APPENDIX E

Geotechnical Report

AVENUES SILICON VALLEY
SAN JOSE, CALIFORNIA

DESIGN-LEVEL GEOTECHNICAL REPORT

SUBMITTED TO
Mr. Thomas M. Gannon
Avenues World Holdings LLC
11 Madison Square North, 17th Floor
New York, NY 10010

PREPARED BY
ENGEO Incorporated

May 14, 2019

PROJECT NO.
15929.000.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
15929.000.000

May 14, 2019

Mr. Thomas M. Gannon
Avenues World Holdings LLC
11 Madison Square North, 17th Floor
New York, NY 10010

Subject: Avenues Silicon Valley
San Jose, California

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

Dear Mr. Gannon:

We are pleased to present this geotechnical report for the planned Avenues Silicon Valley project located in San Jose, California. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Based on the results of our exploration, from a geotechnical engineering viewpoint the planned development at the site is feasible provided that the recommendations contained in this report are properly incorporated into the design plans and specifications, and implemented during construction. The main considerations for the planned development at this site include presence of existing "non-engineered" fills, seismicity and risk of seismic-induced settlement, near surface expansive soils, and potentially corrosive soils. We address these hazards and provide our recommendations for design and construction in the following report.

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 observation and testing 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

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design and construction of the proposed foundation enhancement structures, new buildings, and associated improvements planned at the Avenues Silicon Valley site in San Jose, California. Avenues World Holdings LLC authorized ENGEO to conduct the following scope of services:

- Review of available literature and geologic maps
- Subsurface field exploration
- Soil laboratory testing
- Data Analysis and conclusions
- Report preparation

For our use, we received the following:

1. Avenues Silicon Valley Master Plan prepared by Efficiency Lab dated January 30, 2019.
2. Preliminary Grading and Drainage Plan prepared by Kimley-Horn dated March 27, 2019.
3. Foundation Demands, 550 and 570 Meridian; transmitted by Skidmore, Owings & Merrill electronically May 6, 2019.

This report was prepared for the exclusive use of Avenues World Holdings LLC 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 AND DESCRIPTION

The project site is located approximately 500 feet north of Interstate Highway 280 and immediately northwest of a Valley Transportation Authority (VTA) rail line in San Jose, California (Figure 1). The site includes approximately 11 acres with six contiguous parcels. The site is bordered by Meridian Avenue to the west, Parkmoor Avenue to the south, Race Street to the east, and Harmon Avenue to the north. The site is currently occupied by two office buildings, parking garage, and three warehouse buildings, associated paved parking, and landscaped areas.

1.3 PROPOSED DEVELOPMENT

Based on the information provided, we understand the proposed development will consist of foundation enhancement of two existing office buildings (Buildings 1 and 2) and one parking garage (Building 3), demolition of three warehouse buildings, and construction of four new buildings ranging between 35 and 120 feet in height, connected by elevated walkways.

We understand Buildings 4 and 5 will incorporate one underground level to a depth of about 10 feet. Associated improvements will include a sports field, paved parking, hardscape and landscaped areas. Depictions of the proposed development are provided in Exhibits 1.3-1 and 1.3-2.

EXHIBIT 1.3-1: Building Rendering (Efficiency Lab, 2019)

EXHIBIT 1.3-2: Site Plan Rendering (Efficiency Lab, 2019)

Table 1.3-1 provides a summary of the existing and proposed site structures, their planned uses, and the number of stories.

TABLE 1.3-1: Site Structures

	BUILDING	NO. OF STORIES
EXISTING	Building 1 (550 Meridian)	3
	Building 2 (570 Meridian)	3
	Building 3 (Parking Garage)	3
PROPOSED	Building 4 (Sports Building)	3*
	Building 5 (Theatre Building)	3*
	Building 6 (Secondary Classrooms)	5
	Building 7 (Student Labs & Support)	12

*Buildings 4 and 5 also include a basement level

2.0 FINDINGS

2.1 SITE HISTORY

We reviewed relevant information regarding geotechnical and geological aspects of the site, including:

- Aerial photographs from various years starting in 1948.
- Available published geologic maps and reports.

Review of historic aerials indicates that the majority of the site was used for agricultural purposes, specifically orchards prior to 1948. In 1948, small farmhouses and associated access roads are visible in the northwestern and northeastern corners of the property. By 1956, the entire southeastern half of the property appears occupied by small farmhouses and additional access roads are visible across the site. All agricultural activities appear to cease by 1968 photographs, and additions of a large warehouse building in the southeastern corner and another warehouse building in the northeastern corner of the site are visible. By 1980 photographs, Interstate 280 (I-280) is visible. By 1987 photographs, it appears that paved parking occupies the entire western half of the site. Construction of the existing buildings appears complete by the 2002 photographs, and the site has largely remained unchanged since that time.

2.2 FIELD EXPLORATION

Our field exploration included drilling six borings and advancing six Cone Penetration Tests (CPTs) at various locations on the site. We performed our field exploration on March 29 and April 10 through 12, 2019. The approximate locations of our borings and CPTs are shown on Figure 2. We estimated the locations of our explorations using a recreational grade handheld GPS and from features visible in aerial photographs. A summary of exploration surface elevations

(based on the topographic survey provided by Kimley-Horn) and total exploration depths are provided in Table 2.2-1.

TABLE 2.2-1: Exploration Summary

EXPLORATION LOCATION	APPROXIMATE SURFACE ELEVATION (FEET, NAVD88)	TOTAL DEPTH (FEET)
1-B1	117.5	50
1-B2	116.5	80
1-B3	115.7	36
1-B4	114.0	51½
1-B5	116.5	50
1-B6	118.2	61½
1-CPT1	119.0	52
1-SCPT2	116.5	101
1-CPT3	116.8	52
1-CPT4	117.5	52
1-CPT5	116.5	57
1-CPT6	116.5	75

2.2.1 Borings

We observed drilling of six borings at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained the services of a drilling crew operating a truck-mounted drill rig and advanced borings using solid-flight auger and mud-rotary methods. We advanced the borings to depths ranging from 36 and 80 feet below existing ground surface (bgs). We permitted and backfilled the borings in accordance with the requirements of the Santa Clara Valley Water District.

We obtained soil samples at various intervals using standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D. split-spoon sampler) and California Modified samplers with 2½-inch inside diameter (I.D.). We obtained the blow counts shown on our bore logs with an automatic trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. We have not converted the blow counts presented on the borelogs using any correction factors. We also obtained hydraulically pushed Shelby tube samples at select locations. We present the fluid pressures recorded for the hydraulically pushed samples on the exploration logs in Appendix A.

We provide additional information about specific subsurface conditions at each location in our exploration logs in Appendix A. The soil type, color, consistency, and visual classification provided in the logs are in general accordance with the Unified Soil Classification System.

2.2.2 Cone Penetration Tests

We retained the services of a contractor with a CPT rig to advance CPTs at six locations to depths of approximately 50 to 100 feet below ground surface (bgs) in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). We present

the CPT logs in Appendix B. We drilled borings in proximity to 1-SCPT2 and 1-CPT4 to allow direct comparison of the data (matched pair). We also performed shear wave velocity (Vs) measurements in exploration 1-SCPT2.

2.3 REGIONAL AND SITE GEOLOGY

The site is located within the broad, north-south trending, alluvial-filled Santa Clara Valley. The Santa Clara Valley is an alluvium-filled trough bounded by the Diablo Range to the East and the Santa Cruz Mountains to the West. The valley is bounded to the north by the northern extent of Santa Clara County, and to the south roughly by the town of Morgan Hill. Sediments within the valley are comprised primarily of alluvium, typically consisting of interbedded clays, silts, sands and gravels, punctuated by bay deposits within proximity to the San Francisco Bay.

2.3.1 Site Geology

Wentworth (1999) mapped surficial deposits at the site as older Holocene alluvial fan deposits. Geologic mapping prepared by Dibblee (2007) (Figure 3) also indicates the site is predominantly underlain by Holocene-age alluvial fan deposits (Qya), consisting primarily of fine-grained sand, silt, and clay deposits. The Dibble mapping also indicates younger stream alluvium in fan deposits (Qa.2) consisting of gravel, silt, sand and clay are present along the southwestern edge of the site.

2.4 SEISMICITY

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 5 shows the approximate locations of active faults and significant historic earthquakes recorded within the San Francisco Bay Region.

The site is not located within a designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site; as such, the risk of fault rupture through the site is low. The San Jose and Stanford faults run approximately 0.9 and 1.25 miles to the southwest of the site, based on review of the Quaternary Fault and Fold Database of the United States (USGS, 2010). The faults are identified as concealed older faults in the South San Francisco Bay Area and generally do not require further fault study according to State criteria.

Based on the United States Geological Survey's (USGS) 2008 National Seismic Hazard Maps, the closest active fault in the area is the Monte Vista - Shannon, which is approximately 5.7 miles from the site. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

TABLE 2.4-1: Regional Faults and Seismicity

FAULT NAME	DISTANCE FROM SITE (MILES)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE
Monte-Vista Shannon	5 ½	Southwest	6.5
Calaveras	10	Northeast	7.0
Hayward-Rodgers Creek	10	Northeast	7.3
North San Andreas	10 ½	Southwest	8.1
Zayante-Vergeles	16	Southwest	7.0

Latitude: 37.317761 Longitude: -121.912599

The United States Geologic Survey evaluated Bay Area seismicity through a study by the 2014 Working Group on California Earthquake Probabilities (USGS, 2016). The WGCEP estimated that the probability of a moment magnitude (M_w) 6.7 or greater earthquake occurring before 2043 is 22 percent on the San Andreas Fault, 33 percent on the Hayward Fault, and 26 percent on the Calaveras Fault. The aggregate probability of a similarly sized earthquake in the San Francisco Bay Area was estimated to be 72 percent in the study.

2.5 SURFACE AND SUBSURFACE CONDITIONS

Our explorations encountered up to 11 inches of combined asphalt concrete pavement and aggregate base at the boring locations in flexible pavement areas. In areas of rigid pavement, up to 20 inches of combined concrete pavement and aggregate base was encountered.

Directly beneath the aggregate base we encountered lean (low- to moderate-plasticity) clay in the borings in the northern portion of the site, and fat (high-plasticity) clay in the borings in the southern portion of the site. Clayey soils were generally underlain by medium dense to very dense silty and clayey sand below a depth of 25 to 30 feet.

We developed Cross Sections (Figure 6) depicting the generalized subsurface profile based on our interpretation of the soil conditions encountered in our field explorations.

2.6 GROUNDWATER CONDITIONS

We estimated the depth to static groundwater in our CPT explorations from pore pressure dissipation tests. Groundwater was generally encountered approximately 32 to 34½ feet below ground surface. These depths correspond to elevations ranging from approximately 82½ to 84½feet (Datum = NAVD 88). We summarize our findings in the table below:

TABLE 2.6-1: Recorded Groundwater Levels

EXPLORATION LOCATION	GROUNDWATER DEPTH FROM CPT DETERMINATION (FEET BGS)*	CORRESPONDING GROUNDWATER ELEVATION FROM GROUND SURFACE (FEET, NAVD 88)
1-CPT1	34½	84½
1-SCPT2	32	84½
1-CPT3	33¼	83½
1-CPT4	33½	84
1-CPT5	32	84½
1-CPT6	34	82½

*Assumed phreatic surface (CPT pore pressure dissipation tests assuming hydrostatic conditions)

Plate 1.2 of the Seismic Hazard Zone Report for the San Jose West 7.5 Minute Quadrangle (CGS, 2002) maps the highest historical groundwater in the site vicinity as roughly 40 feet below existing grade. Likewise, historical groundwater elevation data available through Santa Clara Valley Water District based on groundwater monitoring wells indicates groundwater in the vicinity of the project site ranges between 30 to 50 feet below ground surface.

For purposes of our analyses and recommendations, we consider an appropriate design groundwater depth of approximately 30 feet below the existing ground surface. Fluctuations in

the level of groundwater may occur due to variations in rainfall, irrigation practices, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

We performed laboratory tests on select soil samples to evaluate their engineering properties. For this project, we performed laboratory testing as shown in the table below.

TABLE 2.7-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Natural Unit Weight	ASTM D7263	Appendix A
Natural Moisture Content	ASTM D2216	Appendix A
Atterberg Limits	ASTM D4318	Appendix C
Grain Size Distribution	ASTM D1140	Appendix C
Triaxial Compression – Unconsolidated, Undrained (TXUU)	ASTM D2850	Appendix C
Consolidation – Constant Rate of Strain	ASTM D4186	Appendix C
Sulfate Content	ASTM C1580	Appendix C

We include the laboratory test results in Appendix C and additional corrosivity tests performed by CERCO Analytical in Appendix E.

3.0 DISCUSSION AND CONCLUSIONS

Based on the results of our exploration, from a geotechnical engineering viewpoint, the planned development at the site is feasible provided that the recommendations contained in this report are properly incorporated into the design plans and specifications, and implemented during construction. The main considerations for the planned development at this site include presence of existing “non-engineered” fills, seismicity and risk of seismic-induced settlement, near surface expansive soils, and potentially corrosive soils. We address these hazards and provide our recommendations for design and construction in the following report.

3.1 EXISTING “NON-ENGINEERED” FILL

During our subsurface exploration, we encountered buried wood at a depth of 1 foot and buried concrete at a depth of 5 feet in the vicinity of Boring 1-B2, located in a loading dock area currently occupied by rigid pavement. We were unable to bypass the buried concrete encountered at 5 feet and stepped over our exploration approximately 5 feet in order to advance our boring, therefore, the total depth of buried concrete in this area was not determined. Undocumented fill and buried materials should be completely removed from this area during demolition of the existing site improvements.

In addition, based on the current site use and previous development activities, it is likely that existing fill deposits are present throughout the site underlying existing pavement around buildings, along utility trenches, landscape areas, and possibly buried structures.

Existing “non-engineered” fill could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. At the current time, we do not have records indicating the potential existing fill was engineered and

monitored during placement consistent with standards consistent with the proposed project; without documentation that the material was engineered when placed, we recommend considering all existing fill to be non-engineered. Non-engineered fill can undergo excessive settlement under new fill or building loads. To reduce the risk of settlement, the existing fil should be removed and recompacted in accordance with compaction specifications in this report. The extent and quality of existing fill should be evaluated and mitigated during grading activities.

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. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, landslides, tsunamis, or seiches is low to negligible at the site.

3.2.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, ground rupture is unlikely at the subject property.

3.2.2 Ground Shaking

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 comparable 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 nonstructural damage, and (3) resist major earthquakes without collapse, but with some structural as well as nonstructural 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 (SEAOC, 1996).

3.2.3 Liquefaction / Cyclic Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, 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, the sand may undergo deformation. If the sand undergoes virtually unlimited deformation without developing significant resistance, it is said to have liquefied, and 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.

3.2.3.1 Liquefaction Analysis

We performed an analysis of liquefaction potential based on the CPT data using the computer software CLiq (Version 2.1) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). This software also incorporates the method introduced by Idriss and Boulanger (2008) and updated by Boulanger and Idriss (2014).

For our analysis, we used a Peak Ground Acceleration (PGA_M) value of 0.50g with an earthquake magnitude of 8.1, based on the proximity of the San Andreas fault, and a groundwater depth of 30 feet. We performed the liquefaction assessment based on the methodology by Boulanger and Idriss (2014). We present the results of the liquefaction analysis in Appendix D.

To assess seismically induced settlements, we considered the methodology presented by Zhang et al (2002). Our estimates of total seismically induced settlement range from 0.3 to 2.5 inches, as presented in Table 3.2.3.1-1 (Boulanger and Idriss, 2014).

TABLE 3.2.3.1-1: Total Liquefaction-Induced Ground Settlement

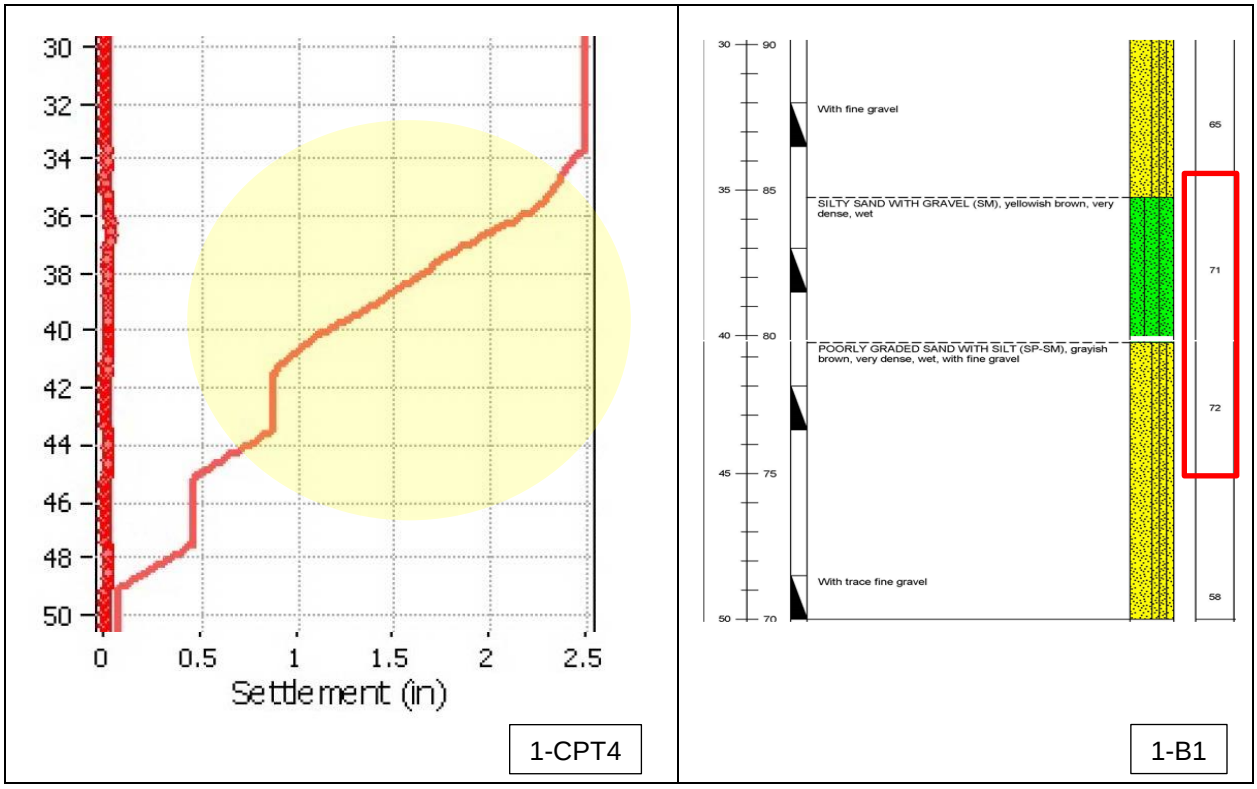
LOCATION	SETTLEMENT (INCHES)
1-CPT1	0.4
1-SCPT2	0.4
1-CPT3	1.1
1-CPT4	2.5
1-CPT5	0.4
1-CPT6	0.3

1-CPT4 and 1-B1:

To further assess liquefaction potential for CPT location 1-CPT4, which resulted in the higher estimated seismically induced settlement , we performed SPT-based liquefaction analyses of the sandy layer encountered in Boring 1-B1 (matched pair to 1-CPT4) using Youd et al. (NCEER 1998) (2001), Seed (2003), and Idriss and Boulanger (2008).

The CLiq analysis results in 2.5 inches of settlement between depths approximately 34 feet and 49 feet. The material encountered at these depths in the matched-pair boring (1-B1) is very dense poorly graded sand and silty sand with SPT blow counts of 58 to 72, as shown in Exhibit 3.2.3.1-1. Based on the SPT-based liquefaction analyses of the material between these depths, the sand is too dense to liquefy.

EXHIBIT 3.2.3.1.-1: 1-CPT4 and 1-B1 Comparison of Settlements and SPT Blowcounts



Based on the results of our CPT liquefaction analysis, considering comparison of 1-CPT4 to 1-B1, we estimate the overall total liquefaction-induced settlement at the project site to be less than ½ inch. Based on the findings of 1-CPT3, in some isolated areas outside the building footprints, the settlement value can be up to 1¼ inches.

Conservatively, we recommend assuming potential seismic-induced settlement of 1 inch is possible for design.

3.2.3.1 Liquefaction-Induced Surface Rupture

In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient enough force to break through the overlying soil and vent to the surface resulting in sand boils or fissures.

In 1985, Ishihara presented preliminary empirical criteria to assess the potential for ground surface disruption at liquefiable sites based on the relationship between thickness of liquefiable sediments and thickness of overlying non-liquefiable soil. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 308 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration. Based on review of the thickness of potentially liquefiable deposits and thickness of non-liquefiable cap materials, the risk for sand boils is negligible.

3.2.4 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 toward a free face or down a gentle slope. The project site and surrounding area is relatively flat and sloping ground conditions were not observed in the project vicinity. Therefore, we consider the risk of lateral spreading at the project site is very low.

3.3 EXPANSIVE SOIL

Moderately to highly expansive clay soils were encountered at the site. The results of Atterberg Limit tests showed Plasticity Indices (PIs) ranging between 10 to 29, indicating the soils exhibit moderate to high shrink/swell potential.

Where encountered, expansive soil can shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Generally, the weights of the structures should be high enough that expansive soil will not damage them though we do recommend special attention during construction in structural areas as well as areas with ridged surface improvements. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil is a generally cost-effective measure to address the expansive potential of the foundation soils. We provide specific grading recommendations for compaction of the moderately- to highly-expansive clay soil at the site.

3.4 CONSOLIDATION SETTLEMENT

We did not encounter highly compressible clay layers in our borings and CPTs. We did encounter medium stiff lean clay between approximately 15 and 25 feet bgs in Boring 1-B2, in the vicinity of the proposed 12-story tower (Building 7).

We performed a Constant Rate of Strain (CRS) consolidation test on a sample recovered from this medium stiff clay layer, the laboratory test indicates that this layer is “over-consolidated”. Based on our evaluation, considering preliminary building loads of approximately 1,500 to 3,000 pounds per square feet, we estimate re-consolidation settlement ranging from ¼ to ½ inch based on these bearing pressures, respectively. Differential settlement would be estimated to be less than half of the total settlement over similarly loaded areas.

3.5 SOIL CORROSION POTENTIAL

As part of this study, we collected soil samples for corrosivity testing. We delivered three of the samples to CERCO Analytical, who performed testing according to ASTM Test Methods for sulfate, chloride, resistivity, pH, and redox. We include the CERCO laboratory results in Appendix E. The remaining sample was tested for sulfate content in our in-house laboratory; the test result is included in Appendix C. We present a summary of the results in the table below.

TABLE 3.5-1: Soil Corrosivity Test Results

SAMPLE LOCATION	DEPTH (FEET, BGS)	SOIL TYPE	SULFATE (MG/KG)	CHLORIDE (MG/KG)	RESISTIVITY (OHMS-CM)	PH	REDOX (MV)
1-B1	8-8½	Clayey SAND	28	N.D.	1,800	8.02	280
1-B2	8½-9	Lean CLAY	N.D.	N.D.	2,100	7.63	290
1-B4	3-3½	Lean CLAY	31	N.D.	1,200	8.23	270
1-B4	6-6½	Lean CLAY	N.D. (< 50)	Not Tested			
1-B6	6-6½	Lean CLAY	N.D. (< 50)	Not Tested			

N.D. – None Detected

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Chapter 19, Sections 19.3.1.1 for structural concrete requirements. Based on the test results and ACI criteria, the tested soil would classify as 'Not Applicable' for sulfate exposure; there is no requirement for cement type or water-cement ratio for this category; however, a minimum concrete compressive strength of 2,500 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.

Based on the resistivity measurements, the samples are considered 'moderately corrosive' with respect to corrosion of buried cast/ductile iron and steel structures according to the National Association of Corrosion Engineers' 2006 "Corrosion Basics: An Introduction" interpretation of resistivity.

The above recommendations are provided 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.

3.6 SEISMIC DESIGN PARAMETERS

We used in-situ shear wave velocity measurements from our seismic CPT (1-SCPT2) to estimate the average shear wave velocity of the upper 100 feet of site soil. The average shear wave velocity in our measurement is approximately 990 feet per second, which classifies as a Site Class D soil. Based on this classification, we provide 2016 and 2019 CBC Seismic Design Parameters.

