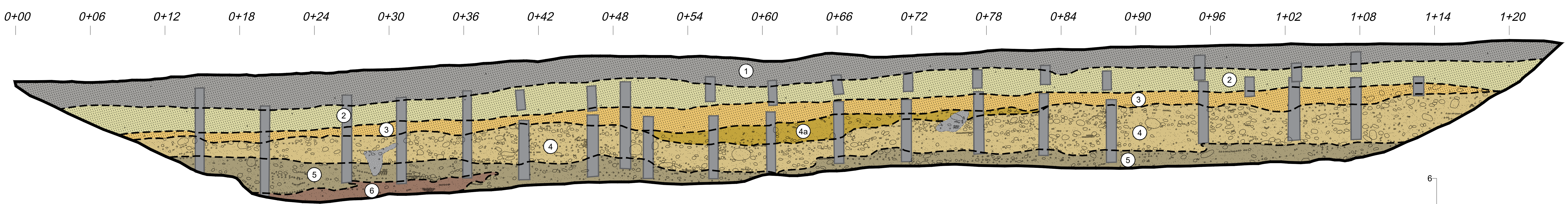
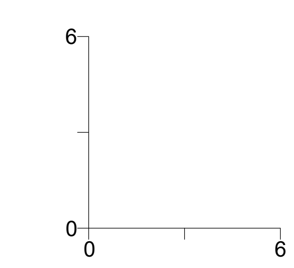




EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Quaternary Surface Deposits

1 **A** - CLAYEY SAND with GRAVEL (SC), light yellowish brown (10YR6/4), dense, moist, fine- to medium-grained, clay is lean (~15%), gravel is subangular to subrounded and fine to coarse (~10%)

2 **ABt** - CLAYEY SAND with GRAVEL (SC), light yellowish brown (10YR6/4) to reddish brown (2.5YR4/4), dense, moist, fine- to medium-grained, clay is lean (~15%), gravel is subangular to subrounded and fine to coarse (~15%)

3 **Bt** - CLAYEY SAND with GRAVEL (SC), light yellowish brown (10YR6/4) to reddish brown (2.5YR4/4), dense, moist, fine- to medium-grained, clay is lean (~20%), gravel is subangular to subrounded and medium to coarse with some cobbles (~20%)

4 **Ct**- POORLY GRADED SAND with GRAVEL and CLAY (SP-SC), dark grayish brown (2.5Y3/2), dense, moist, medium- to coarse-grained sand, clay is lean (~10%), gravel is subrounded and fine to cobble size (~20 - 30%), somewhat well developed clay lenses on the gravels, zones of silty sand deposits, some zones of massive deposition, gravels are mostly Franciscan or Santa Clara Formation rock

4a Fewer gravels (~15%) and increased medium-grained sand

5 **Ct2**- POORLY GRADED SAND with GRAVEL and CLAY (SP-SC), dark brown (7.5YR7/2), dense, moist, medium- to coarse-grained sand, clay is lean (~10%), gravel is subrounded and fine to cobble size (~20 - 30%), well developed clay lenses on the gravels, zones of silty sand deposits, gravels are mostly Franciscan or Santa Clara Formation rock

5a Clay films on gravel clasts very well developed

6 **Ct3** - POORLY GRADED SAND with GRAVEL and CLAY (SP-SC), dark brown (7.5YR7/2), dense, moist, medium- to coarse-grained sand, clay is lean (~10%), gravel is subrounded and fine to cobble size (~15-20%), well developed clay lenses on the gravels, deposits are thin and more regular, gravels are mostly Franciscan or Santa Clara Formation rock

Soil Bedding Contact

Trench Profile

High Fines Concentration Zone

Gravels and Cobbles

Shoring

Zones of Higher Fines Concentration and Clay Films. Secondary Feature Related to Soil Development