3.6.1 2016 CBC Seismic Design Parameters

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. We provide the 2016 CBC seismic design parameters in Table 3.6.1-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.6.1-1: 2016 CBC Seismic Design Parameters, Latitude: 37.317761 **Longitude:** -121.912599

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.5
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.6
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.5
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.9
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.0
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.6
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.5
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.5
Long period transition-period, T_L	12 sec

MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE = Maximum Considered Earthquake

3.6.2 2019 CBC Seismic Design Parameters

Additionally, as requested by the Structural Engineer, we provide the 2019 California Building Code (CBC) seismic design parameters in Table 3.6.2-1 below, in accordance with ASCE 7-16, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.6.2-1: 2019 CBC Seismic Design Parameters, Latitude: 37.317761 **Longitude:** -121.912599

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.5
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.6
Site Coefficient, F_A	1.00
Site Coefficient, F_V	Refer to ASCE Section 11.4.8*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.5
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Refer to ASCE Section 11.4.8*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.0
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Refer to ASCE Section 11.4.8*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.528
Site Coefficient, F_{PGA}	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.58
Long period transition-period, T_L	12 sec

MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE = Maximum Considered Earthquake
*A site-specific seismic hazard analysis is required to obtain these parameters unless the exception discussed in Section 11.4.8 is met.

4.0 FOUNDATION RECOMMENDATIONS

Based on our experience and the anticipated structure types, the proposed structures may be founded on structural mat foundations or conventional spread footing systems that consist of a perimeter strip footing with interior column spread footings. Based on discussions with the project Structural Engineer, we understand a structural mat foundation system is the preferred option for the foundation enhancement of Buildings 1 and 2.

4.1 PRELIMINARY FOUNDATION DATA

Tables 4.1-1 and 4.1-2 below present structural loading information provided by the project Structural Engineer, Skidmore, Owings & Merrill (transmitted electronically May 6, 2019), for Buildings 1 and 2 of the subject project.

TABLE 4.1-1: Building 1 Foundation Loads

FOUNDATION ELEMENT	AREA (SQUARE FEET)	TOTAL LOAD* (KIPS)
MAT 1	612	462
MAT 2	990	994
MAT 3	612	427

*ASD Gravity Loads (D+L)

TABLE 4.1-2: Building 2 Foundation Loads

FOUNDATION ELEMENT	AREA (SQUARE FEET)	TOTAL LOAD* (KIPS)
MAT 1	864	455
MAT 2	960	814

*ASD Gravity Loads (D+L)

Structural loading information was not available for Buildings 3 through 7, at the time of preparation of this report. Our analyses have assumed structural loads for Buildings 3 through 7 will be in the range of 1,500 to 3,000 psf. Once the loads have been determined by the Structural Engineer for Buildings 3 through 7, these should be provided to us for review. If loads are greater than assumed in our analyses, our recommendations should be revisited and revised as necessary.

4.2 VERTICAL SETTLEMENT

Shallow foundation design should consider the following estimated total and differential settlements. The differential value should be assumed to act between adjacent column supports or over a 50-foot distance.

TABLE 4.2-1: Total and Differential Settlement Estimates

SETTLEMENT TYPE	ESTIMATED TOTAL VERTICAL SETTLEMENT (INCHES)	ESTIMATED DIFFERENTIAL VERTICAL SETTLEMENT (INCHES)
Liquefaction-Induced Settlement	1	½
Static Settlement*	½	¼

*Assumed structural load of 3,000 psf

4.3 SHALLOW FOOTINGS

Conventional footings may be used to support the proposed structures. We recommend the following minimum footing dimensions.

TABLE 4.3-1: Minimum Footing Dimensions

FOOTING TYPE	MINIMUM DEPTH (INCHES)*	MINIMUM WIDTH (INCHES)
Continuous	30	12
Isolated	30	12

*below lowest adjacent pad grade

The footing foundations recommended in the above table can be designed for a maximum allowable bearing pressure of 3,000 psf for dead-plus-live loads. This bearing capacity can be increased 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.

If a two-pour system is used for buildings with footings and slab-on-grade floors, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finished exterior grade. If this is not done, we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

The Structural Engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, continuous footings should be designed to structurally span a clear distance of 5 feet.

4.4 STRUCTURAL MAT FOUNDATION

A structural mat foundation is planned for the foundation enhancement of Buildings 1 and 2. A structural mat foundation is also a suitable foundation system for the proposed buildings. Foundation design should consider the recommended parameters provided in the following table.

TABLE 4.4-1: Structural Mat Foundation Parameters

DESIGN PARAMETER	RECOMMENDATION
Average Allowable Bearing Pressure	1,500 psf for dead-plus-live loads (increased by one-third for the short-term effects of wind or seismic loading).
Edge Cantilever Distance	6 feet
Unsupported Interior Free Span	20 feet
Modulus of Subgrade Reaction, k_s*	100 psi/in**

**See Section 4.4.1 for additional recommendations.
**Pounds per square inch per inch of deflection.

If consideration of short-term loads is necessary, we recommend a one-third increase of the design modulus value.

4.4.1 Subgrade Modulus Variation

Subgrade modulus determinations consider a square plate, therefore, variation of subgrade modulus based on mat shape should be considered in design of structural mat foundations, in accordance with the guidelines presented by Terzaghi (1955). Table 4.4.1-1 presents reduction factors to apply to subgrade modulus, based on mat length to width ratio.

TABLE 4.4.1-1: Subgrade Modulus Variation

LENGTH/WIDTH RATIO	REDUCTION FACTOR
1	1.00
1.5	0.90
2	0.85
2.5	0.80
3	0.78

4.5 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 presented as an equivalent fluid weight in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 250 pcf
- Coefficient of Friction: 0.30

4.6 BUILDING SUBGRADE PREPARATION

To improve foundation performance for the planned structures, undocumented non-engineered existing fill within the bounds footprint of the building footprints should be completely excavated and replaced with properly compacted uniform engineered fill, placed and compacted in accordance with Section 6.5.

As previously discussed, high plasticity clay was encountered at shallow depths on site. We recommend that the structures be situated on a layer of low-expansive engineered fill material that extends at least 18 inches below foundation subgrade elevation for conventional footings and 24 inches below foundation subgrade elevation for mat foundations; the low expansive material should extend 5 feet beyond the building footprints.

4.7 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade or mat foundations, water vapor from beneath the slab will migrate through the slab/mat 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. Construct a moisture retarder system directly beneath the slab on-grade or mat that consists of the following:

a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be underlain by,

b. 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve. This layer is only required for slab-on-grade floors.

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. If a sand or pea gravel is used with a structural mat above the vapor retarder membrane, the edge of the mat should be thickened to cut off water getting in between the slab and the membrane. The thickened edge should be as thick as the sand or pea gravel layer and at least 12 inches wide.

5.0 BASEMENT WALLS AND NON-BUILDING WALLS

5.1 LATERAL SOIL PRESSURES

Basement walls for Buildings 4 and 5 should be designed for at-rest lateral loading conditions. Should cantilever retaining walls at the site be required, they can be designed for active lateral loading conditions. The recommended lateral equivalent fluid pressures (static case) are presented below:

TABLE 5.1-1: Lateral Earth Pressures

LOADING CONDITION	EQUIVALENT FLUID PRESSURES (PCF)	
	WITHOUT HYDROSTATIC PRESSURES	WITH HYDROSTATIC PRESSURES
Cantilevered (Active)	45	85
Restrained (At-Rest)	65	105

We recommend placing a drain behind all walls to reduce hydrostatic pressure, if a drain is not feasible, the basement walls should be designed with hydrostatic pressure. Recommendations for wall drainage follow in the next section.

Where surcharge loads from vehicles are expected within a distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 125 psf applied over the entire height of the wall or 10 feet, whichever is less.

Passive pressures acting on retaining walls may be assumed as 250 pounds per cubic foot (pcf), provided that the area in front of the retaining wall is level for a distance of at least 10 feet. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or concrete slab.

5.2 RETAINING WALL DRAINAGE

Wall drainage for any walls may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about 1 foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soils. Drainage should be collected by perforated pipes and directed to sump.

All backfill should be placed in accordance with recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls.

5.3 SEISMIC DESIGN CONSIDERATIONS

Where seismic pressures are considered in wall design, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures (regardless if the wall is restrained or unrestrained), and can be calculated as follows:

$$\Delta P = 12 \times H^2$$

H is the design retained height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.3 x H from the base of the wall.

5.4 EXCAVATIONS AND TEMPORARY SHORING SYSTEMS

Grading and construction activities and any structure excavations should be sloped in accordance with OSHA requirements. It is the responsibility of the contractor to establish and maintain stable excavation slopes in accordance with OSHA requirements. For planning purposes, the site soil would generally be classified as a Type B soil by OSHA criteria. If the project sequencing results in construction of improvements prior to excavations, there may be a need for lateral support.

5.4.1 Tie-Back Anchors

At this time, we do not anticipate the use of tie-back anchors for shoring purposes. Should this type of anchoring be required for shoring, we should be contacted to provide appropriate design recommendations.

6.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as determined by a representative of our firm.

As used in this report, 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.

6.1 DEMOLITION AND STRIPPING

Grading operations should be observed and tested by our qualified field representative. We should be notified a minimum of three days prior to grading in order to coordinate our schedule with the grading contractor.

Site development should commence with the removal of existing pavement and buildings as well as excavation and removal of buried structures, including utilities and foundations. All debris and soft compressible soils should be removed from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow. The depth of removal of such materials should be determined by our representative in the field at the time of grading.

Existing vegetation should be removed from areas to receive fill or improvements and those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. All excavations from demolition below design grades should be cleaned to a firm undisturbed native soil surface determined by our representative. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. All backfill materials should be placed and compacted as engineered fill according to the recommendations in Section 6.5.

Materials and debris should be removed from the project site. With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), the upper 10 feet of subsurface material is suitable for reuse as engineered fill.

6.2 EXISTING FILL REMOVAL

As stated previously, approximately 5 feet of existing fill should be anticipated in the vicinity of existing improvements at the site based on our explorations and due to existing development conditions. In the vicinity of Building 7, undocumented fill may be deeper, based on the findings of Boring 1-B2. Where existing fill is located within the limits of any structural improvements that may be sensitive to settlement, we recommend removal of existing fill to competent native soil, as evaluated by our field representative. If in a fill area, the base of the subexcavations should be processed, moisture conditioned (as needed) and compacted in accordance with the recommendations for engineered fill.

If existing fill is left in place in portions of the site that are being developed with walkways or other improvements that are not sensitive to settlement, on-going maintenance should be anticipated.

6.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or 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 lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation.

6.4 ACCEPTABLE FILL

6.4.1 Soil

Onsite soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 4 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 12 and at least 20 percent passing the No. 200 sieve. It is important that we sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

6.4.2 Reuse of Onsite Recycled Materials

If desired, the aggregate base from the existing pavement section can be considered for use as recycled aggregate to replace some of the import aggregate base for pavements as well as for structural fill. The material will need to be broken down, but not pulverized, to have a maximum particle size less than 6 inches if used for fill and should conform to the gradations of aggregate base if used to substitute for roadway base.

6.5 FILL COMPACTION

6.5.1 Grading in Structural Areas

After removing the loose soil in areas not requiring fill to achieve planned Civil grades and/or outside the lateral extent of the 3 feet of engineered fill recommended for building pads, the contractor should scarify to a depth of at least 8 inches then moisture condition and compact the subgrade in accordance with the table below. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

TABLE 6.5.1-1: Fill Placement Requirements

MATERIALS	FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
Site Soil	General Fill	90	3
Low-Expansive Import Material	General Fill	90	2
Site Soil and Low-Expansive Import Material	Pavement Subgrade*	95	1
Class 2 Aggregate Base	Pavement Section	95	0

*Upper 6 inches

6.5.2 Landscape Fill

In landscaping areas, the contractor should process, place, and compact fill in accordance with Section 6.5.1, except compact fill to at least 85 percent relative compaction.

6.5.3 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

Utility trench backfill should conform to the recommendations in Section 6.5.1.

Where utility trenches cross underneath structures, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the structure. The plug should be constructed using a sand-cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility crosses the structure perimeter.

Jetting of backfill is not an acceptable means of compaction. Thicker loose lift thicknesses may be allowed based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

6.6 SLOPE GRADIENTS

Final slope gradients should be constructed to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

6.7 SITE DRAINAGE

The project Civil Engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, finish grades should 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.

6.8 STORMWATER BIORETENTION AREAS

We do not expect the existing site soil to have adequate permeability for stormwater infiltration, unless subdrains are installed. We recommend assuming little stormwater infiltration will occur through the existing site soil. If bioretention areas are planned, we recommend that, when practical, they be placed a minimum of 5 feet away from property lines and 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, site improvements located adjacent to bioretention areas that are underlain by base rock, 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 that will experience lateral loads (such as from impact or traffic), additional design considerations may be required. 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.

7.0 PAVEMENT DESIGN

We prepared pavement design recommendations based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The sections provided below should be reviewed and revised, if applicable, based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading.

7.1 FLEXIBLE PAVEMENTS

We developed the following pavement sections for parking areas and access streets using Traffic Indices of 5 to 9, based on an assumed R-value of 5 and Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety).

TABLE 7.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (AC) (INCHES)	CLASS 2 AGGREGATE BASE (AB) (INCHES)
5	4	7½
6	4	11½
7	4	15½
8	4½	18½
9	5	21½

The Civil Engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

7.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections and reinforcement should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.

- Provide concrete with a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

7.3 PAVEMENT SUBGRADE PREPARATION

Pavement subgrade preparation should comply with the following minimum requirements:

- All pavement subgrades should be scarified to a depth of 10 inches below finished subgrade elevation and compacted in accordance with Section 6.5.1. Pavement subgrades should also be prepared in accordance with City of San Jose requirements if they are located in public streets.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted in accordance with Section 6.5. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.

7.4 CUT-OFF 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 towards pavement. If it is desired to install pavement cutoff barriers, they should be placed 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 6 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, the cutoff barrier may be eliminated.

7.5 SECONDARY SLABS-ON-GRADE

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor plazas exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches and include control and construction joints in accordance with current Portland Cement Association guidelines.

Exterior slabs should slope away from the buildings to prevent water from flowing toward the foundations. Site soil should be moistened just prior to concrete placement.

We recommend that flatwork leading to a building entrance area be structurally independent of the building foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinge-slab should be designed to accommodate movements of approximately ½ inch. Flatwork should be reinforced to allow for the appropriate span in the event of settlement. Maintenance or replacement of entry slabs should also be expected following a seismic event as the ground settles at the perimeter of buildings.

8.0 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 identify certain changes, which may 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 confirm that the site is properly prepared, the selected fill materials are satisfactory, and that the placement and compaction of the fills have been performed in accordance with our recommendations and the project specifications. Sufficient notifications to us prior to earthwork is important.

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

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the Avenues Silicon Valley project discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations. 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 strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed 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 assume that our subsurface exploration data are representative of the actual subsurface conditions across the

site. Considering possible underground variability of soil, rock, fill, and groundwater, additional unexpected costs may be incurred in completing the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO should 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, or flood potential. 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 should be notified immediately.

This document must not be subject to unauthorized reuse, that is, reuse 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 recommendations. 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 onsite 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 boundaries 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.

SELECTED REFERENCES

American Concrete Institute, (2005), Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures, ASCE Standard, ASCE/SEI 7-10.

American Society of Civil Engineers, 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE Standard, ASCE/SEI 7-16.

Boulanger, R. W., & Idriss, I. M. (2014), CPT and SPT based liquefaction triggering procedures. Rep. No. UCD/CGM-14, 1.

Bray, J. D., & Sancio, R. B, (2006), "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotechnical and Geoenvironmental Engineering, 132(9), 1165-1177.

California Building Code, (2016). http://www.bsc.ca.gov/codes.aspx.

California Geologic Survey, (2008), Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

California Geologic Survey, (2002), Seismic Hazard Zone Report, Official Map, San Jose West Quadrangle.

California Department of Transportation; Highway Design Manual, 6th Edition; July 15, 2016.

California Department of Transportation; Caltrans Storm Water Quality Handbook; Pervious Pavement Design Guidance; May 2016.

California Department of Transportation; Standard Specification Revisions, Section 26-1.02.

Dibblee, T.W., 2007, Geologic Map of the San Jose West Quadrangle, Santa Clara County, California.

Hart, E.W. and Bryant, W.A., (1997), Fault rupture hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.

Historical Aerials, www.historicaerials.com.

Idriss, I.M. and Boulanger, R.W., (2008), Soil Liquefaction During Earthquakes; Earthquake Engineering Research Institute.

Ishihara, K. (1985), Stability of Natural Deposits During Earthquakes, Proc 11th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

Robertson, P. K. and Campanella, R. G. (1988), Guidelines for Geotechnical Design Using CPT and CPTU Data. Civil Engineering Department, University of British Columbia.

SELECTED REFERENCES (Continued)

Robertson, P. K. (2009), Performance based earthquake design using the CPT, Gregg Drilling and Testing, Inc.

SEAOC, (1996), Recommended Lateral Force Requirements and Tentative Commentary. Structural Engineers Association of California.

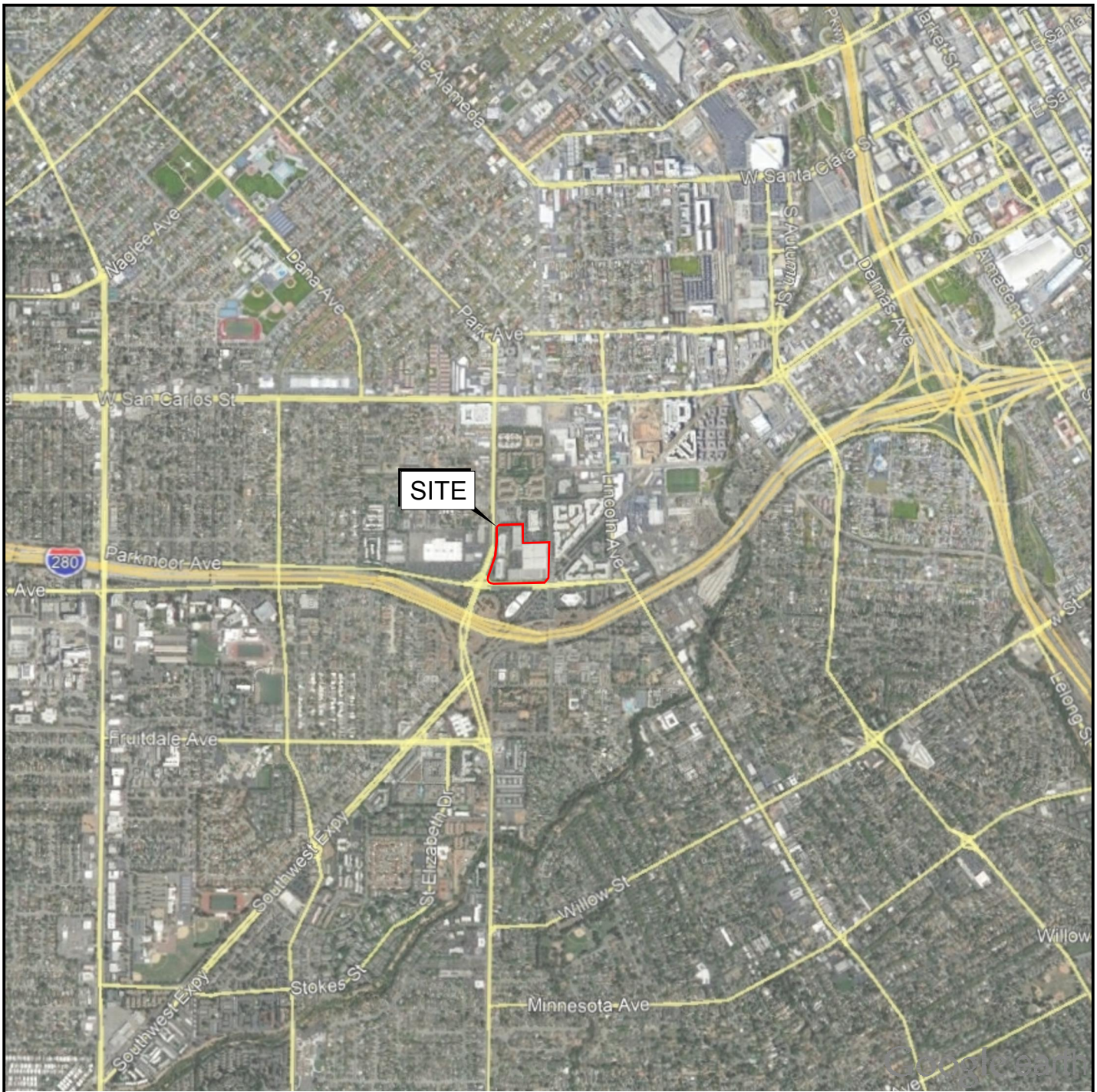
Wentworth et al., 1999, Preliminary Geologic Map of the San Jose 30 x 60-Minute Quadrangle, California, U.S. Geological Survey, Open-File 98-795 Part 7.

Youd, T. L. and C. T. Garris, (1995), Liquefaction induced Ground-Surface Description: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 – 809.

Youd, T. L. and I. M. Idriss, (2001), Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils.

Zhang, G. Robertson. P.K, Brachman, R., (2002), Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.

FIGURES



N

0 2000
FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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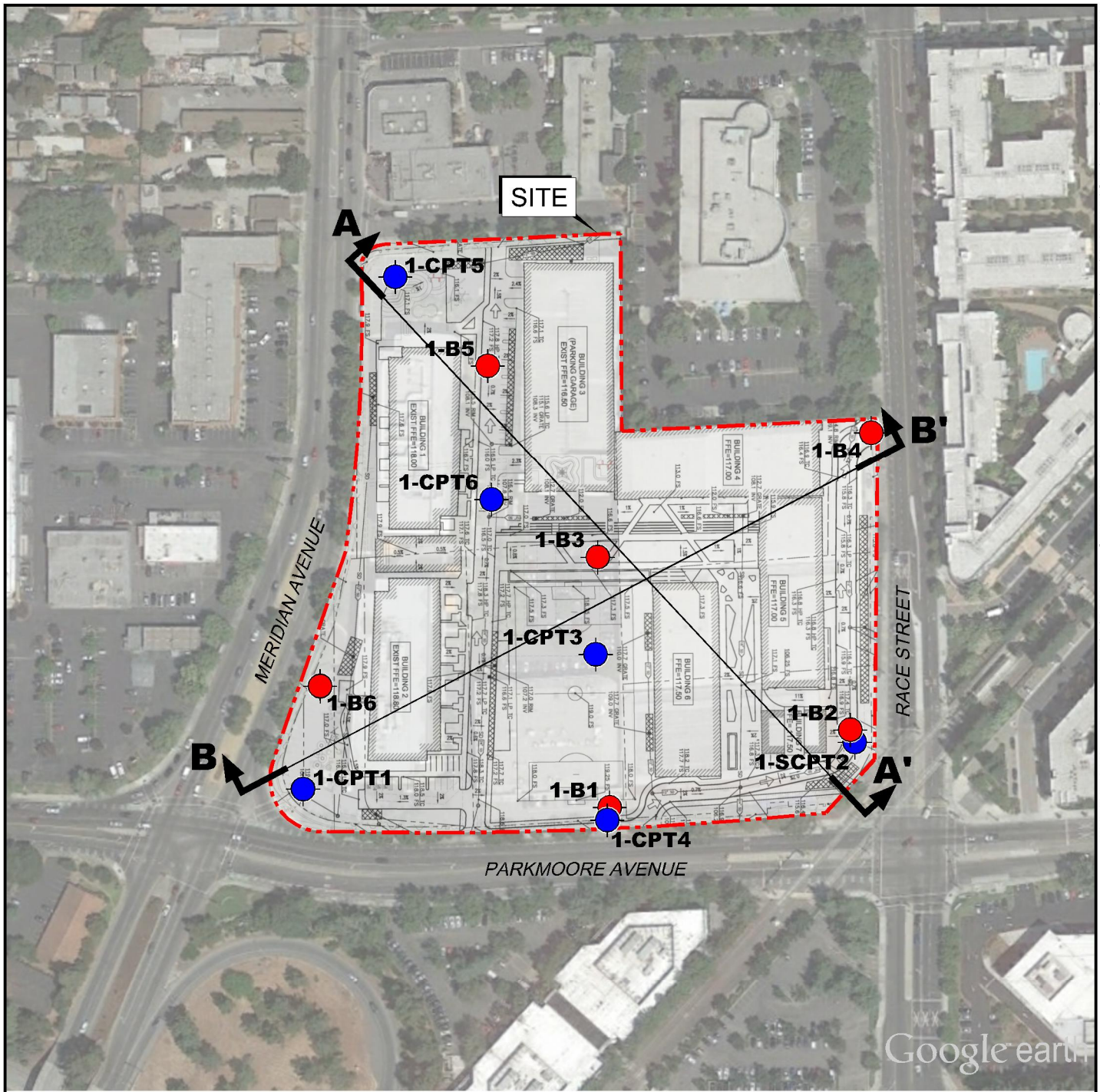
VICINITY MAP AVENUES SILICON VALLEY SAN JOSE, CALIFORNIA	PROJECT NO.: 15929.000.000		FIGURE NO.
	SCALE: AS SHOWN		1
	DRAWN BY: JV	CHECKED BY: YZ	

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-1-VicMap-0319.dwg Plot Date: 3-12-19 JVergara

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B6 — BORING (ENGEO, 2019)

1-CPT6 — CONE PENETRATION TEST (ENGEO, 2019)

B — B' CROSS SECTIONS (SEE FIGURE 6)

N

0 200 FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE, KIMLEY HORN 2019

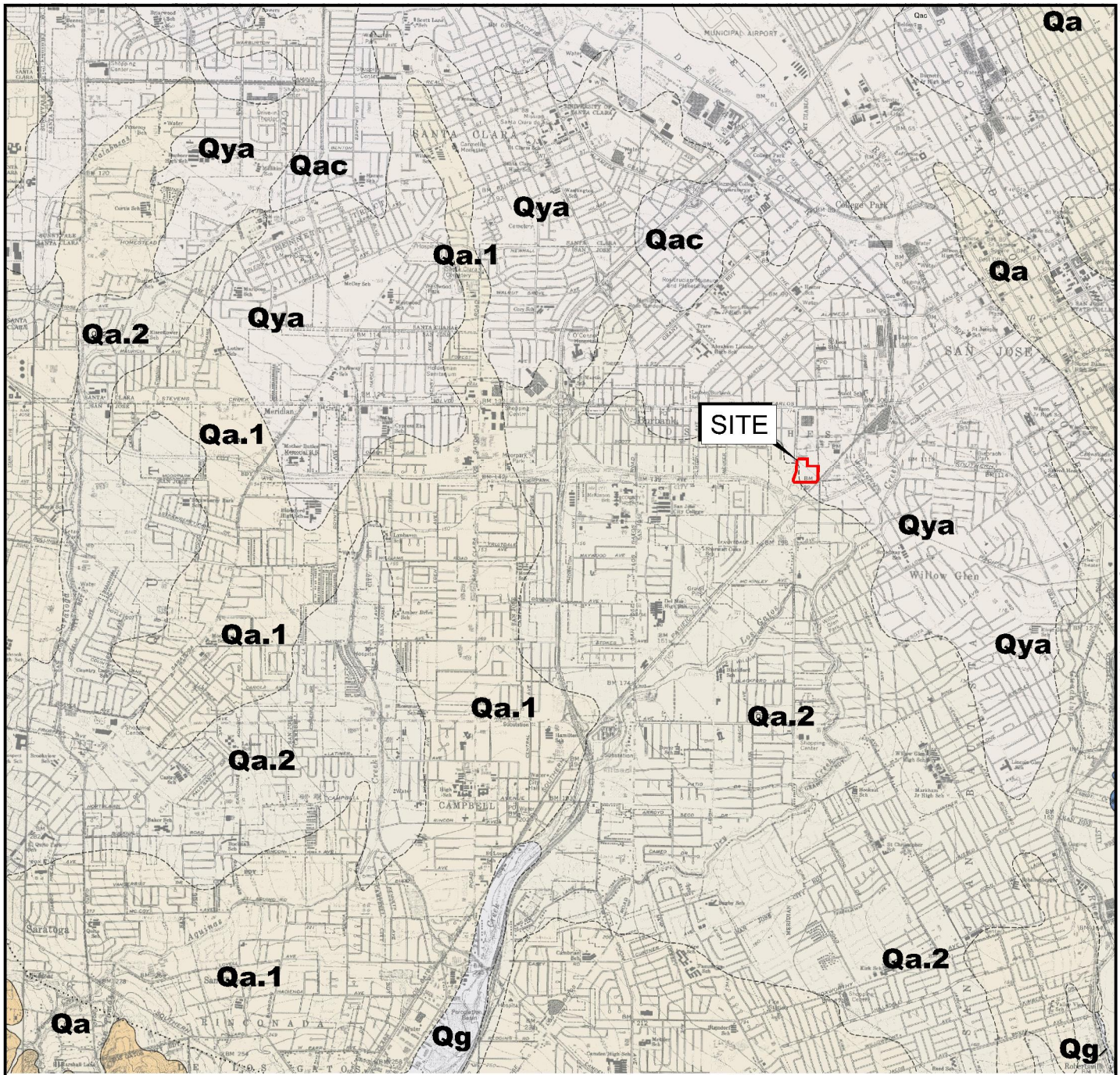
ENGEO Expect Excellence

SITE PLAN AVENUES SILICON VALLEY SAN JOSE, CALIFORNIA	PROJECT NO.: 15929.000.000	FIGURE NO.
	SCALE: AS SHOWN	2
	DRAWN BY: EJ / CHECKED BY: YZ	

FILE PATH: G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-2-SitePlan-0519.dwg SAVE DATE: 5/10/2019 11:03:56 AM SAVED BY: EJimenez

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-2-SitePlan-0519.dwg Plot Date: 5-10-19 EJimenez

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qa	UNDIFFERENTIATED STREAM ALLUVUIM AND YOUNGER ALLUVIAL FAN DEPOSITS
Qg	UNDIFFERENTIATED STREAM ALLUVIUM
Qa.1	ALLUVIAL FAN DEPOSITS
Qa.2	YOUNGER STREAM ALLUVIUM IN FAN DEPOSITS
Qya	DISTAL ALLUVIAL FAN DEPOSITS
Qac	ALLUVIAL INTRA-FAN DEPOSITS; FOSSILIFEROUS

N

0 5000
FEET

BASE MAP SOURCE: DIBBLEE, 2007

ENGEO
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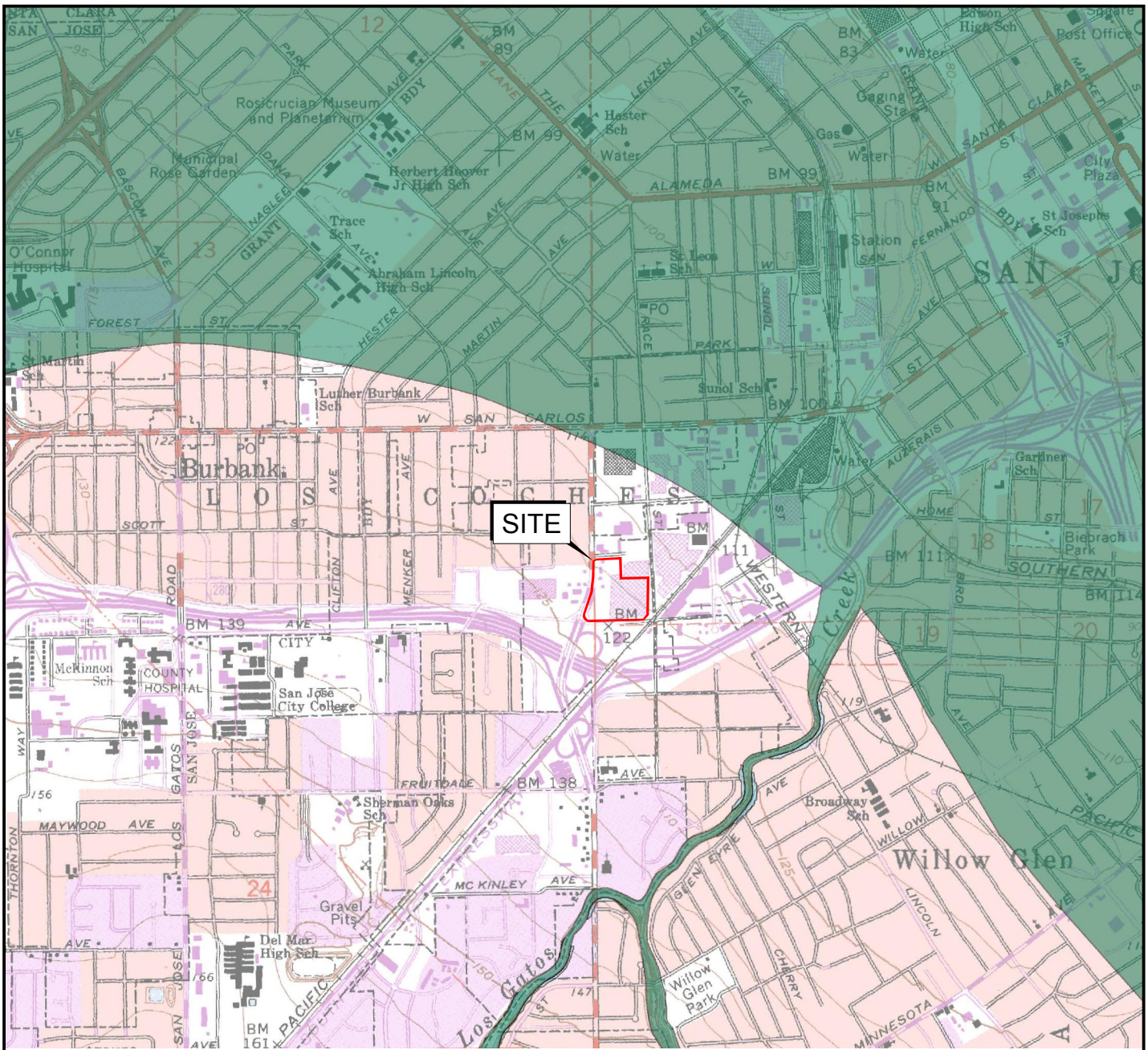
REGIONAL GEOLOGIC MAP
AVENUES SILICON VALLEY
SAN JOSE, CALIFORNIA

PROJECT NO.: 15929.000.000
SCALE: AS SHOWN
DRAWN BY: EJ
CHECKED BY: YZ
FIGURE NO. 3

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-3-RegGeologic-0519.dwg SAVE DATE: 5/10/2019 4:21:36 PM SAVED BY: EJimenez
G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-3-RegGeologic-0519.dwg Plot Date: 5-13-19 EJimenez



EXPLANATION

N

0 2000
FEET

LIQUEFACTION

AREAS WHERE HISTORIC 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

EARTHQUAKE-INDUCED LANDSLIDES

AREAS WHERE 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

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2002

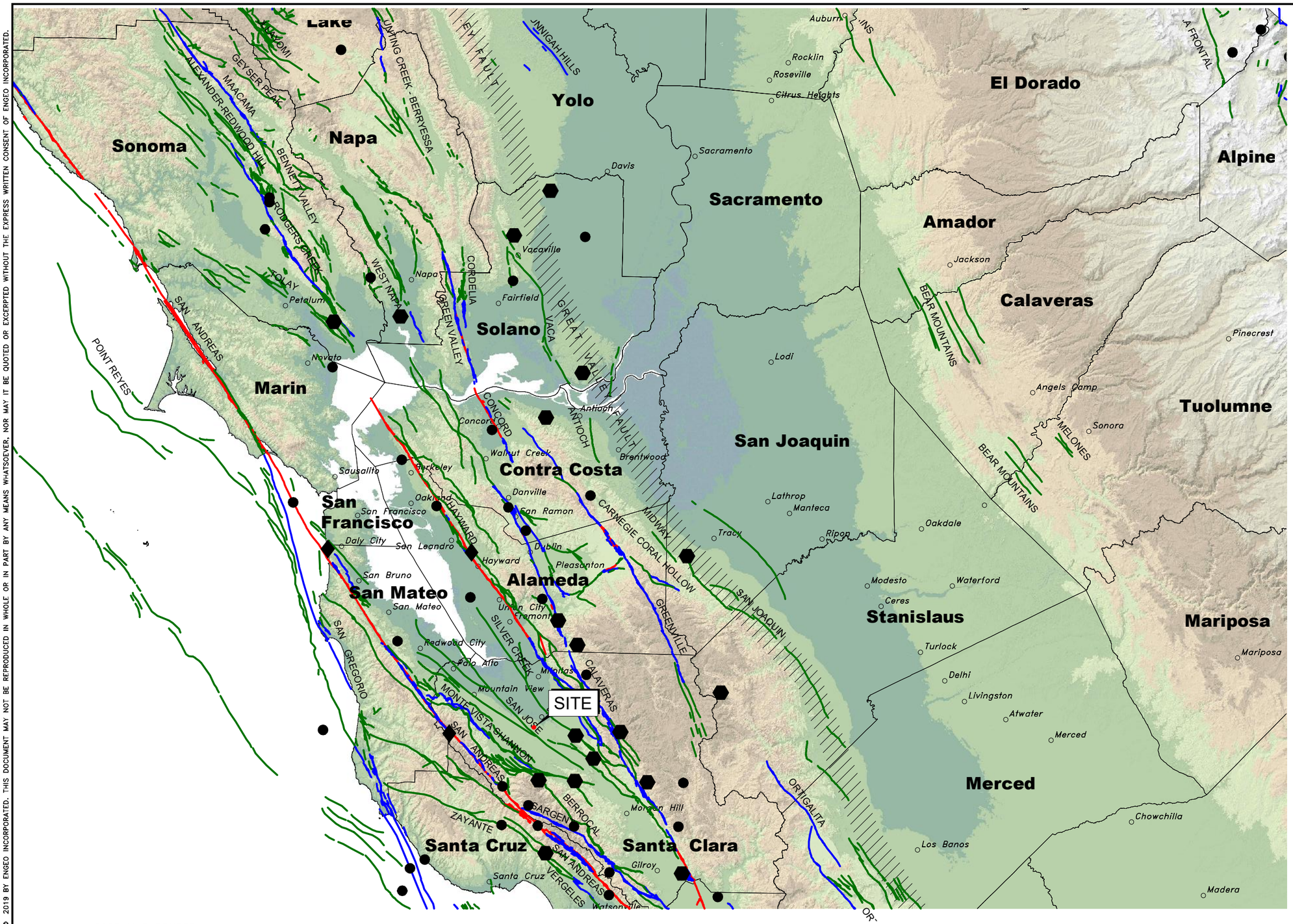
ENGEO
Expect Excellence

SEISMIC HAZARD ZONES MAP
AVENUES SILICON VALLEY
SAN JOSE, CALIFORNIA

PROJECT NO.: 15929.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-4-SeismicHazard-0319.dwg Plot Date: 3-12-19 JVergara

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

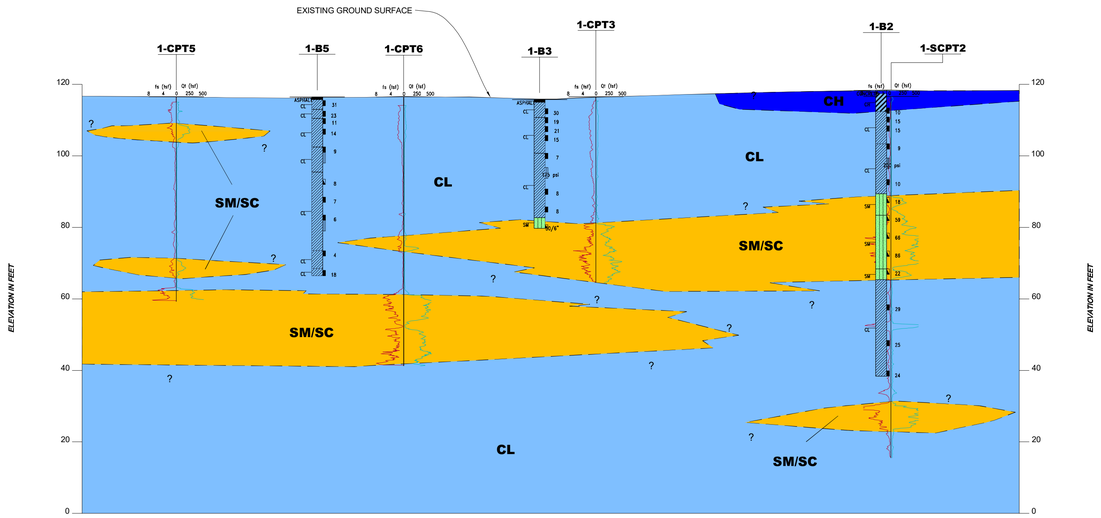
ENGEO Expect Excellence

REGIONAL FAULTING AND SEISMICITY
AVENUES SILICON VALLEY
SAN JOSE, CALIFORNIA

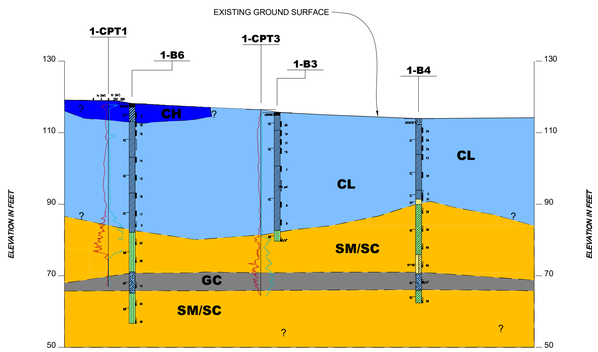
PROJECT NO.: 15929.000.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: YZ

ORIGINAL FIGURE PRINTED IN COLOR

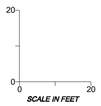
G:\Drafting\DRAFTING2_Dwg_13000 Plus\15929\000\031219-GEX\15929000000-5-RegionalFault-0319.dwg Plot Date: 5-02-19 EJimenez



SECTION A-A'
1"=20'



SECTION B-B'
1"=20'



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

CL	PRIMARILY LOW TO MODERATE PLASTICITY CLAY
CH	HIGH PLASTICITY CLAY
SM/SC	SILTY AND CLAYEY SAND
GC	CLAYEY GRAVEL
B-3	BORING
?	QUERY WHERE UNKNOWN
— — —	GEOLOGIC CONTACT

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CROSS SECTIONS
AVENUES SILICON VALLEY
SAN JOSE, CALIFORNIA

PROJECT NO.: 15929.000.000		FIGURE NO.
SCALE: AS SHOWN		6
DRAWN BY: EJ	CHECKED BY: YZ	

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

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 LESS THAN 5% 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 LESS THAN 5% 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 LIMIT 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 - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	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.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

Solid - Layer Break

Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

Groundwater level during drilling

Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 1-B1

ENGEO — Expect Excellence

LATITUDE: 37.31677 LONGITUDE: -121.912408

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 117½ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			2½" AC over 8" AB												
			FAT CLAY WITH GRAVEL (CH), very dark brown, slightly moist, [FILL]												
			LEAN CLAY WITH GRAVEL (CL), dark yellowish brown, hard, slightly moist, low plasticity, [NATIVE]			12					11.8	128.7		4.5*	PP
	115														
			Dark brown, very stiff			34	33	17	16					3.75*	PP
5															
			CLAYEY SAND (SC), reddish brown, medium dense, moist			15							1000*	2*	PP+TV
	110														
10			Stiff			250 psi				48	18	104.7			
	105														
			LEAN CLAY WITH SAND (CL), reddish brown, stiff, moist												
15															
	100					9					22.7	100.9	525		UU
20															
	95		CLAYEY SAND (SC), dark yellowish brown, loose, wet			9	28	19	9	45					
25															

LOG OF BORING 1-B1

ENGEO
Expect Excellence

LATITUDE: 37.31677 LONGITUDE: -121.912408

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 117½ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED SAND WITH SILT (SP-SM), brown to yellowish brown, medium dense, wet, with trace fine gravel												
	90					28				12					
30															
	85		Very dense, with fine gravel			65									
35			SILTY SAND WITH GRAVEL (SM), yellowish brown, very dense, wet												
	80					71									
40			POORLY GRADED SAND WITH SILT (SP-SM), grayish brown, very dense, wet, with fine gravel												
	75					72									
45															
	70		With trace fine gravel			58									
50			End of boring at 50 feet below ground surface. Groundwater not measured due to drilling method.												

LOG OF BORING 1-B2

ENGEO — Expect Excellence

LATITUDE: 37.317064 LONGITUDE: -121.91129

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 80 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4" Concrete over 8" AB												
			FAT CLAY (CH), very dark brown, soft, moist, high plasticity, [FILL]												
	115														
5			LEAN CLAY (CL), light brown, very stiff, very moist, [NATIVE]			10	36	26	10		19.4	103.8		2.75*	PP
	110					15								2.25*	PP
10			With trace fine-grained sand			15					19.4	107.6			
	105														
15			LEAN CLAY WITH SILT (CL), light brown, medium stiff, moist			9					24.9	100.3	575		UU
	100														
20						200 psi	28	17	11		21.7	100.5	500*	2*	PP+TV
	95														
25			Medium stiff, with fine-grained sand			10					26.4	98.8	1386		UU

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.317064 LONGITUDE: -121.91129

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 80 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	90														
30			SILTY SAND (SM), reddish brown, medium dense, wet, with trace fine gravel			18				16					
	85														
35			Grayish brown, very dense, wet, with trace fine gravel			59									
	80														
40						66									
	75														
45			Light reddish brown			86									
	70														
50			SILTY SAND WITH GRAVEL (SM), light brown, medium dense, wet			22									

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.317064 LONGITUDE: -121.91129

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 80 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	65		LEAN CLAY WITH SILT (CL), grayish green, stiff, moist												
55															
	60														
60						29							900*	1.5*	PP+TV
	55														
65															
	50														
70			With fine-grained sand			25					28.2	100.1	300*	1.25*	PP+TV
	45														
75															

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.317064 LONGITUDE: -121.91129

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 80 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	40														
			Light brown			24									
80			End of boring at 80 feet below ground surface. Groundwater not measured due to drilling method.												

LOG OF BORING 1-B3

ENGEO Expect Excellence

LATITUDE: 37.317699 LONGITUDE: -121.912475

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 36 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 115¾ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			2" AC over 7" AB												
	115		SANDY LEAN CLAY (CL), dark brown, very stiff, moist												
						30	40	19	21					3.25*	PP
5	110		LEAN CLAY (CL), reddish brown, hard, moist, with trace fine-grained sand			19								4*	PP
			Very stiff, very moist			21								3.5*	PP
10	105		Grayish brown			15					20.9	105.8	1200*	3*	PP+TV
15	100		LEAN CLAY WITH SILT (CL), light brown, medium stiff, very moist			7	36	13	23		26.4	97.4	655		UU
20	95					175 psi									
													400*	1*	PP+TV
25															

LOG OF BORING 1-B3

ENGEO
Expect Excellence

LATITUDE: 37.317699 LONGITUDE: -121.912475

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 36 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 115¾ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	90		With trace fine-gravel			8					20.4	95.1	629		UU
30	85					8							500*	1*	PP+TV
			SILTY SAND WITH GRAVEL (SM), grayish brown, very dense, wet												
35	80					50/6"				16					
			End of boring at 36 feet below ground surface. Groundwater not measured due to drilling method.												

LOG OF BORING 1-B4

ENGEO
Expect Excellence

LATITUDE: 37.31818 LONGITUDE: -121.911215

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 114 ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			8" Concrete over 12" AB												
			LEAN CLAY (CL), dark reddish brown, very stiff, moist, with trace fine-grained sand												
						20	37	17	20		20.4	108.2	3838		UU
	110														
5			Hard, with fine-grained sand, Sulfate Content = Non-detect			26								4.5*	PP
			Yellowish brown			20					27.6	91.2		4.5*	PP
	105														
10			SANDY LEAN CLAY (CL), grayish brown, hard, moist to wet, with trace fine gravel			17								4*	PP
	100														
15			Very stiff, very moist, with fine gravel			19								2*	PP
	95														
20			LEAN CLAY WITH SILT (CL), light brown, stiff, very moist, with trace fine-grained sand			5									
						31								1*	PP
			POORLY GRADED SAND WITH GRAVEL (SP), brown to gray, medium dense, wet												
	90		CLAYEY SAND WITH GRAVEL (SC), grayish brown, medium dense, wet												
25						26				13					

LOG OF BORING 1-B4

ENGEO
Expect Excellence

LATITUDE: 37.31818 LONGITUDE: -121.911215

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 114 ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND WITH GRAVEL (SC), grayish brown, medium dense, wet												
	85														
30			Dense			36	28	19	9						
	80														
35			Very dense			65									
			POORLY GRADED SAND WITH CLAY (SP-SC), grayish brown, very dense, wet, with fine gravel												
	75														
40						65									
			CLAYEY GRAVEL WITH SAND (GC), light reddish brown, very dense, wet												
	70														
45						50/6"									
			CLAYEY SAND WITH GRAVEL (SC), grayish brown, very dense, wet												
	65														
50						50									
			End of boring at 51½ feet below ground surface. Groundwater not measured due to drilling method.												