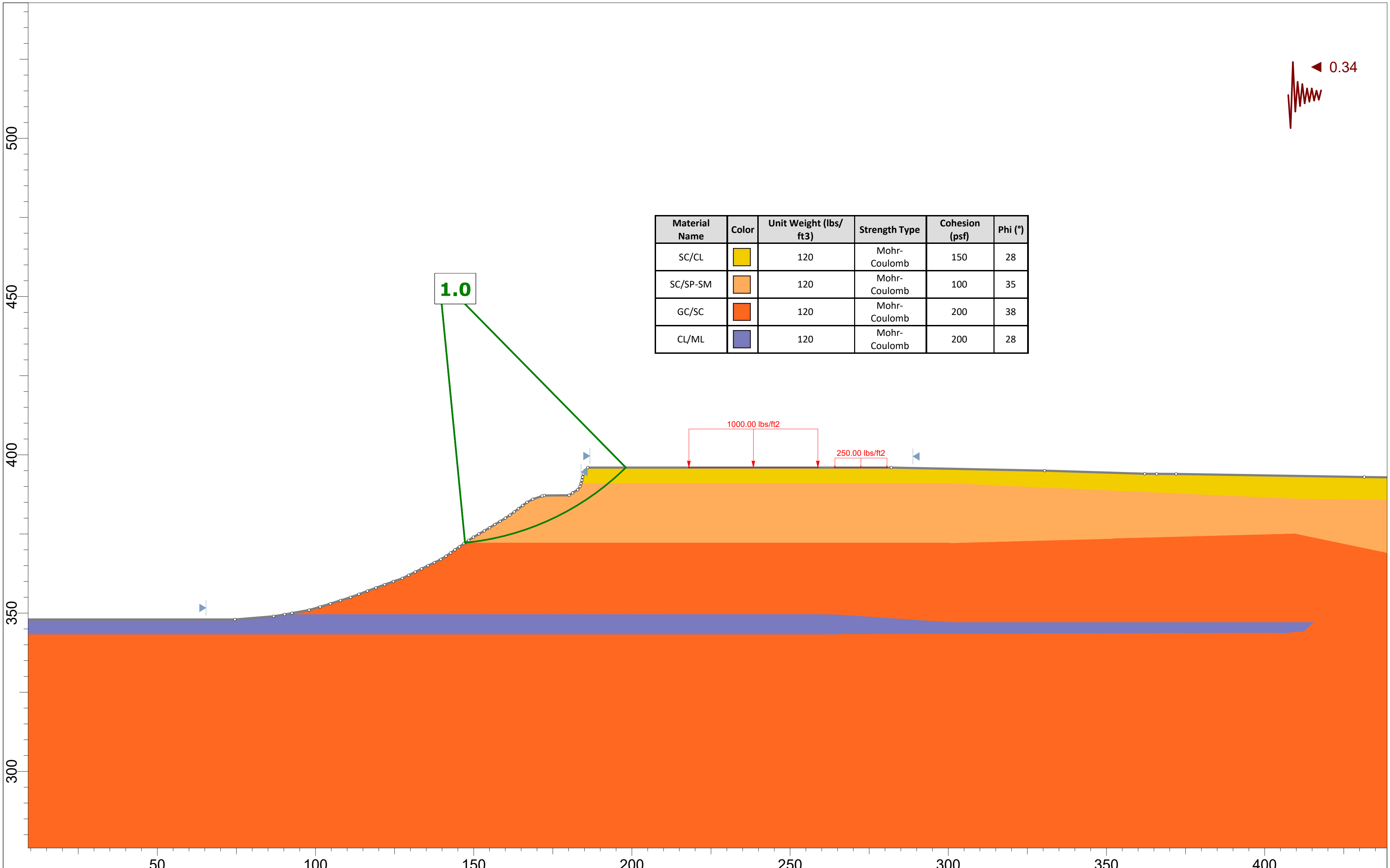
ENGEO Expect Excellence	FAULT TRENCH LOGS LINDA VISTA FAULT EVALUATION CUPERTINO, CALIFORNIA	PROJECT NO.: 25712.000.001		FIGURE NO. 7
		SCALE: AS SHOWN		
		DRAWN BY: NI	CHECKED BY: JBR	

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_24000 to 25999\25712\25712000001\Geotech\Linda Vista Fault Evaluation\CAD\25712000001_Trench Figures_.dwg SAVE DATE: 9/16/2024 10:05:59 AM SAVED BY: nclough