LOG OF BORING 1-B5

ENGEO
Expect Excellence

LATITUDE: 37.318408 LONGITUDE: -121.913009

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 116½ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 6" AB												
	115		LEAN CLAY (CL), very dark brown, hard, moist, [FILL]			31					22.7	97.5		4.5*	PP
5			Dark reddish brown, hard, moist, with trace fine gravel, [NATIVE]			23	35	17	18	72				4.5*	PP
	110		Yellowish brown, very stiff, moist, with fine-grained sand			11					14.2	107.8	2165		UU
10			Light brown, stiff, with silt			14							500*	4*	PP+TV
	105														
15			LEAN CLAY WITH SILT (CL), reddish brown, very stiff, very moist, with silt			9					21.9	102.8	800*	1.75*	PP+TV
	100														
													800*	1.5*	PP+TV
20															
	95														
25			LEAN CLAY WITH SAND (CL), light brown, stiff, very moist, fine-grained sand			8									
	90														

LOG OF BORING 1-B5

ENGEO Expect Excellence

LATITUDE: 37.318408 LONGITUDE: -121.913009

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 50 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 116½ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			Medium stiff, with trace silt			7					25.7	93.9	611		UU
30															
	85														
			Stiff			6					28.6	90.6	600*	1.25*	PP+TV
35															
	80														
													500*	1.25*	PP+TV
40															
	75														
			LEAN CLAY WITH SILT (CL), light brown, soft, very moist, with trace fine-grained sand			4					25.6	96.1	614		UU
45															
	70														
			LEAN CLAY (CL), light brown, stiff, moist, with trace fine-grained sand			18								2*	PP
50			End of boring at 50 feet below ground surface. Groundwater not measured due to drilling method.												

LOG OF BORING 1-B6

ENGEO
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LATITUDE: 37.317206 LONGITUDE: -121.913766

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 61½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3" AC over 8" AB												
			FAT CLAY (CH), dark brown, very stiff, moist, [NATIVE]												
	115					8	53	24	29		23.1	88.7	1700*	3.25*	PP+TV
5			SANDY LEAN CLAY (CL), dark brown, very stiff, moist			18								4*	PP
	110		Hard, with trace fine gravel			18								4.5*	PP
10			With trace silt												
	105													3*	PP
15			LEAN CLAY (CL), reddish brown, stiff, moist, with silt			14							1000*	1.75*	PP+TV
	100														
20						13					27.1	99.2	1384		UU
	95														
25															

LOG OF BORING 1-B6

ENGEO
Expect Excellence

LATITUDE: 37.317206 LONGITUDE: -121.913766

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 61½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SANDY LEAN CLAY (CL), reddish brown, stiff, moist			18								1.75*	PP
	90														
30			Increased sand content			17					21.3	99.4			
	85					5									
35			SILTY SAND (SM), grayish brown, very dense, wet, with trace fine gravel												
	80					52									
40															
	75		Medium dense, with fine gravel			26									
45															
			CLAYEY GRAVEL (GC), light brown, medium dense, wet												
	70														
50															

LOG OF BORING 1-B6

ENGEO
Expect Excellence

LATITUDE: 37.317206 LONGITUDE: -121.913766

Geotechnical Exploration
Avenues Silicon Valley
San Jose, California
15929.000.000

DATE DRILLED: 4/10/2019
HOLE DEPTH: Approx. 61½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 118¼ ft.

LOGGED / REVIEWED BY: J. Bauer / TB
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: HSA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY GRAVEL (GC), light brown, medium dense, wet			10									
	65		SILTY SAND (SM), light brown, dense, wet, with trace fine gravel												
55						40									
	60														
60						46									
			End of boring at 61½ feet below ground surface. Groundwater not measured due to drilling method.												

APPENDIX B

CONE PENETRATION TEST LOGS

PRESENTATION OF SITE INVESTIGATION RESULTS

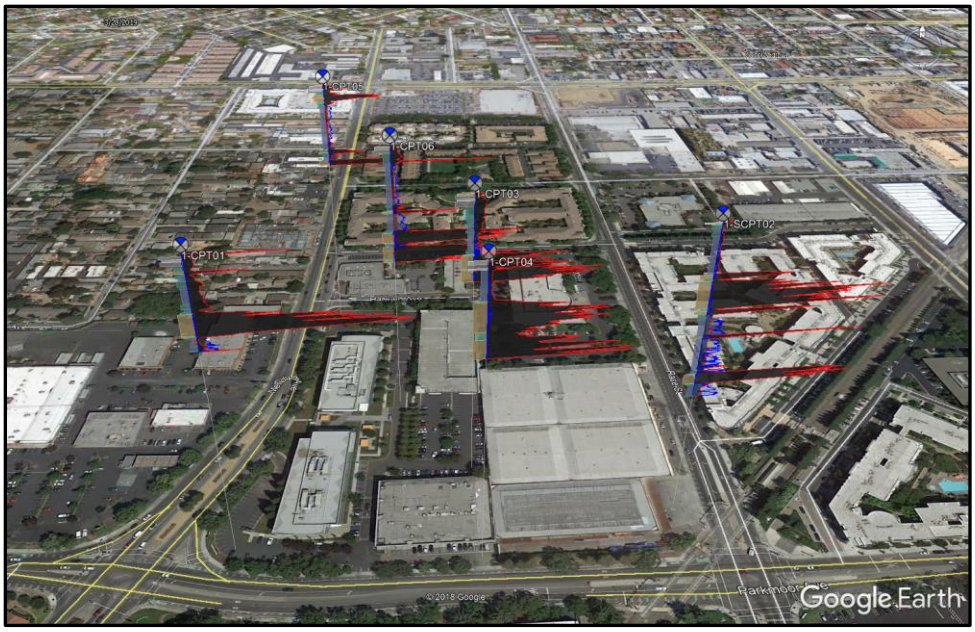
Avenues Silicon Valley

Prepared for:

ENGEO Inc.

ConeTec Inc. Job No: 19-56044

Project Start Date: 29-Mar-2019
Project End Date: 29-Mar-2019
Report Date: 02-Apr-2019



Prepared by:

ConeTec Inc.
820 Aladdin Avenue
San Leandro, CA 94577

Tel: (510) 357-3677

Email: ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com

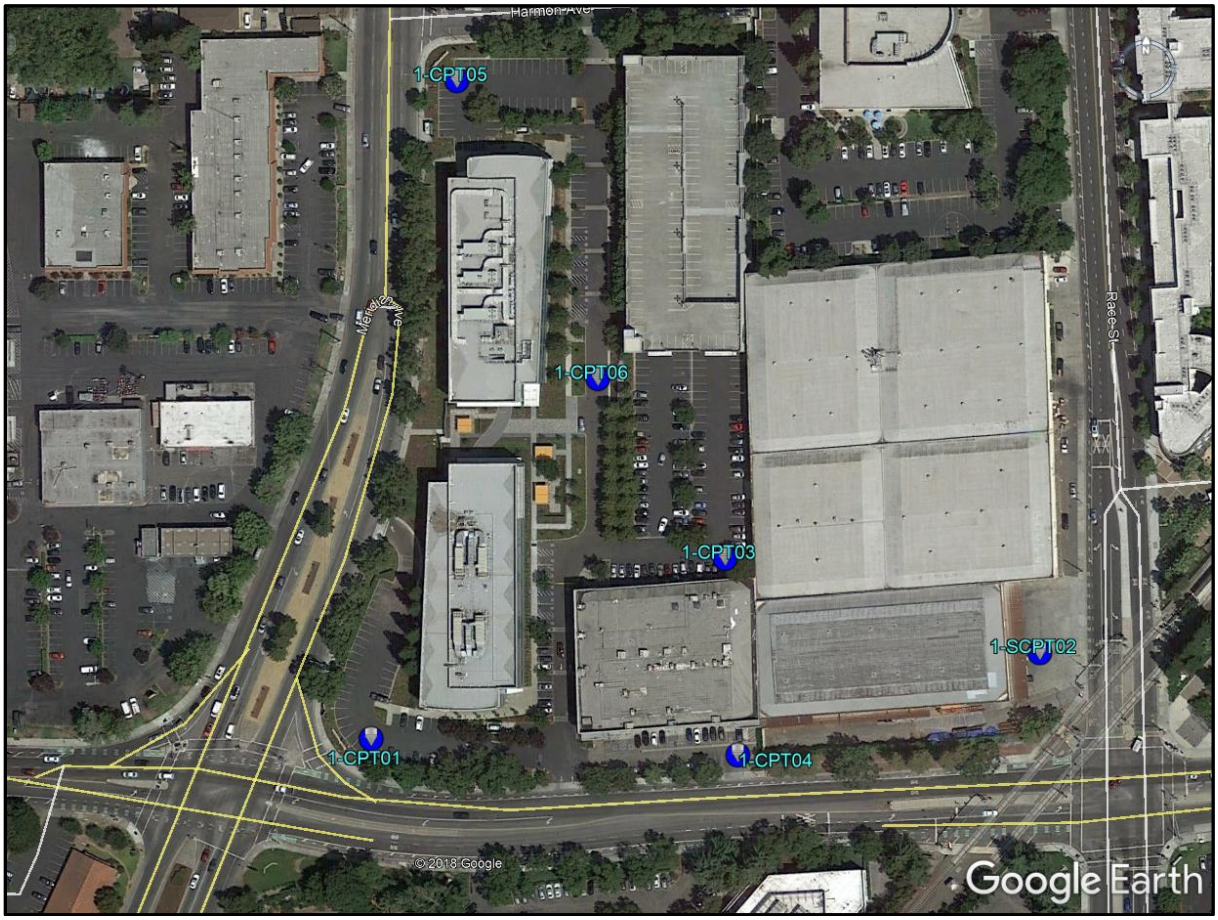
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for ENGEO Inc. at Avenues Silicon Valley, San Jose, CA. The program consisted of five cone penetration tests (CPT) and one seismic cone penetration test (SCPT).

Project Information

Project	
Client	ENGEO Inc.
Project	Avenues Silicon Valley
ConeTec project number	19-56044

An image from Google Earth including the CPT and SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C15)	30 ton rig cylinder	CPT, SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT, SCPT	Consumer-grade GPS	32610

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Seismic plot, Standard-Expanded range plots, Advanced CPT plots with I_c, Su(N_{kt}), Phi and $N_{1(60)}$(IcRW1998) as well as SBT Scatter plots are provided in the release package.

Cone Penetrometers 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 (psi)
447:T1500F15U500	447	15	225	1500	15	500
Cone AD447 was used for all the soundings.						

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

Limitations

This report has been prepared for the exclusive use of ENGEO Inc. (Client) for the project titled "Avenues Silicon Valley". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

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 a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface 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 presented in the first Appendix. 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 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm 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 6 mm 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.

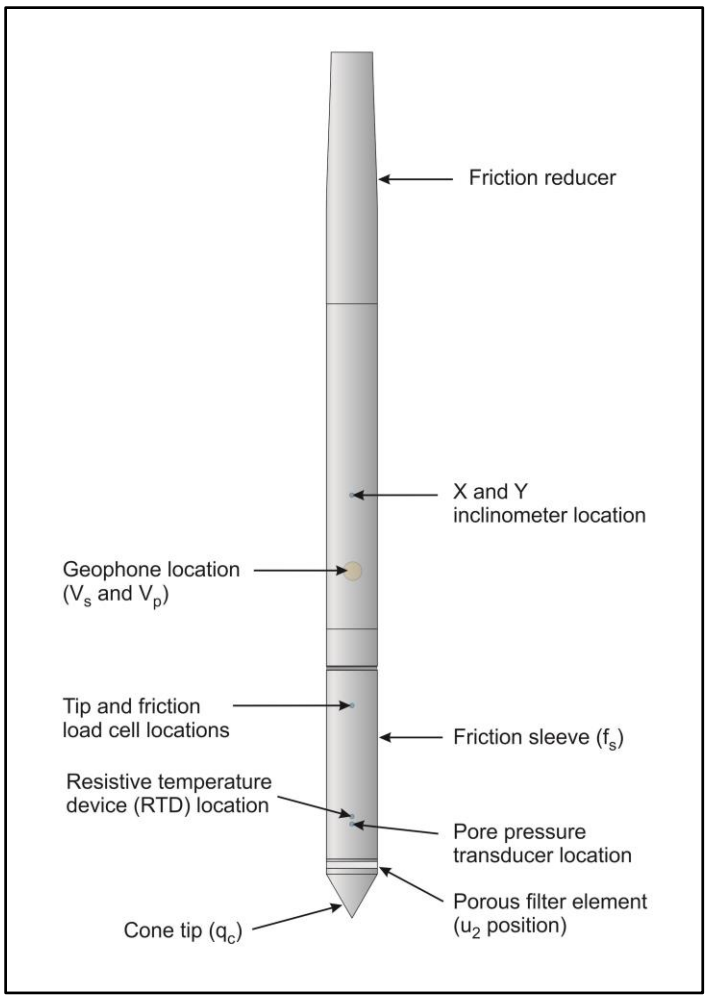


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; 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 CPT 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 either glycerine or 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 2 cm/s, 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 or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- 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 (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

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:

$$q_t = q_c + (1-a) \bullet u_2$$

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.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

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

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

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

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 maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

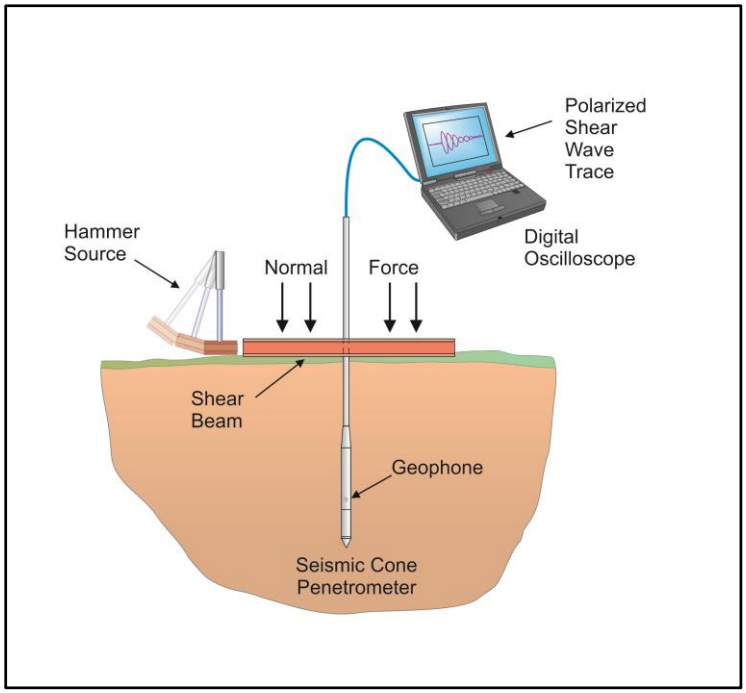


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

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. Multiple wave traces are recorded for quality control 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).

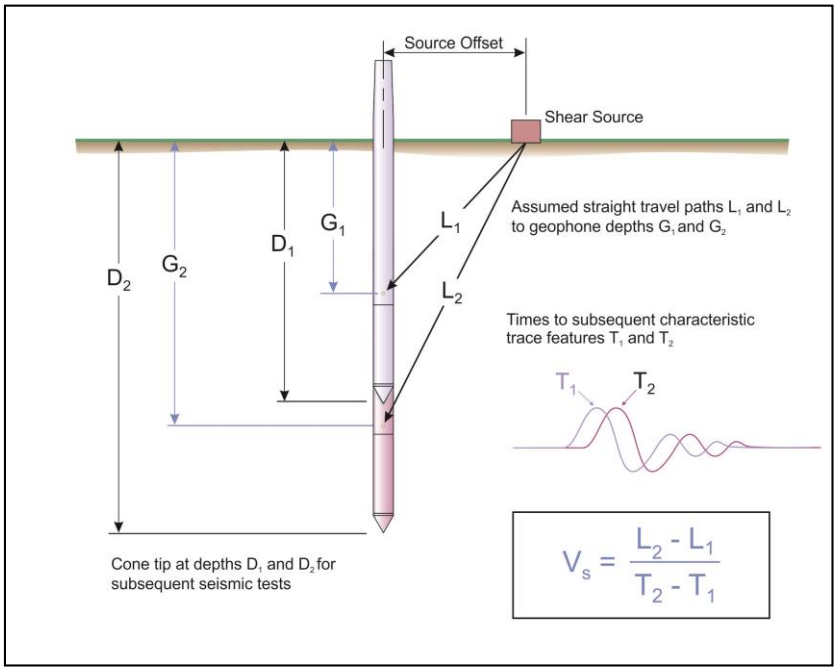


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ (30m)}{\sum(layer\ traveltimes)}$$

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

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

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).

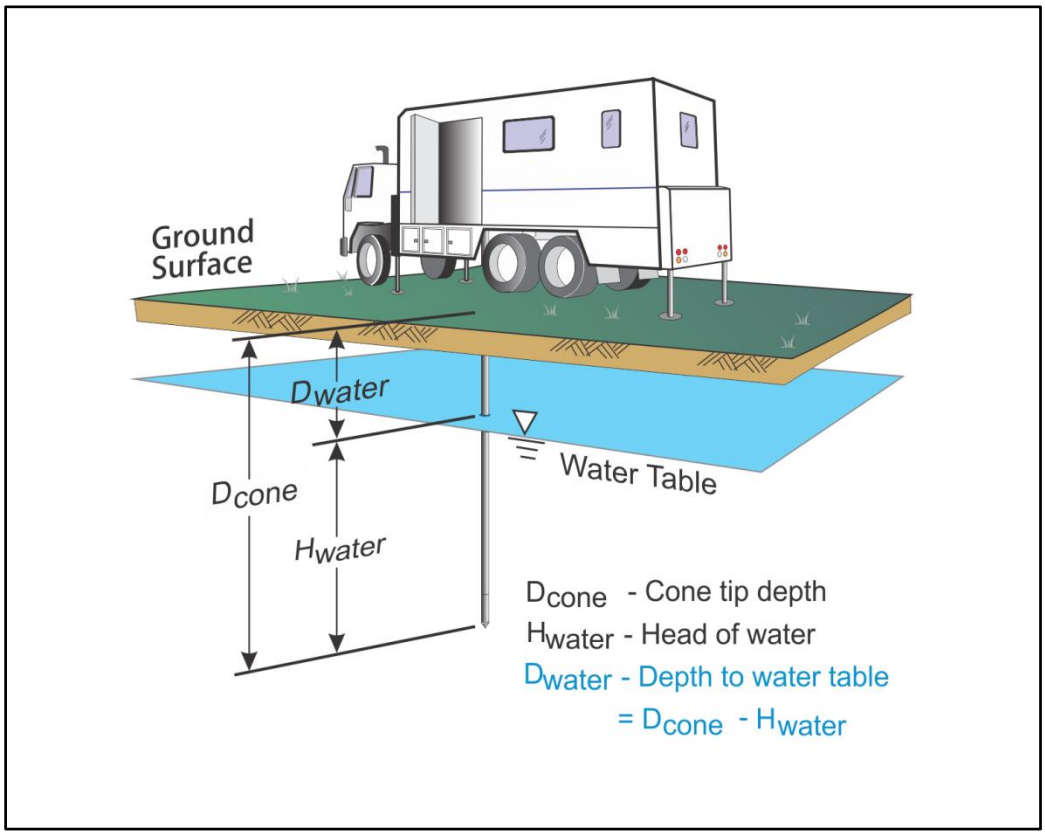


Figure PPD-1. Pore pressure dissipation test setup

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

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.

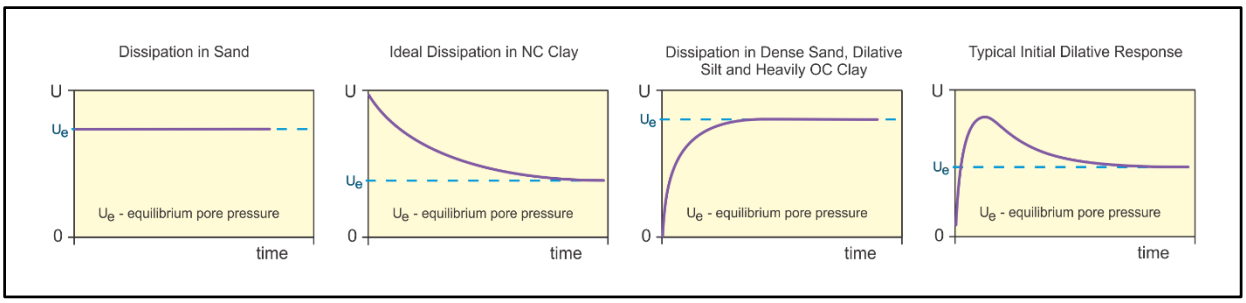


Figure PPD-2. Pore pressure dissipation curve examples

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 of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, “National shear wave measurement guidelines for Canadian seismic site assessment”, GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

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

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Cone Penetration Test Standard Plots – Expanded Range
- Advanced Cone Penetration Test Plots with I_c, Su(N_{kt}), Phi and $N_{1(60)}$ (IcRW1998)
- Seismic Cone Penetration Test Tabular Result
- Seismic Cone Penetration Test Plot
- Seismic Cone Penetration Test Time Domain Traces
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

CONETEC

Job No:	19-56044
Client:	ENGEO Inc.
Project:	Avenues Silicon Valley
Start Date:	29-Mar-2019
End Date:	29-Mar-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
1-CPT01	19-56044_CP01	29-Mar-2019	447:T1500F15U500	34.4	52.00	4130568	596244	
1-SCPT02	19-56044_SP02	29-Mar-2019	447:T1500F15U500	32.0	101.05	4130599	596466	3
1-CPT03	19-56044_CP03	29-Mar-2019	447:T1500F15U500	33.2	52.00	4130629	596361	
1-CPT04	19-56044_CP04	29-Mar-2019	447:T1500F15U500	33.4	51.92	4130564	596366	
1-CPT05	19-56044_CP05	29-Mar-2019	447:T1500F15U500	32.1	57.25	4130786	596270	
1-CPT06	19-56044_CP06	29-Mar-2019	447:T1500F15U500	34.1	75.21	4130688	596318	

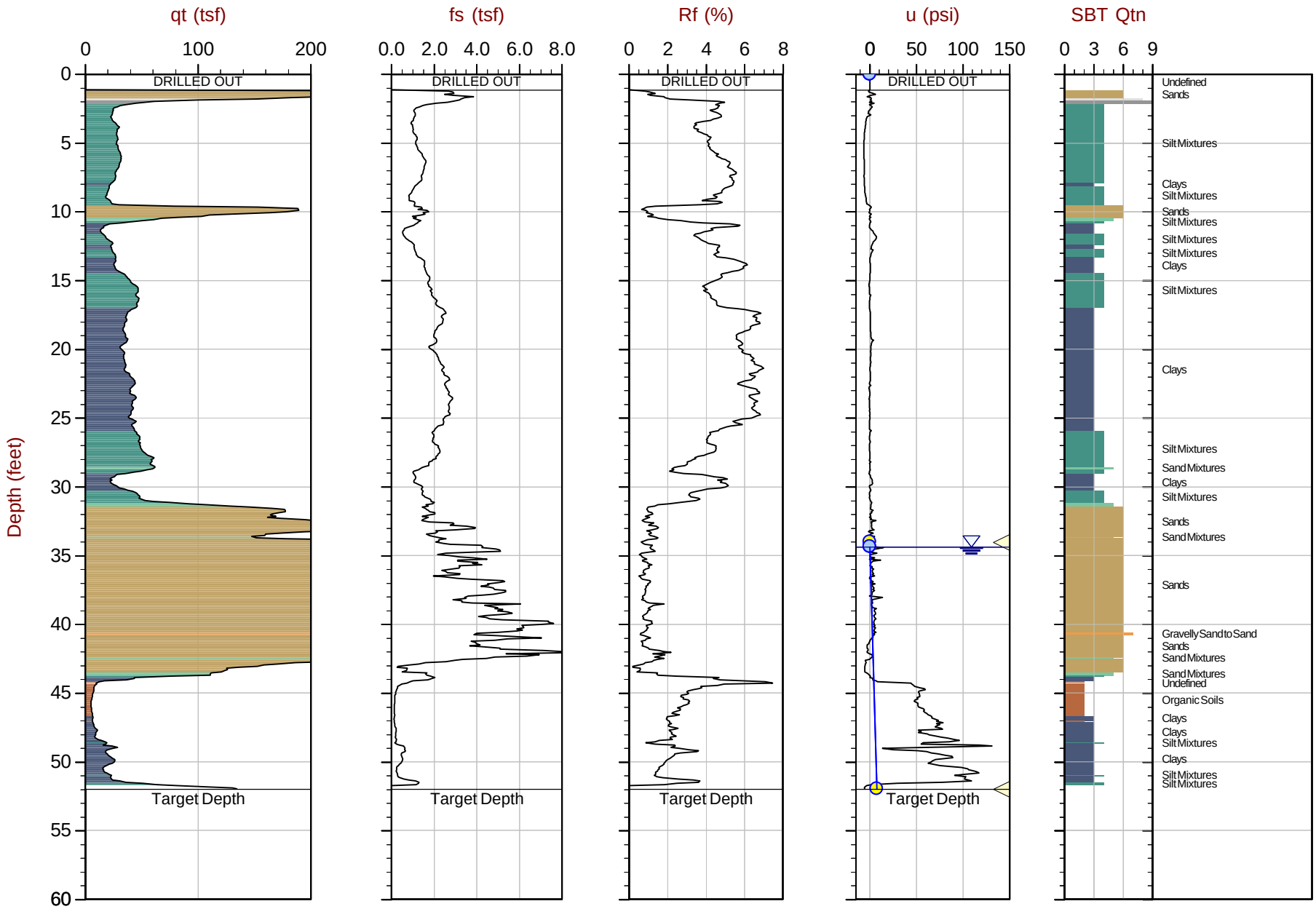
1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. The assumed phreatic surface was based on the dynamic pore pressure response.

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 11:28
Site: Avenues Silicon Valley

Sounding: 1-CPT01
Cone: 447:T1500F15U500



Max Depth: 15.850 m / 52.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56044_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130568m E: 596244m
Page No: 1 of 1

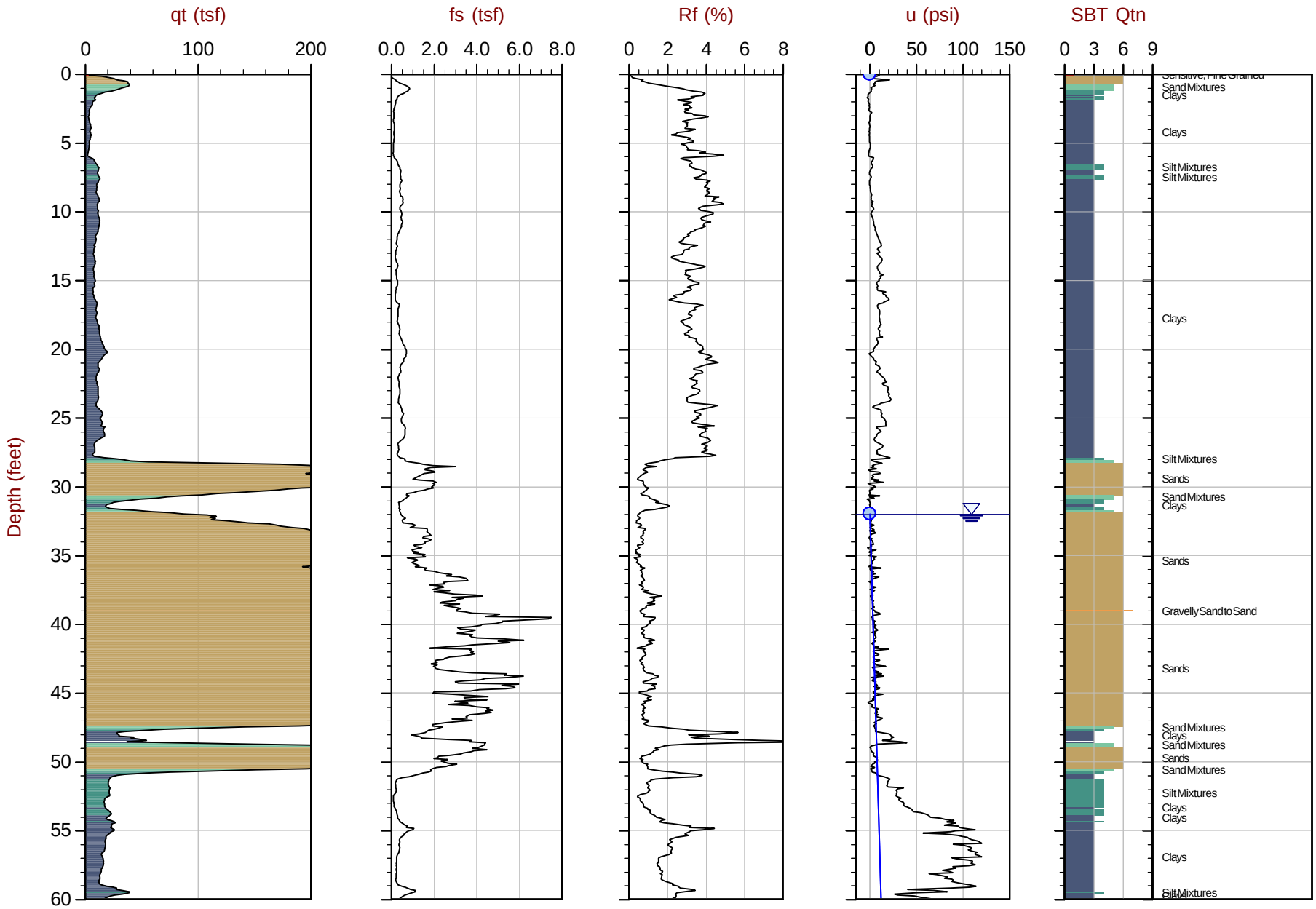
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



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

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 1 of 2

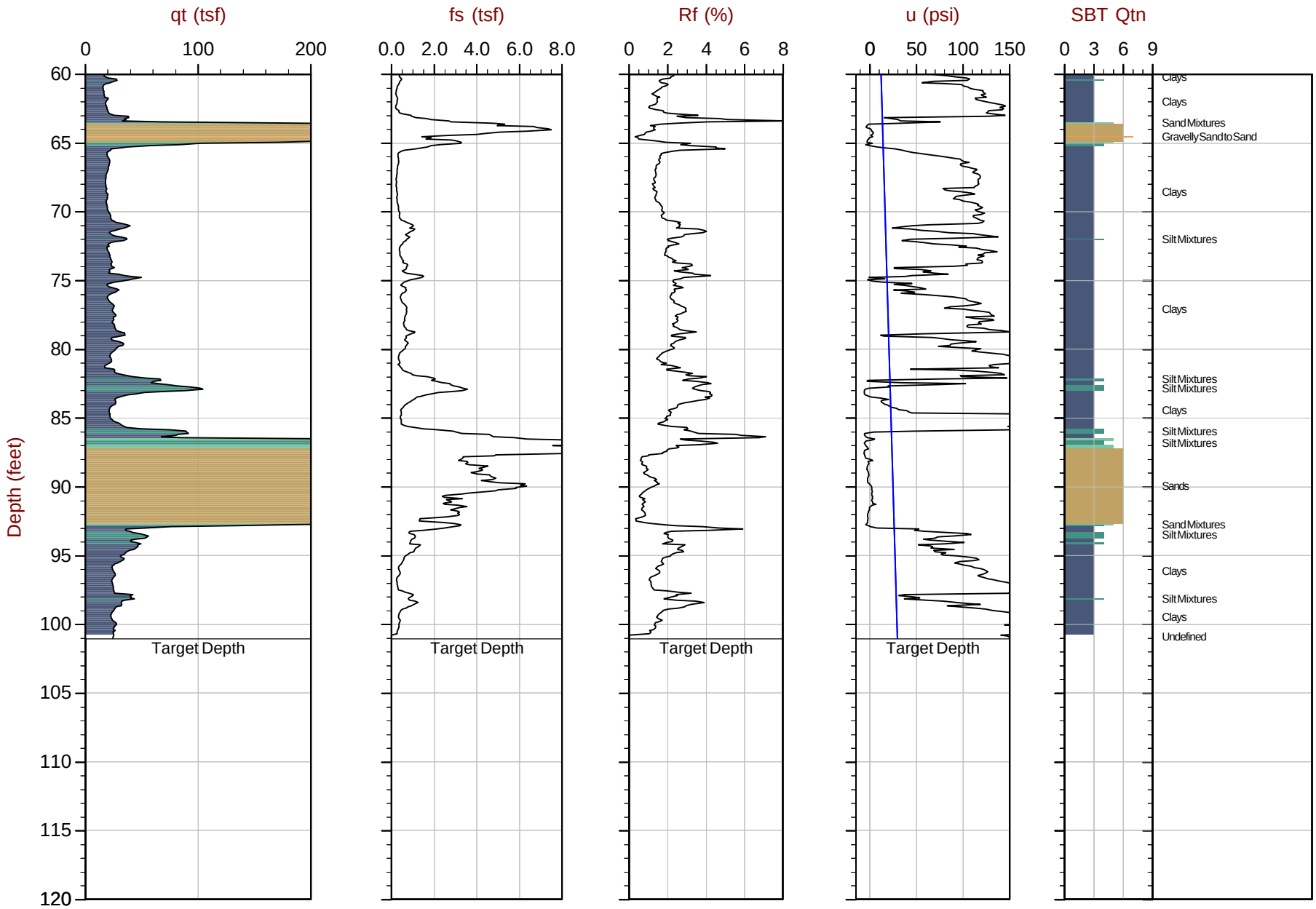
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



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

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 2 of 2

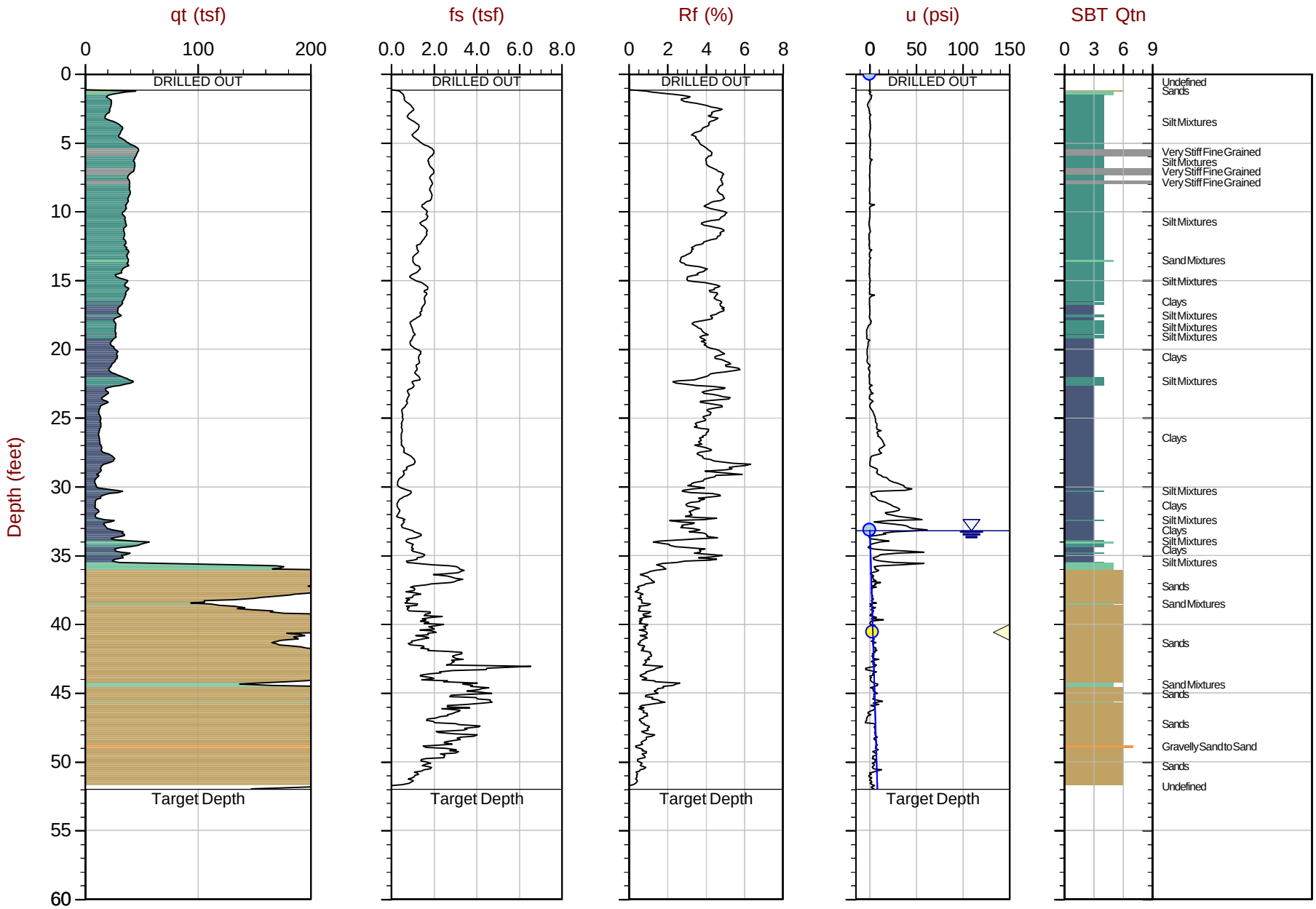
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 10:14
Site: Avenues Silicon Valley

Sounding: 1-CPT03
Cone: 447:T1500F15U500



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Avg Int: Every Point

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Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130629m E: 596361m
Page No: 1 of 1

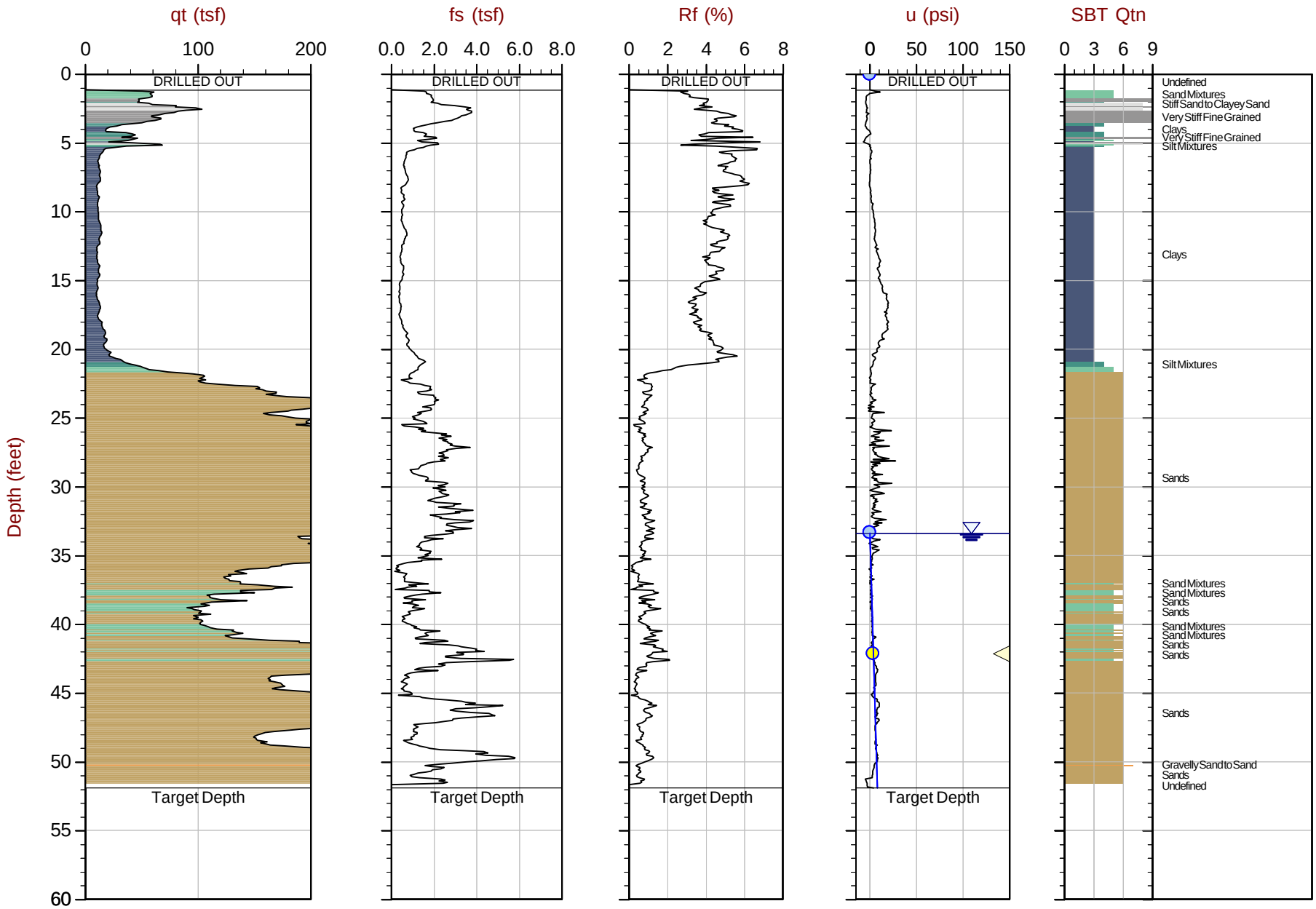
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 14:17
Site: Avenues Silicon Valley

Sounding: 1-CPT04
Cone: 447:T1500F15U500



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

File: 19-56044_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130564m E: 596366m
Page No: 1 of 1

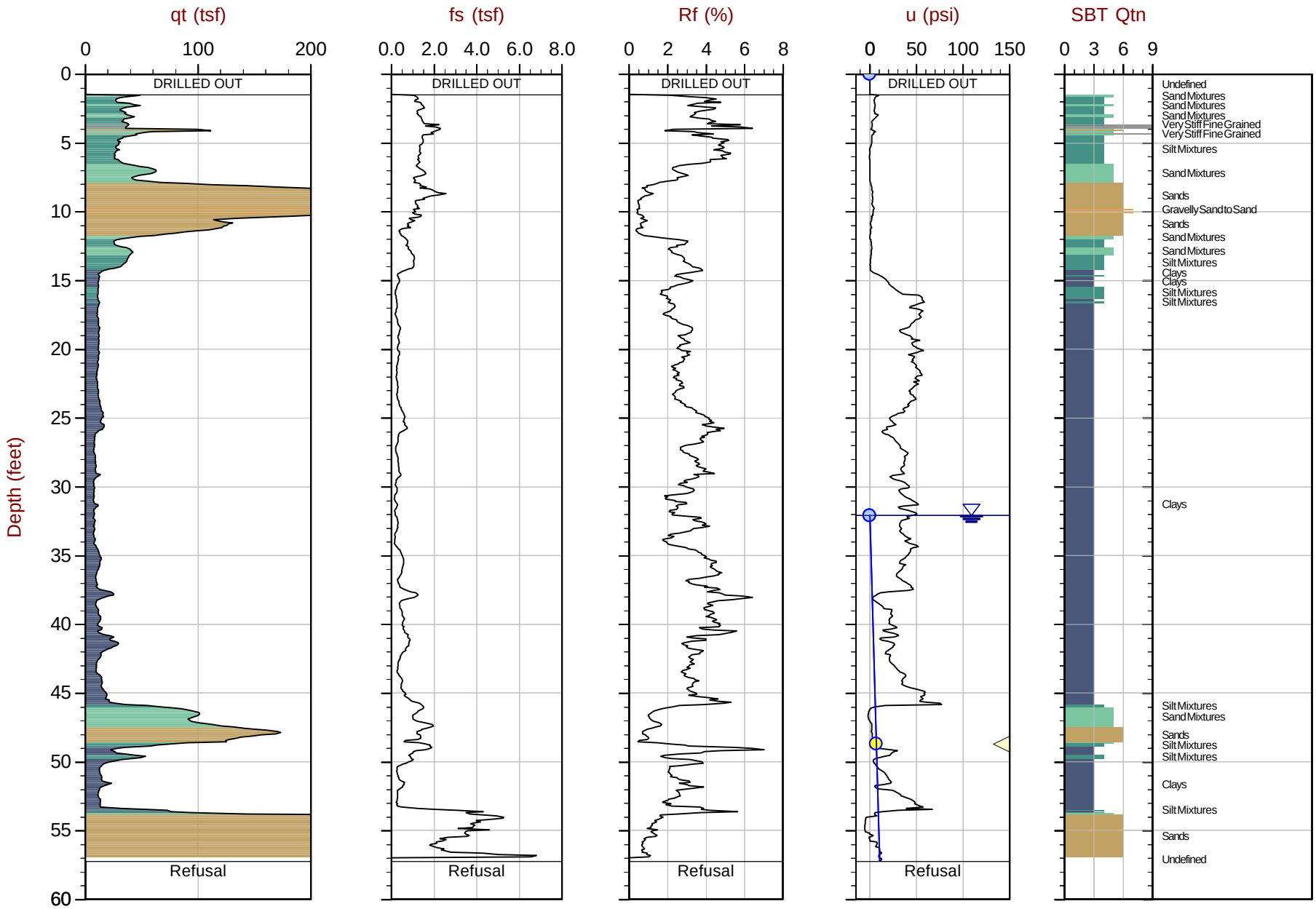
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 07:38
Site: Avenues Silicon Valley

Sounding: 1-CPT05
Cone: 447:T1500F15U500



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

File: 19-56044_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130786m E: 596270m
Page No: 1 of 1

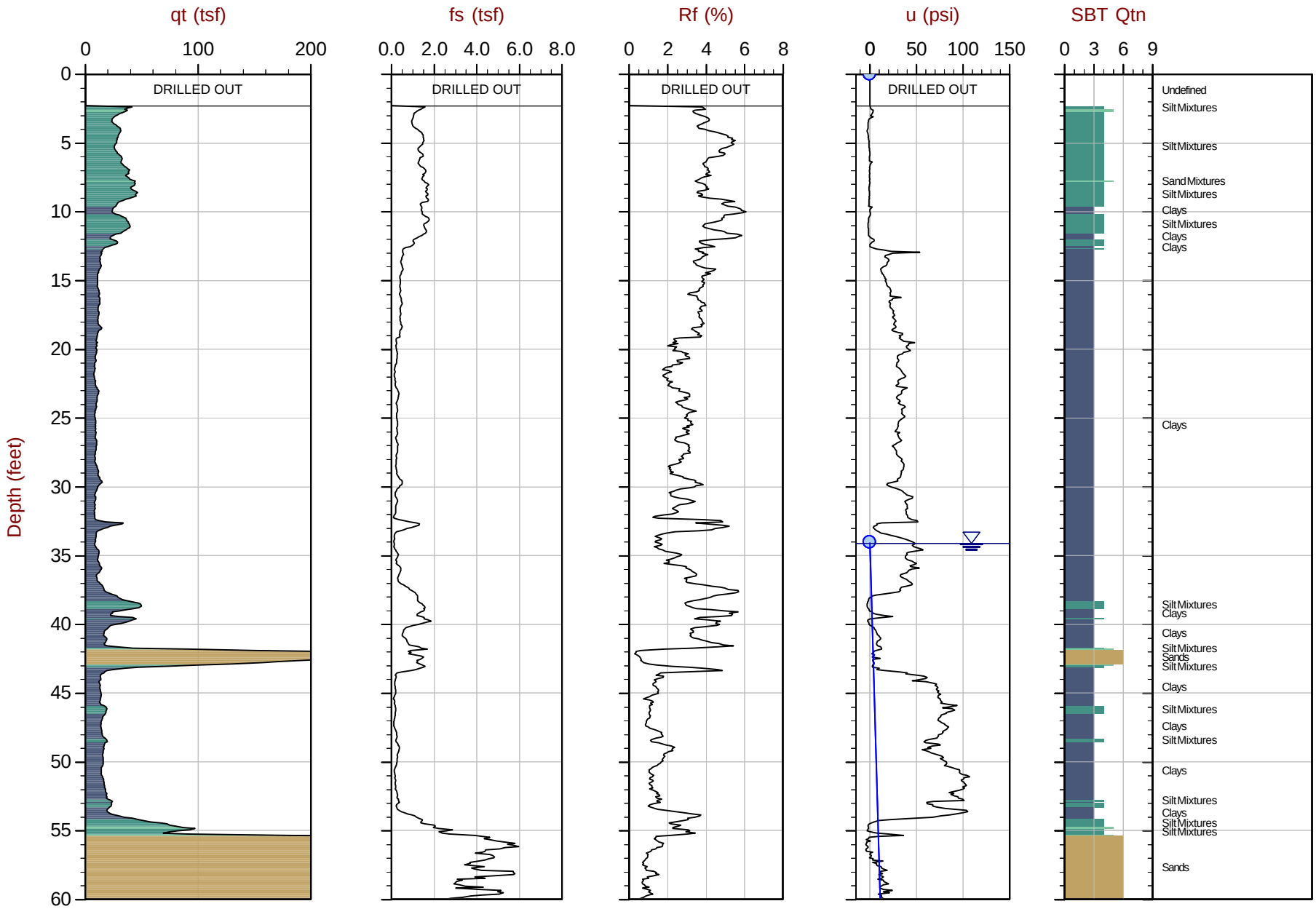
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500