APPENDIX D

SLOPE STABILITY ANALYSIS

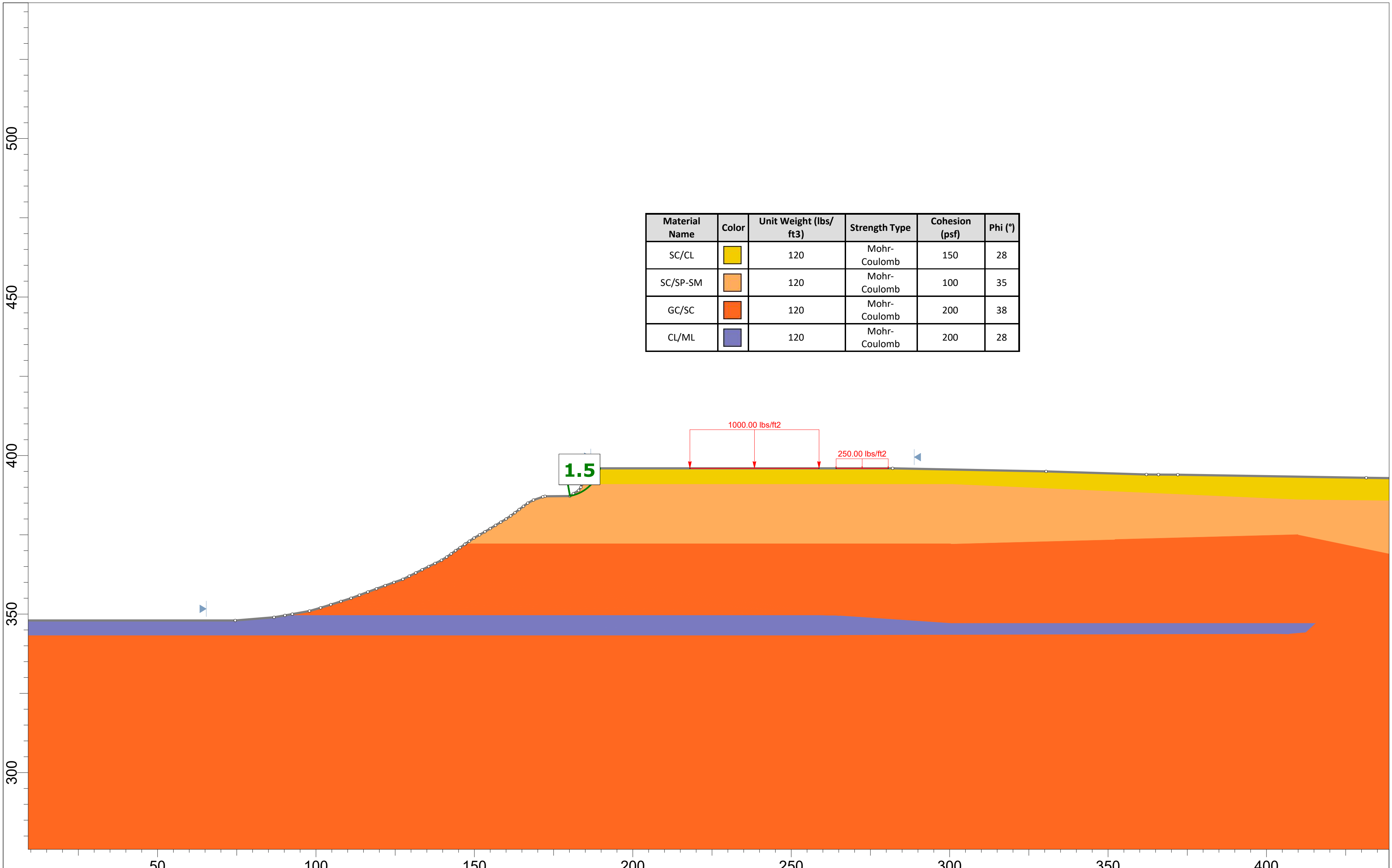


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
SC/CL		120	Mohr-Coulomb	150	28
SC/SP-SM		120	Mohr-Coulomb	100	35
GC/SC		120	Mohr-Coulomb	200	38
CL/ML		120	Mohr-Coulomb	200	28

ENGEO
Expect Excellence

SLIDEINTERPRET 9.035

Project	10857 Linda Vista Drive, Cupertino, CA		
Scale 1:500	*Drawn By* LEBL	*Project No.* 25712.000.001	
Date 4/30/2025	Condition Seismic - Circular		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
SC/CL	Yellow	120	Mohr-Coulomb	150	28
SC/SP-SM	Light orange	120	Mohr-Coulomb	100	35
GC/SC	Orange	120	Mohr-Coulomb	200	38
CL/ML	Purple	120	Mohr-Coulomb	200	28

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SLIDEINTERPRET 9.035

Project	10857 Linda Vista Drive, Cupertino, CA
Scale	1:500
Drawn By	LEBL
Project No.	25712.000.001
Date	4/30/2025
Condition	Static - Circular

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