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

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 1 of 2

Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
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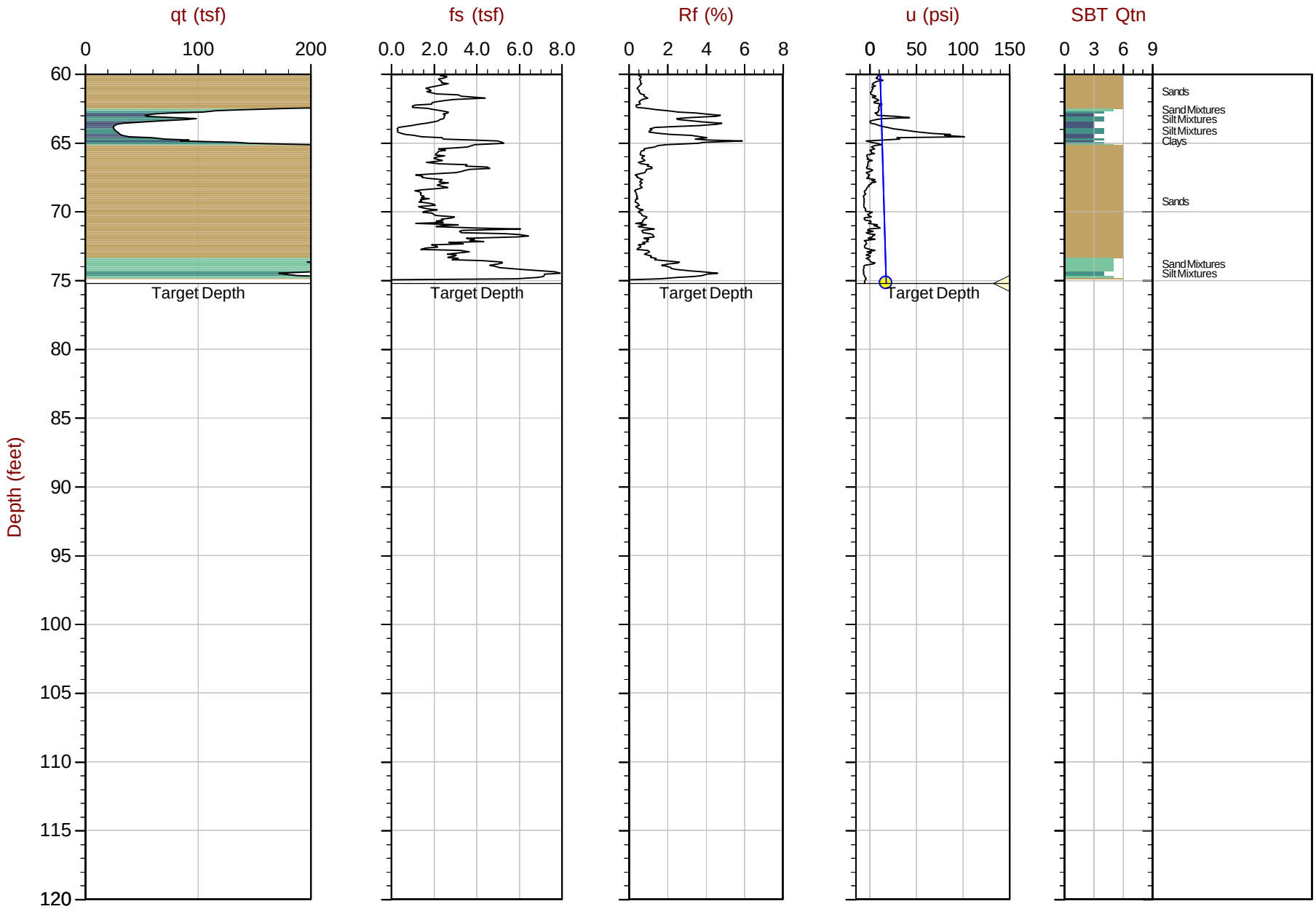

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 2 of 2

Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

Cone Penetration Test Standard Plots – Expanded Range

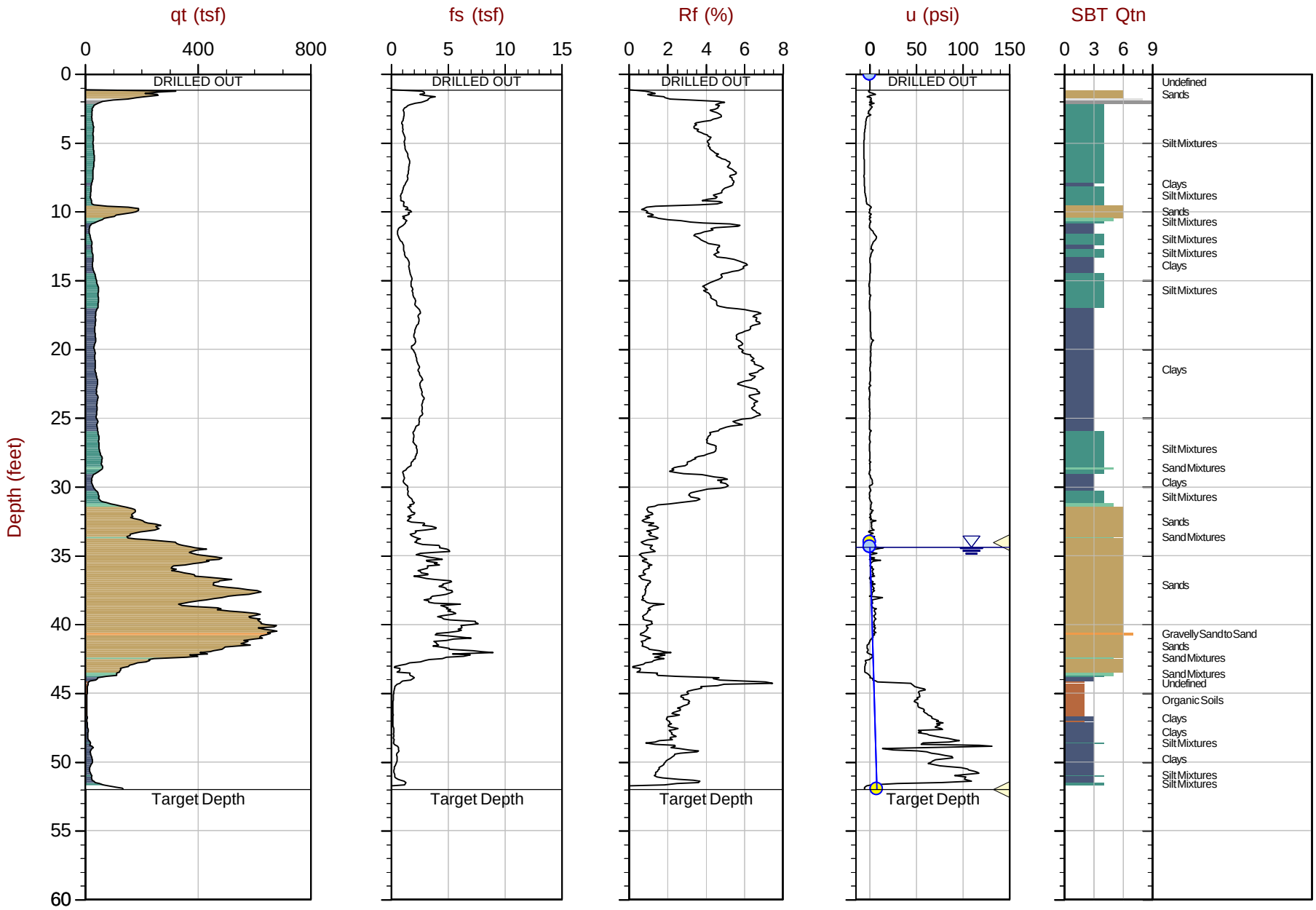
CONETEC

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 11:28
Site: Avenues Silicon Valley

Sounding: 1-CPT01
Cone: 447:T1500F15U500



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Avg Int: Every Point

File: 19-56044_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130568m E: 596244m
Page No: 1 of 1

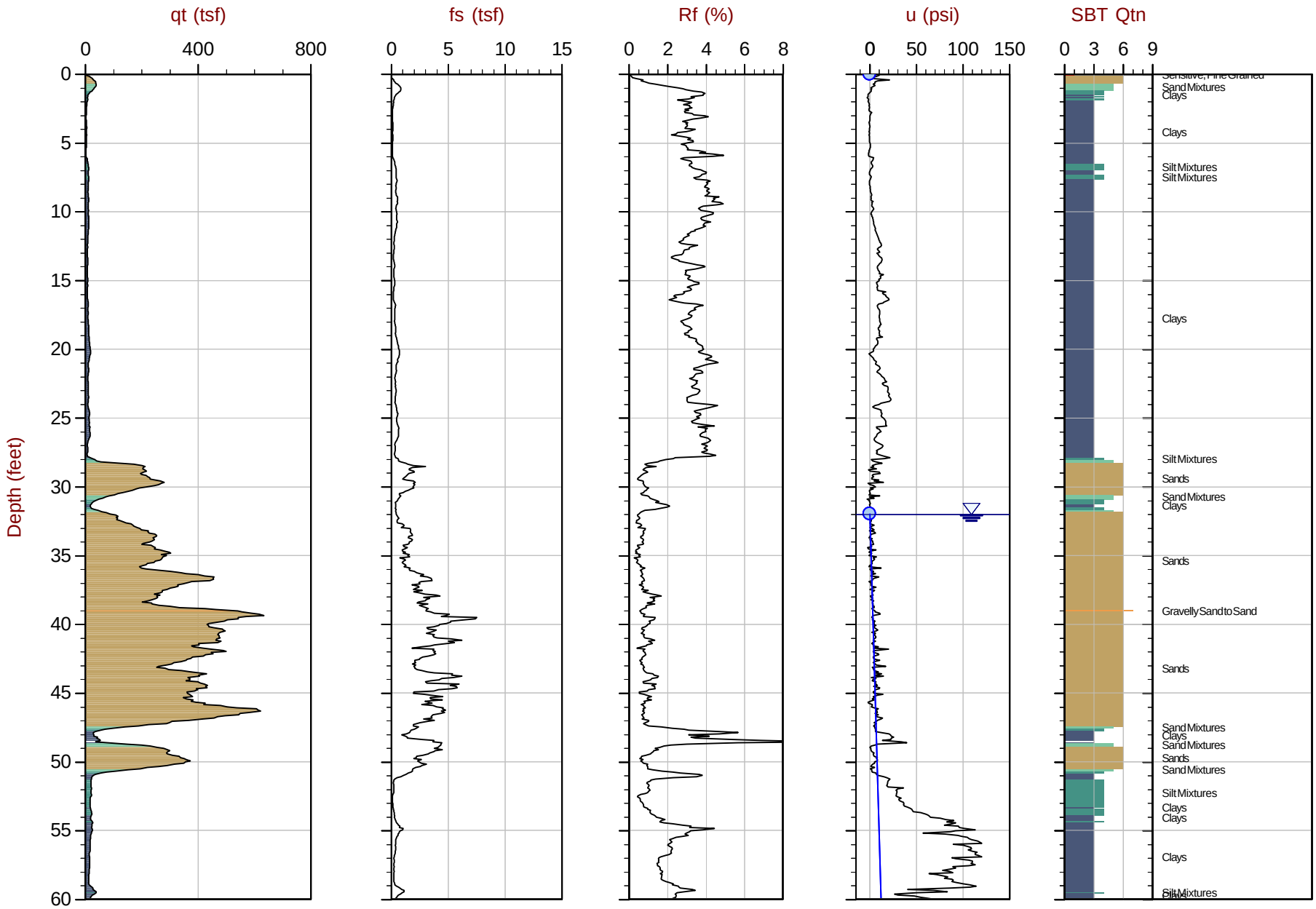
Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



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

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 1 of 2

Overplot Item:
- Assumed Ueq
- Ueq
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CONETEC

ENGEO

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Sounding: 1-SCPT02
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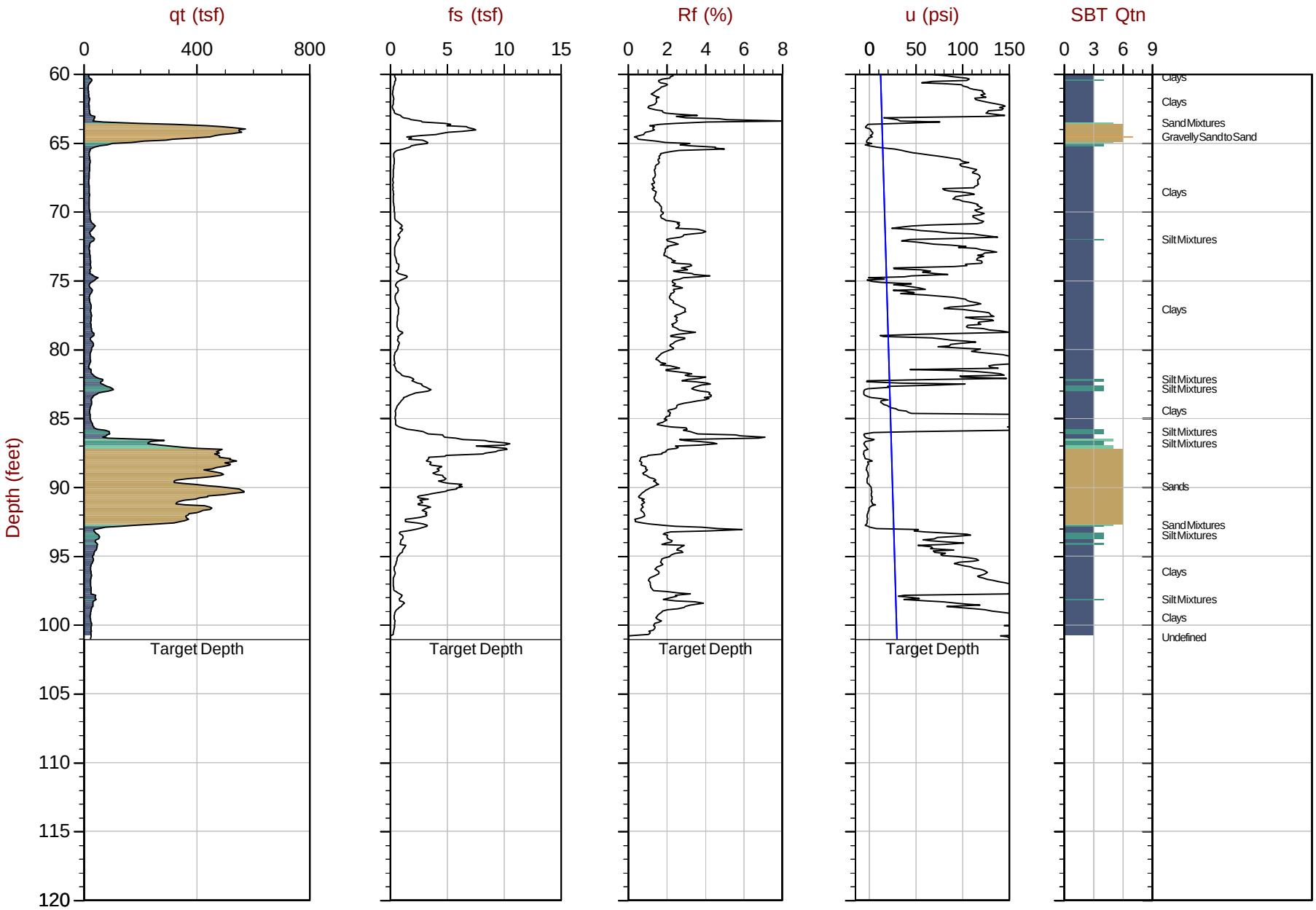

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 2 of 2

Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 10:14
Site: Avenues Silicon Valley

Sounding: 1-CPT03
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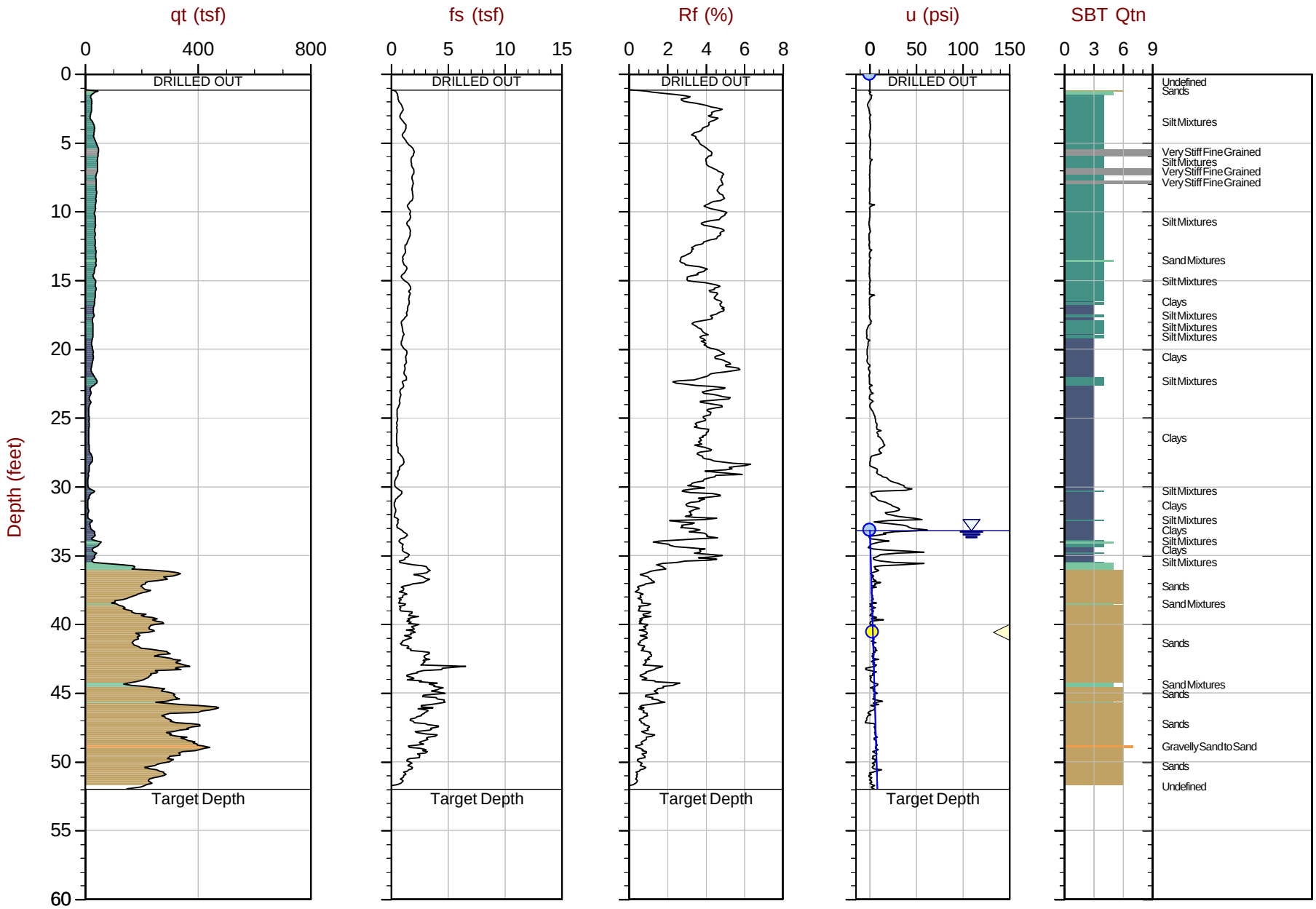

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56044_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130629m E: 596361m
Page No: 1 of 1

Overplot Item:
- Assumed Ueq
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- Dissipation, equilibrium achieved
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CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 14:17
Site: Avenues Silicon Valley

Sounding: 1-CPT04
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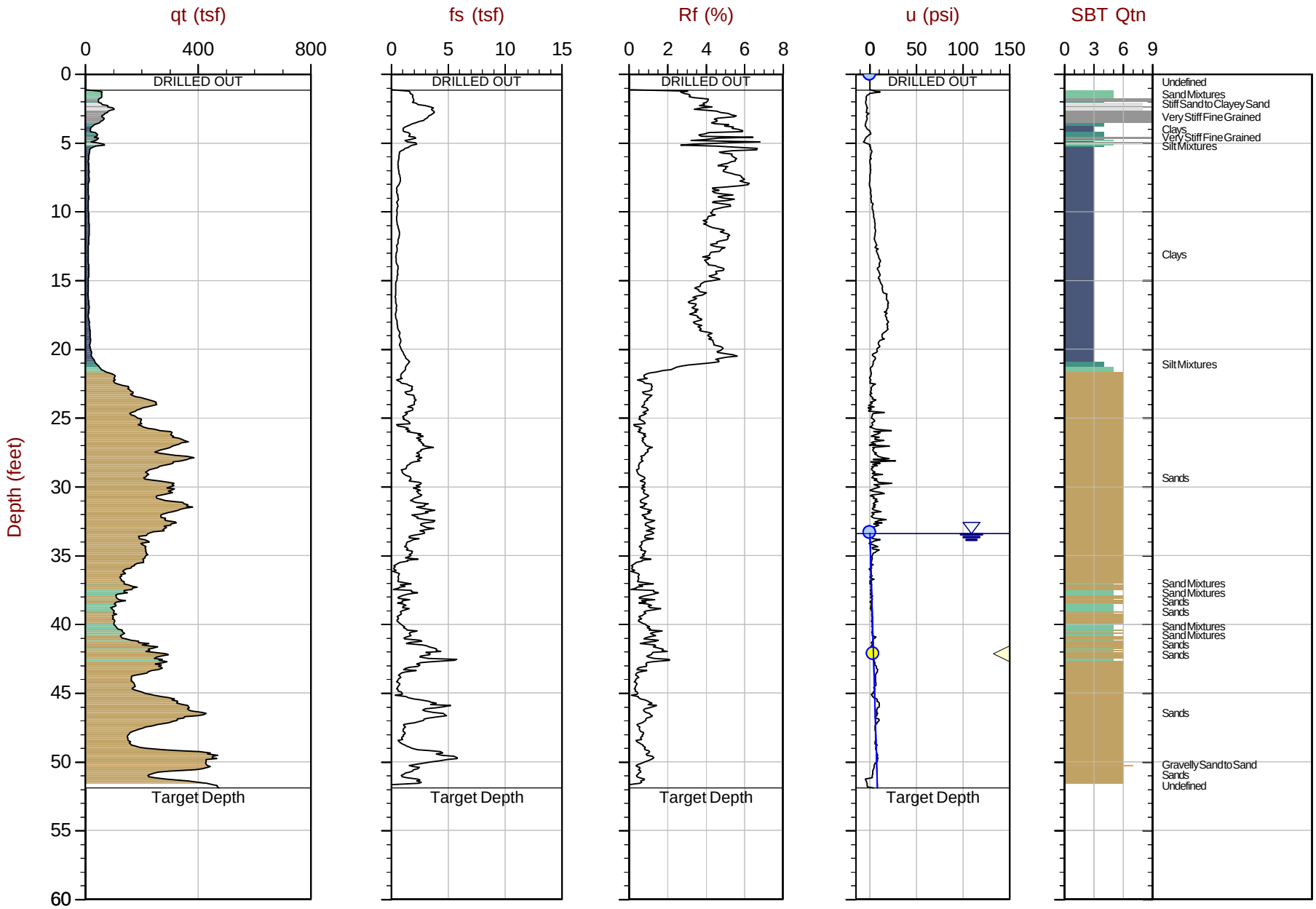

Max Depth: 15.825 m / 51.92 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56044_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130564m E: 596366m
Page No: 1 of 1

Overplot Item:
- Assumed Ueq
- Ueq
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- Dissipation, equilibrium not achieved
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CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 07:38
Site: Avenues Silicon Valley

Sounding: 1-CPT05
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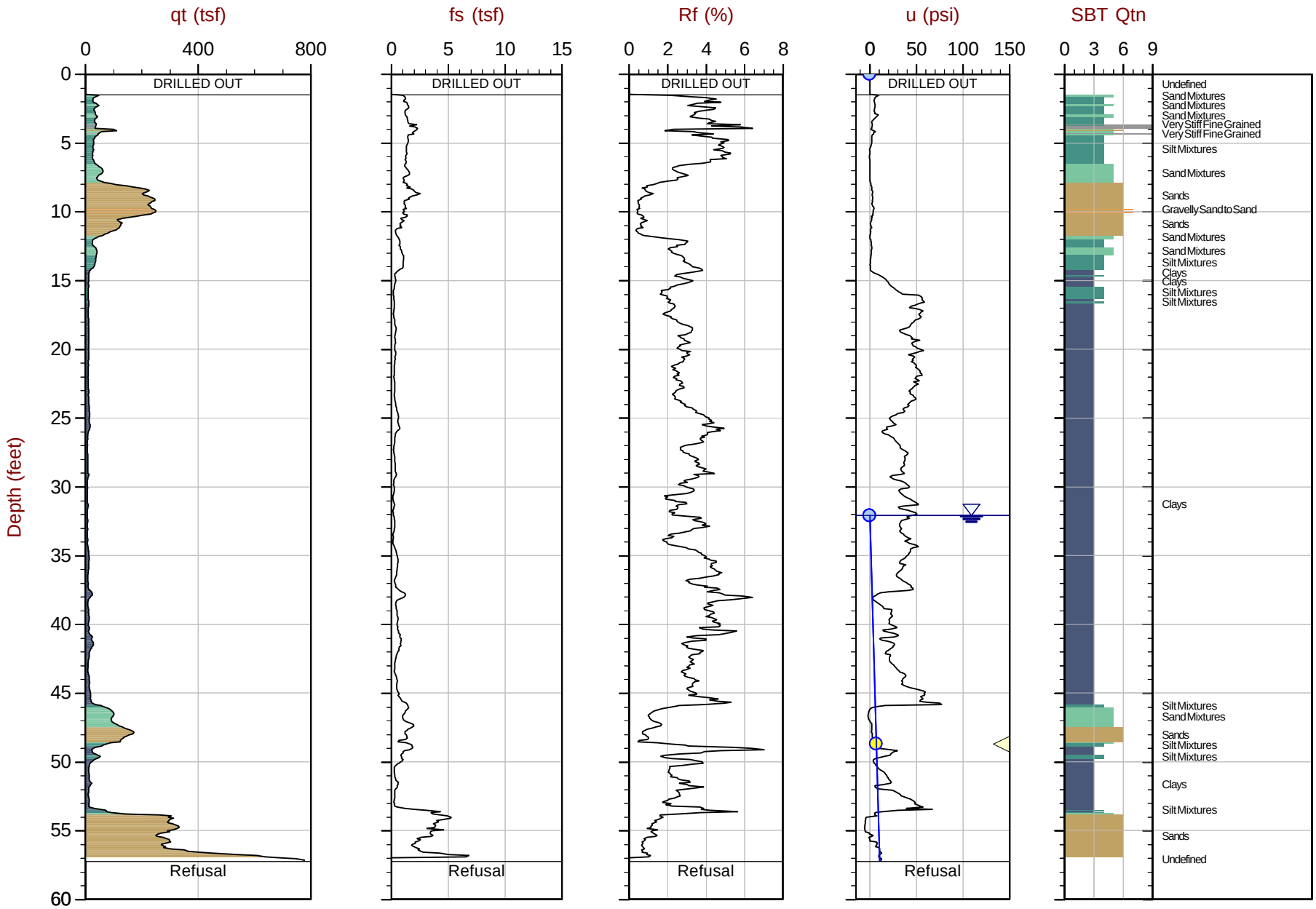

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130786m E: 596270m
Page No: 1 of 1

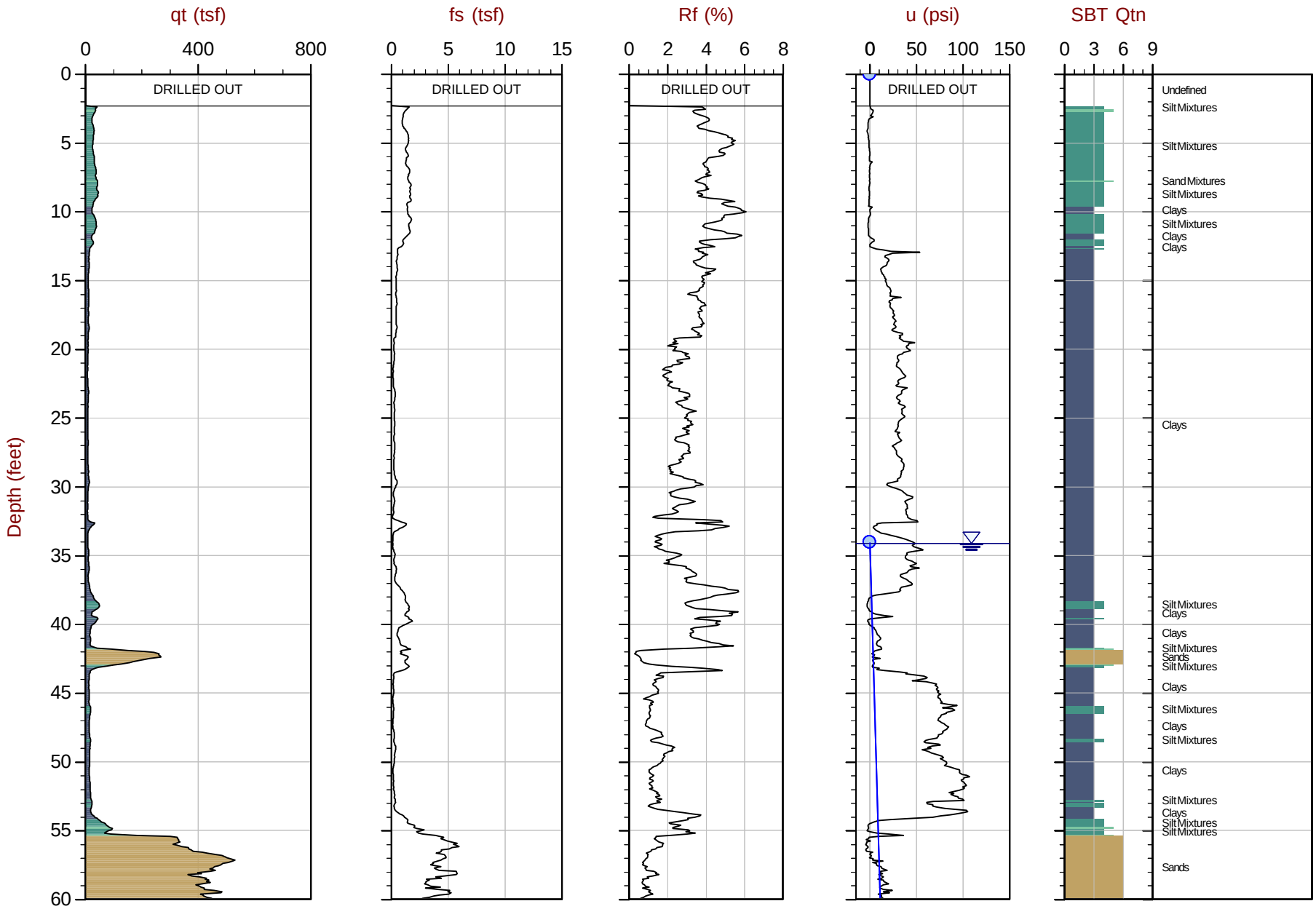
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500



Max Depth: 22.925 m / 75.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 1 of 2

Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
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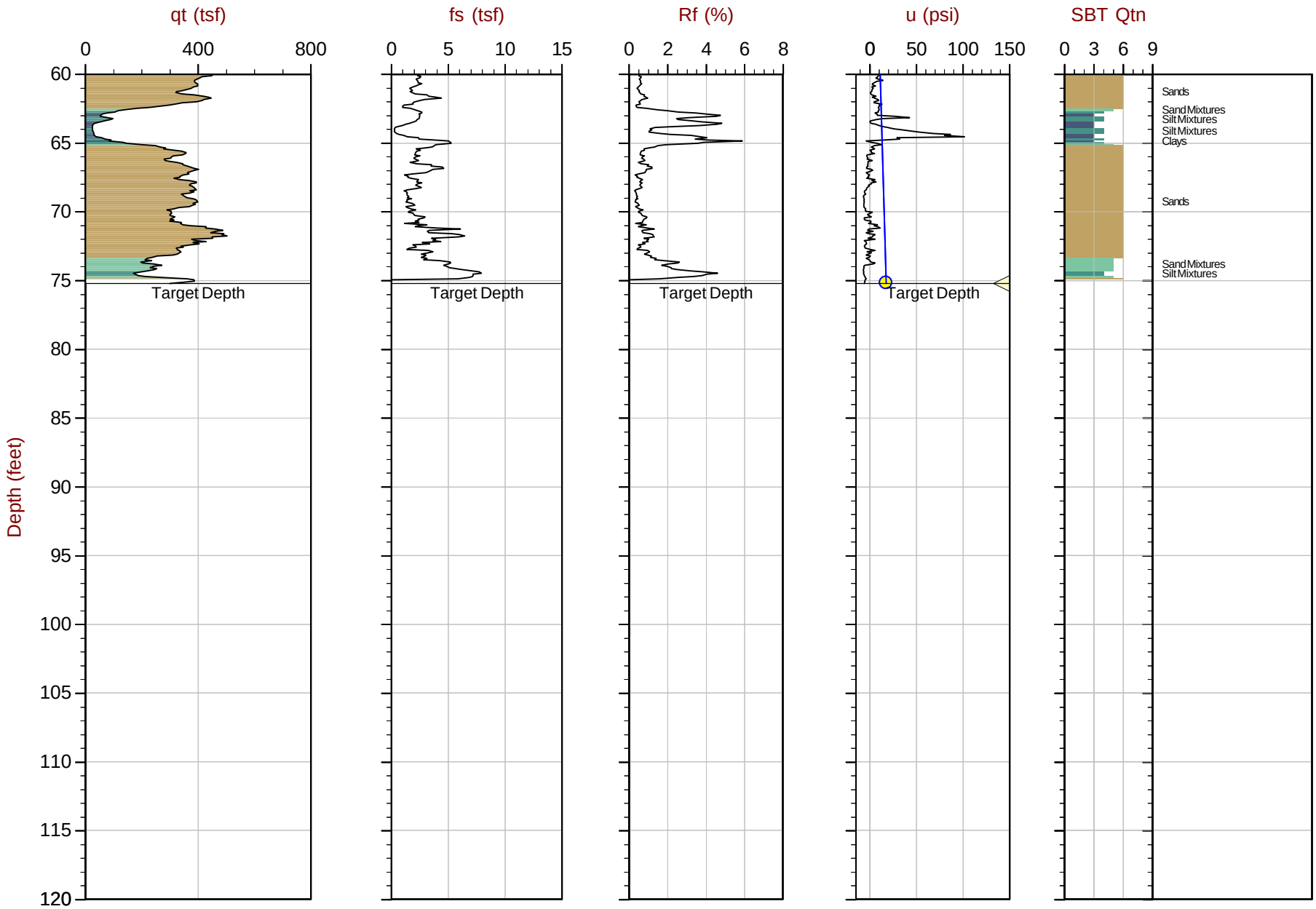

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 2 of 2

Overplot Item:
- Assumed Ueq
- Ueq
- Dissipation, equilibrium achieved
- Dissipation, equilibrium not achieved
- Hydrostatic Line

Advanced Cone Penetration Test Plots with I_c, Su(N_{kt}), Phi and $N_{1(60)}$(Ic RW1998)

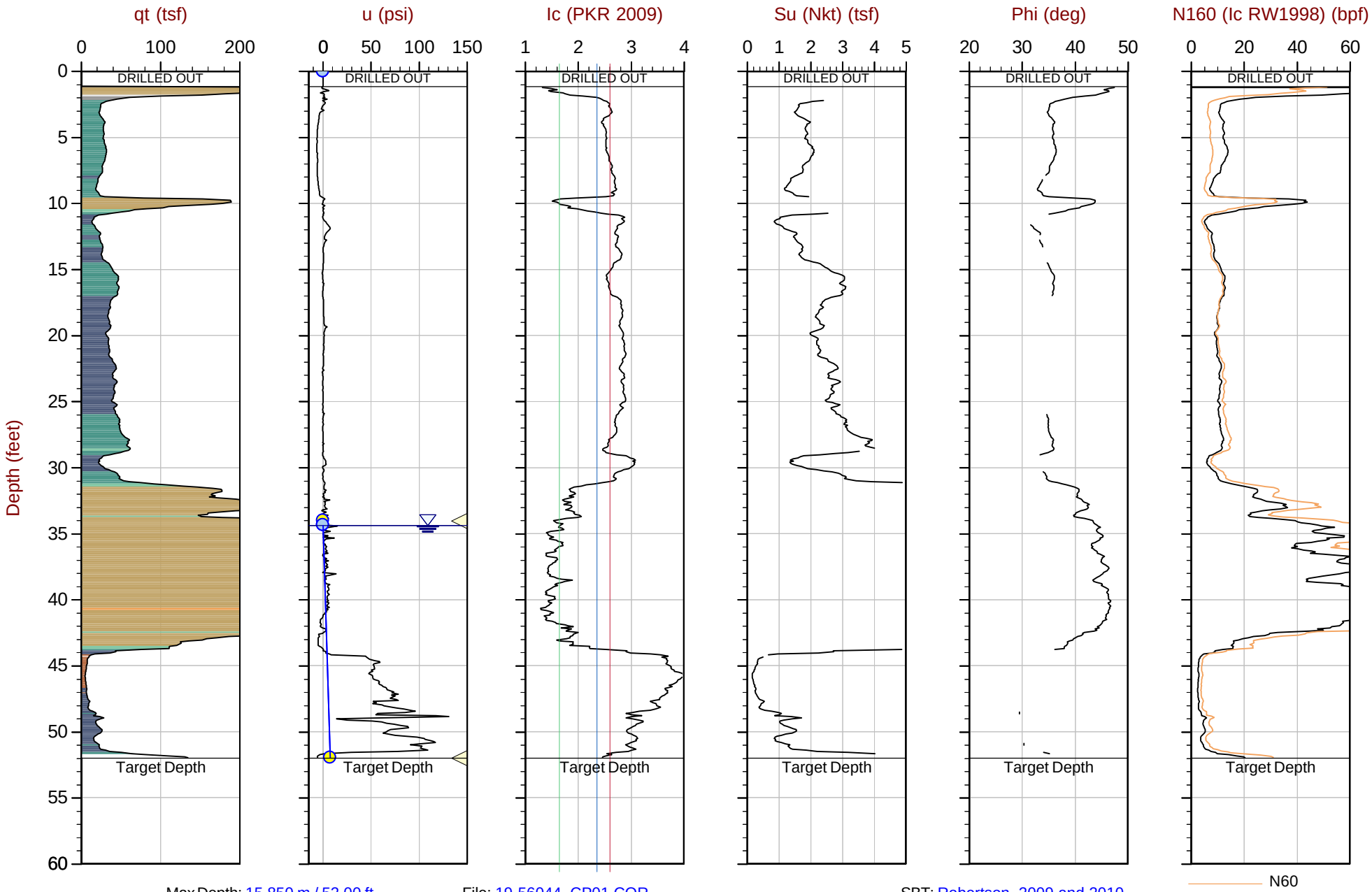
CONETEC

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 11:28
Site: Avenues Silicon Valley

Sounding: 1-CPT01
Cone: 447:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:
Assumed Ueq
Ueq

File: 19-56044_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130568m E: 596244m
Page No: 1 of 1

CONETEC

ENGEO

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Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

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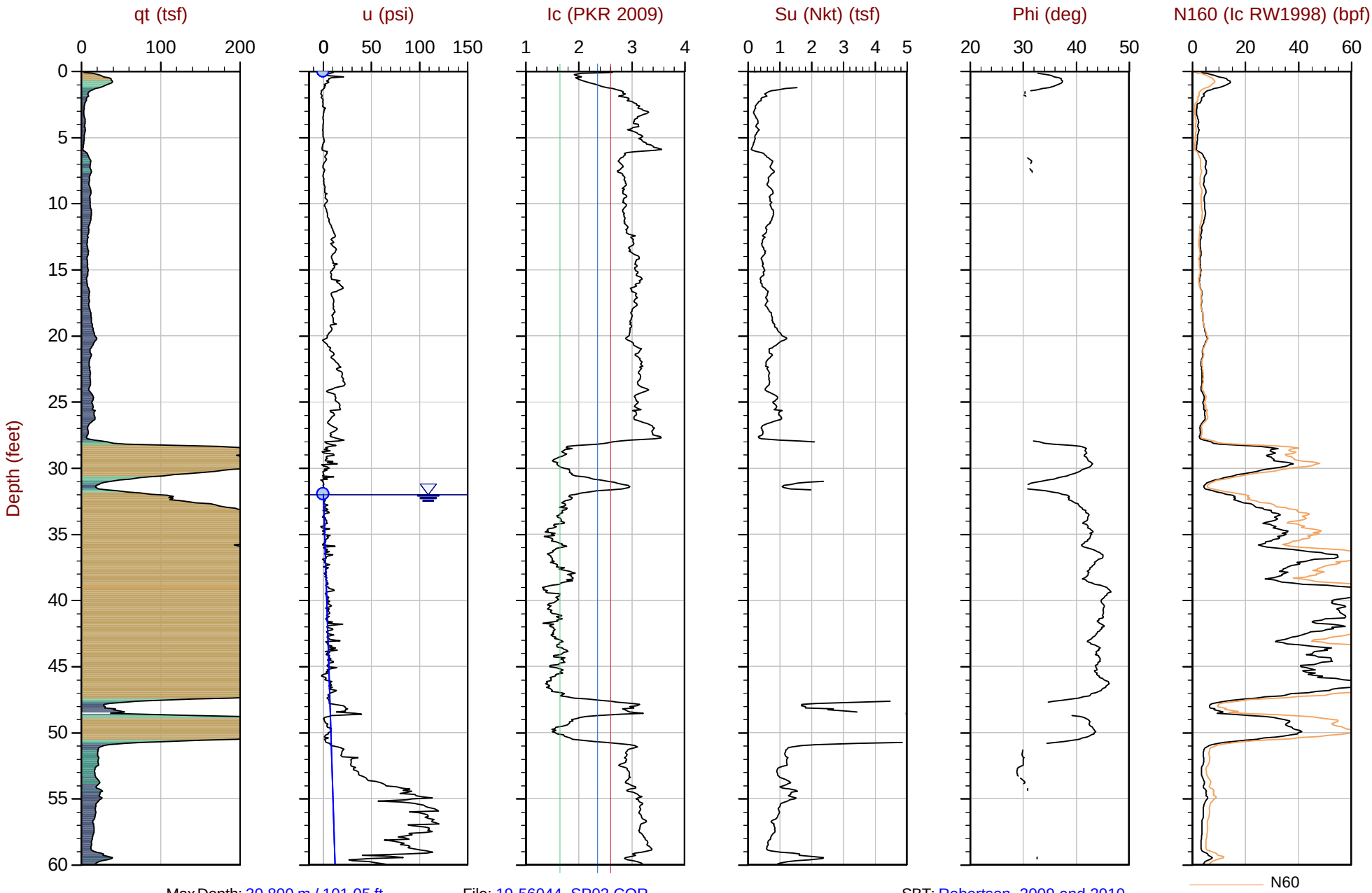

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Avg Int: Every Point
Overplot Item:

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 1 of 2

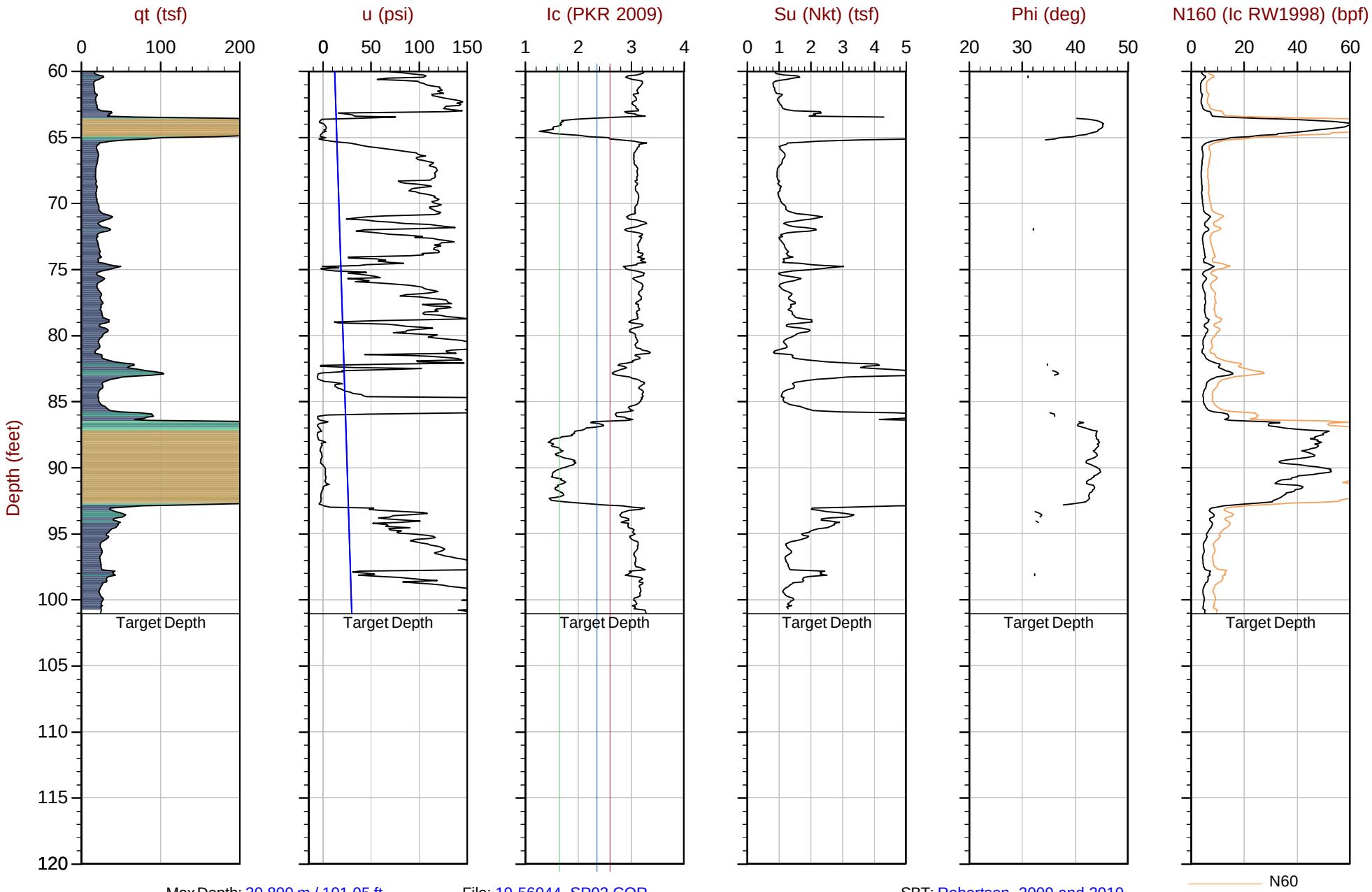
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 2 of 2

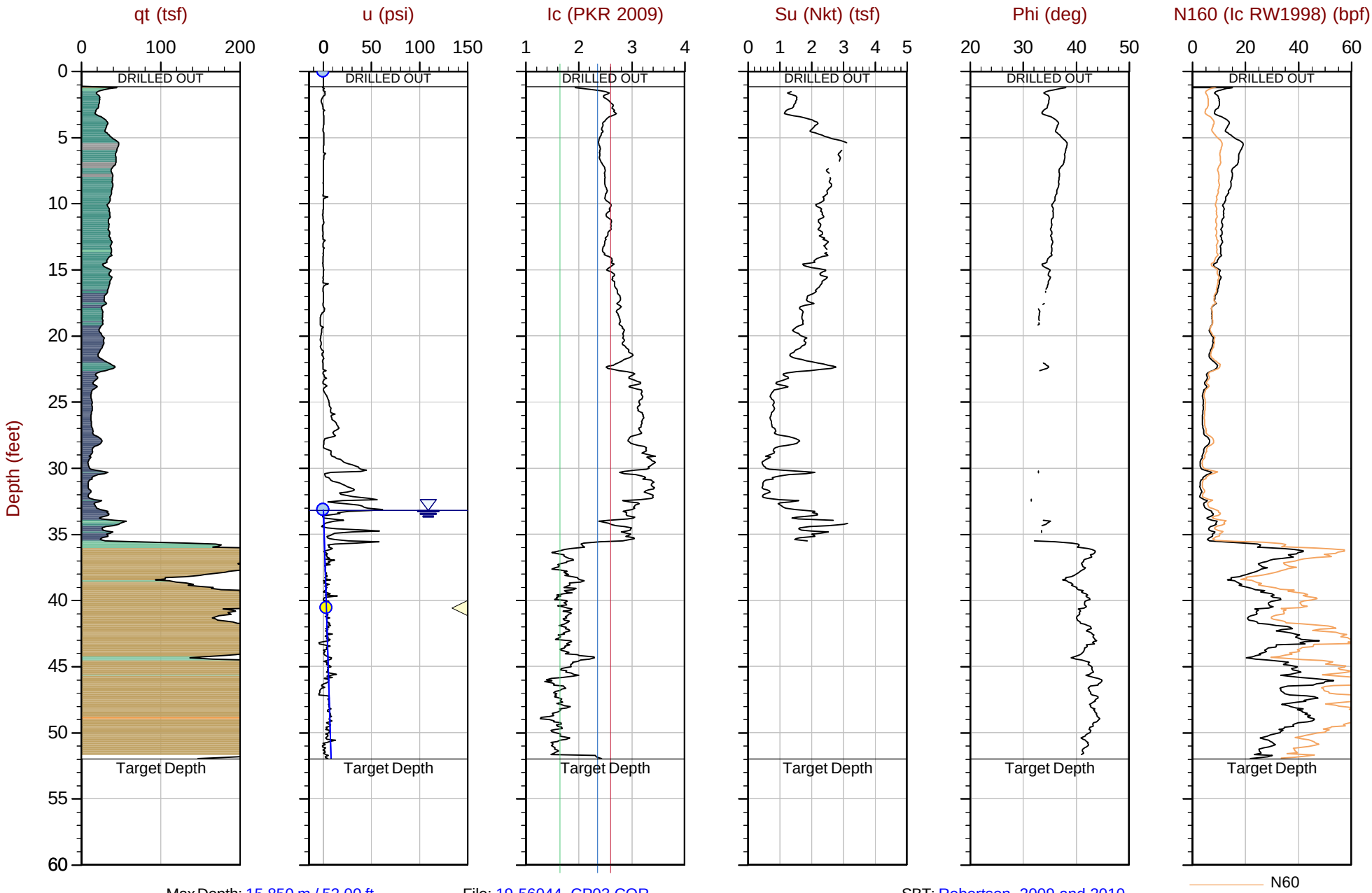
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 10:14
Site: Avenues Silicon Valley

Sounding: 1-CPT03
Cone: 447:T1500F15U500



Max Depth: 15.850 m / 52.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:
Assumed Ueq
Ueq

File: 19-56044_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

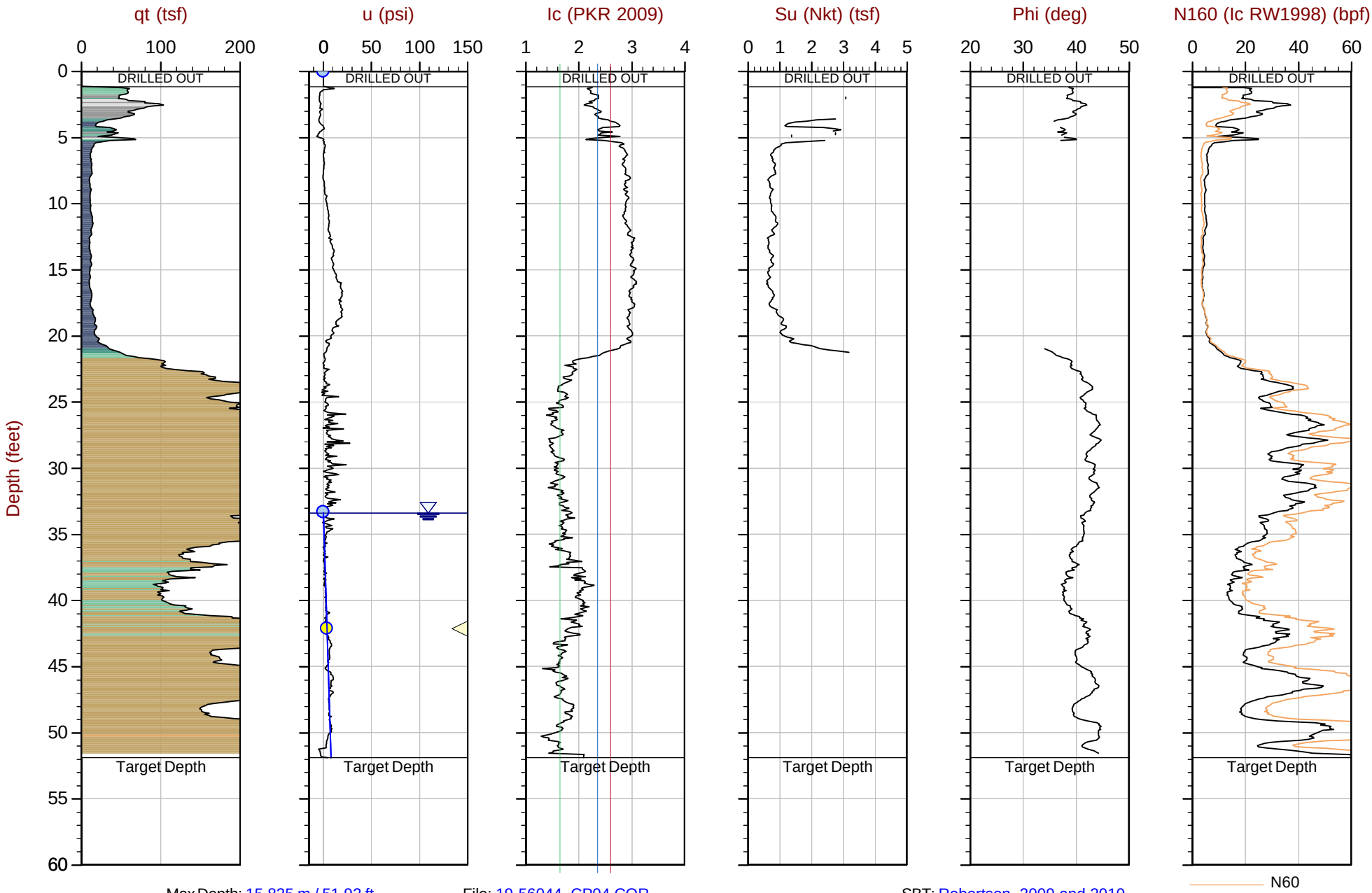
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130629m E: 596361m
Page No: 1 of 1

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 14:17
Site: Avenues Silicon Valley

Sounding: 1-CPT04
Cone: 447:T1500F15U500



Max Depth: 15.825 m / 51.92 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:
Assumed Ueq
Ueq

File: 19-56044_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

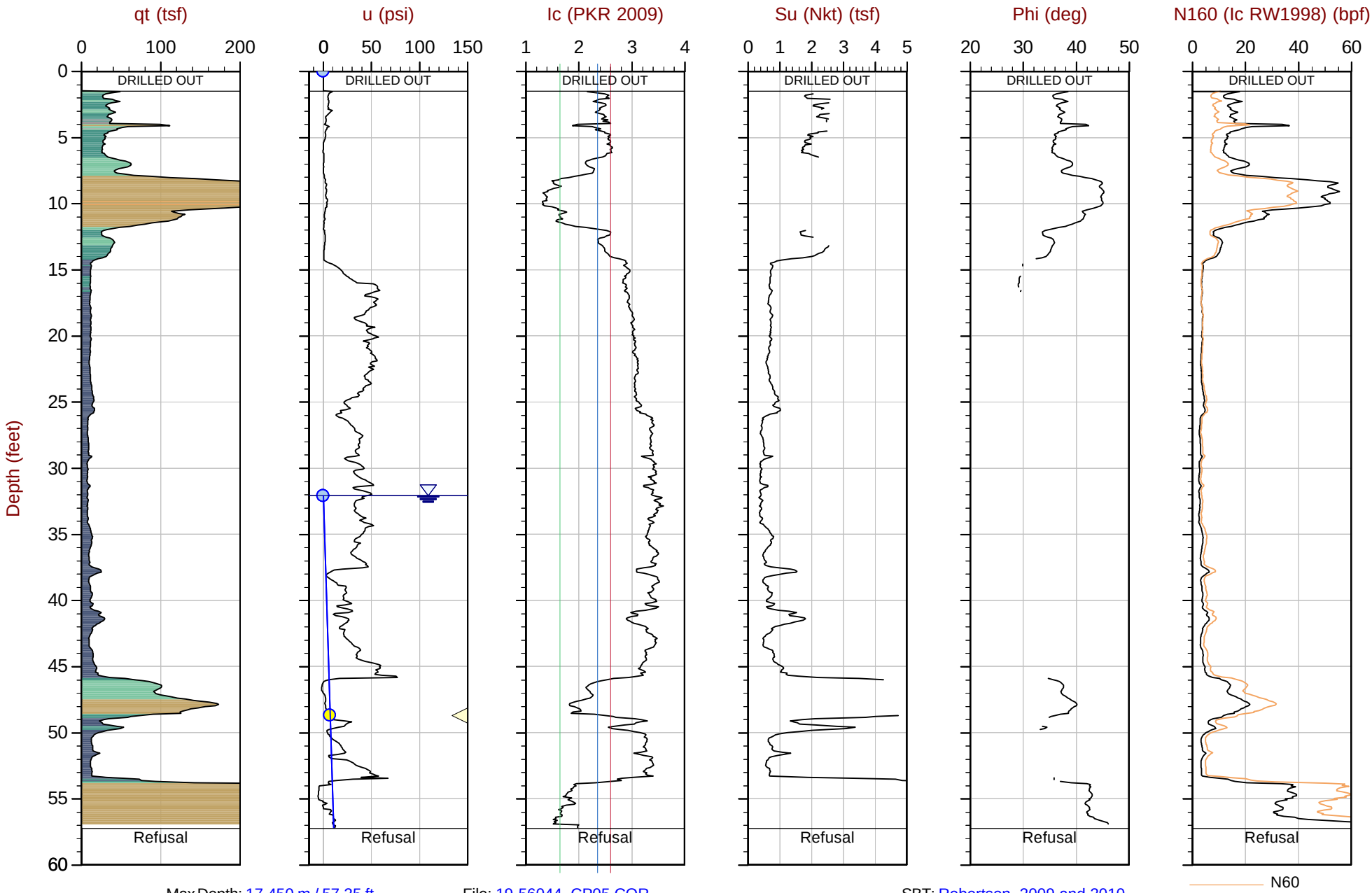
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130564m E: 596366m
Page No: 1 of 1

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 07:38
Site: Avenues Silicon Valley

Sounding: 1-CPT05
Cone: 447:T1500F15U500



Max Depth: 17.450 m / 57.25 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:
Assumed Ueq
Ueq

File: 19-56044_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

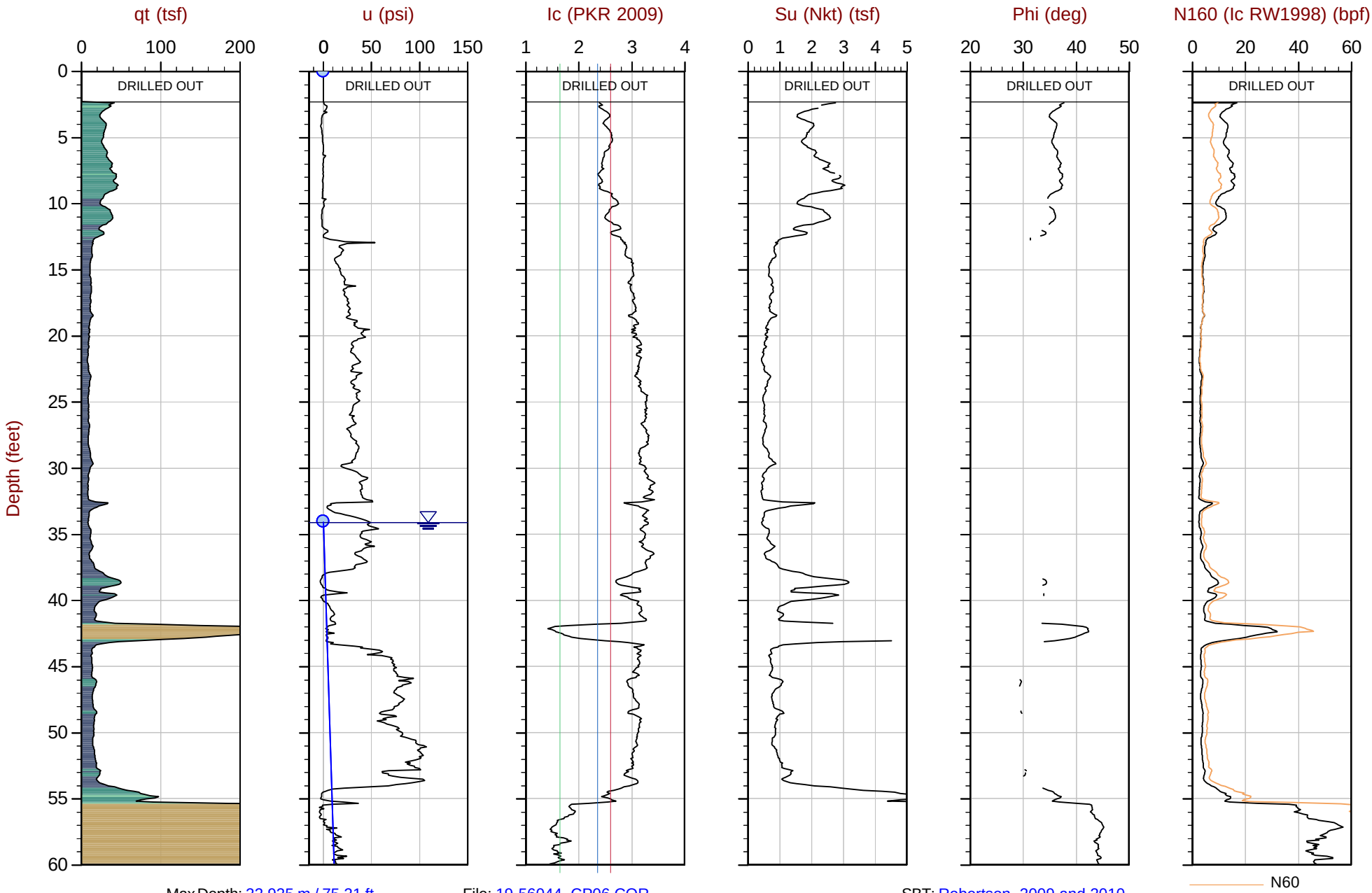
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130786m E: 596270m
Page No: 1 of 1

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500



Max Depth: 22.925 m / 75.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:
Assumed Ueq
Ueq

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

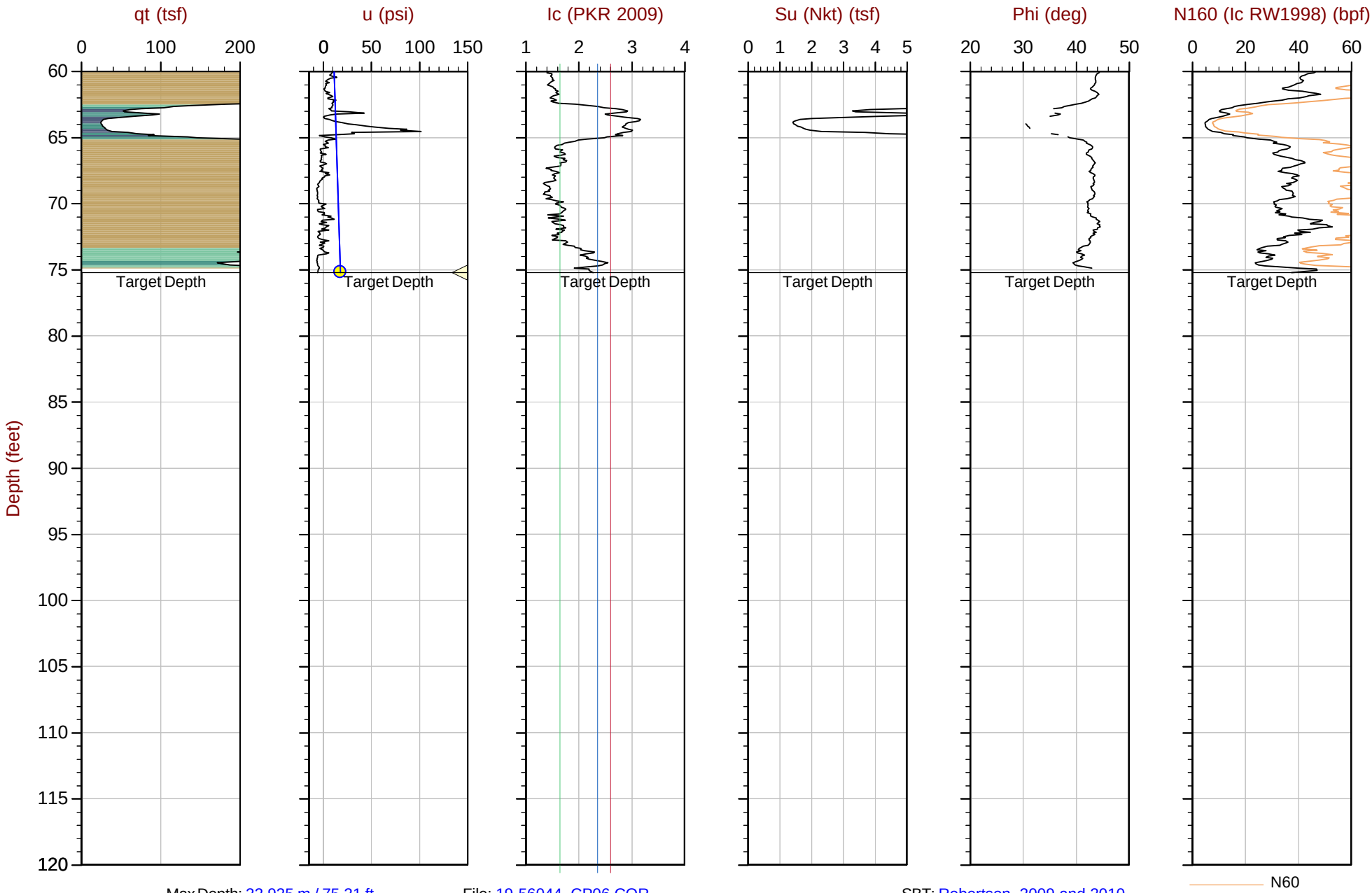
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 1 of 2

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500



Max Depth: 22.925 m / 75.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130688m E: 596318m
Page No: 2 of 2

Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

Seismic Cone Penetration Test Tabular Results

CONETEC

Job No:	19-56044
Client:	ENGEO Inc.
Project:	Avenues Silicon Valley
Sounding ID:	1-SCPT02
Date:	29-Mar-2019

Seismic Source:	Beam
Source Offset (ft):	1.9
Source Depth (ft):	0.0
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)
2.7	2.1	2.8			
6.0	5.3	5.6	2.8	9.6	297
9.2	8.5	8.7	3.1	3.4	903
12.5	11.8	12.0	3.2	3.7	872
15.7	15.1	15.2	3.2	4.5	724
19.0	18.4	18.5	3.3	3.9	836
22.3	21.7	21.7	3.3	4.0	815
25.6	24.9	25.0	3.3	3.9	844
28.9	28.2	28.3	3.3	4.1	803
32.2	31.5	31.6	3.3	2.6	1267
35.4	34.8	34.8	3.3	2.8	1188
38.7	38.1	38.1	3.3	2.5	1291
42.0	41.3	41.4	3.3	2.6	1248
45.3	44.6	44.7	3.3	2.6	1263
48.6	47.9	47.9	3.3	2.1	1589
51.8	51.2	51.2	3.3	3.2	1026
55.1	54.5	54.5	3.3	4.3	770
58.4	57.7	57.8	3.3	3.8	857
61.7	61.0	61.1	3.3	3.8	872
65.0	64.3	64.3	3.3	3.6	921
68.2	67.6	67.6	3.3	3.0	1094
71.5	70.9	70.9	3.3	4.1	807
74.8	74.1	74.2	3.3	4.0	829
78.1	77.4	77.5	3.3	3.7	887
81.4	80.7	80.7	3.3	4.0	821
84.6	84.0	84.0	3.3	3.1	1053
87.9	87.3	87.3	3.3	3.0	1091
91.2	90.6	90.6	3.3	2.5	1323
94.5	93.8	93.9	3.3	2.3	1424
97.8	97.1	97.1	3.3	3.3	1005
101.0	100.4	100.4	3.3	3.4	954

Seismic Cone Penetration Test Plot

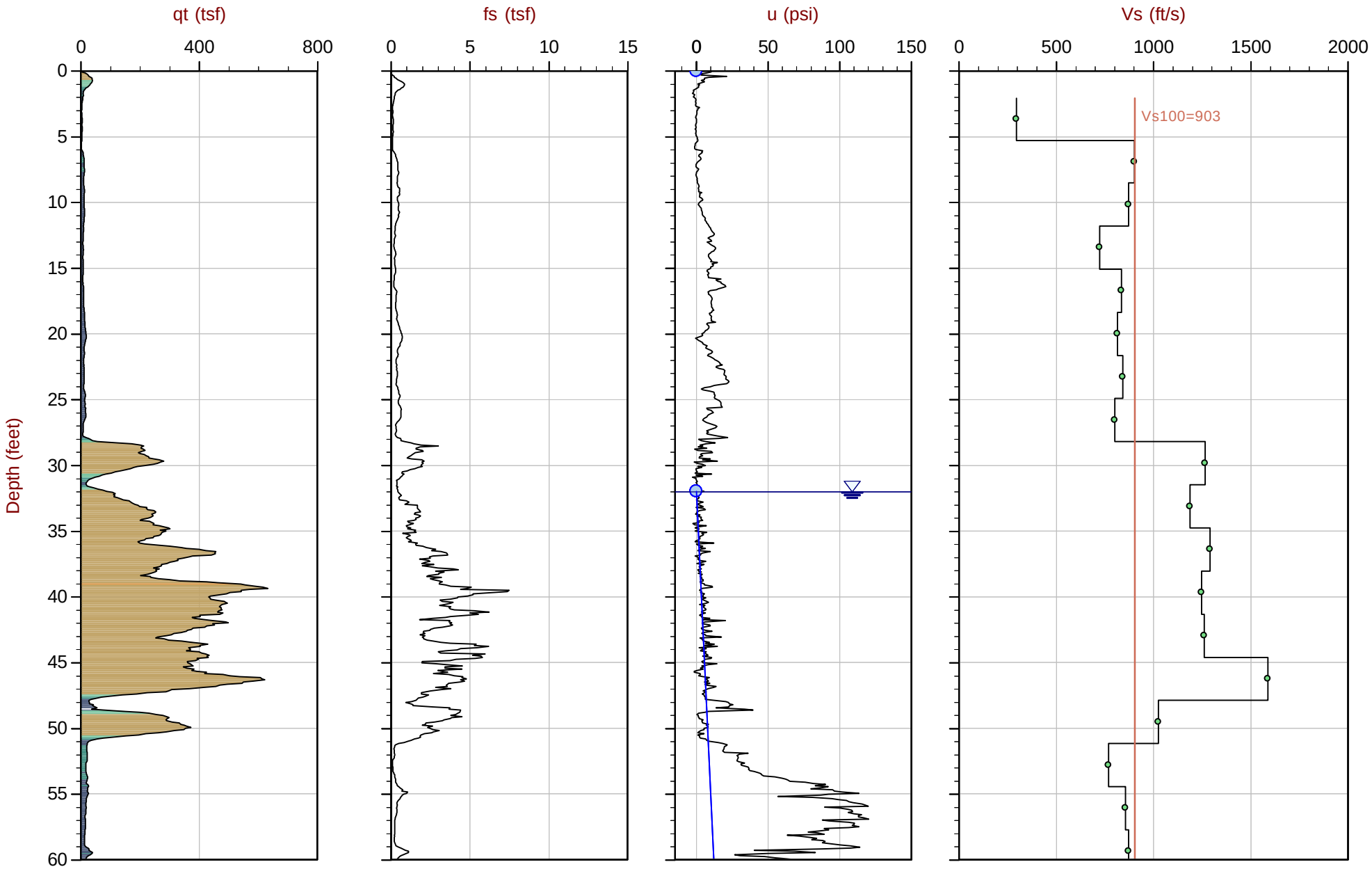
CONETEC

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 1 of 2

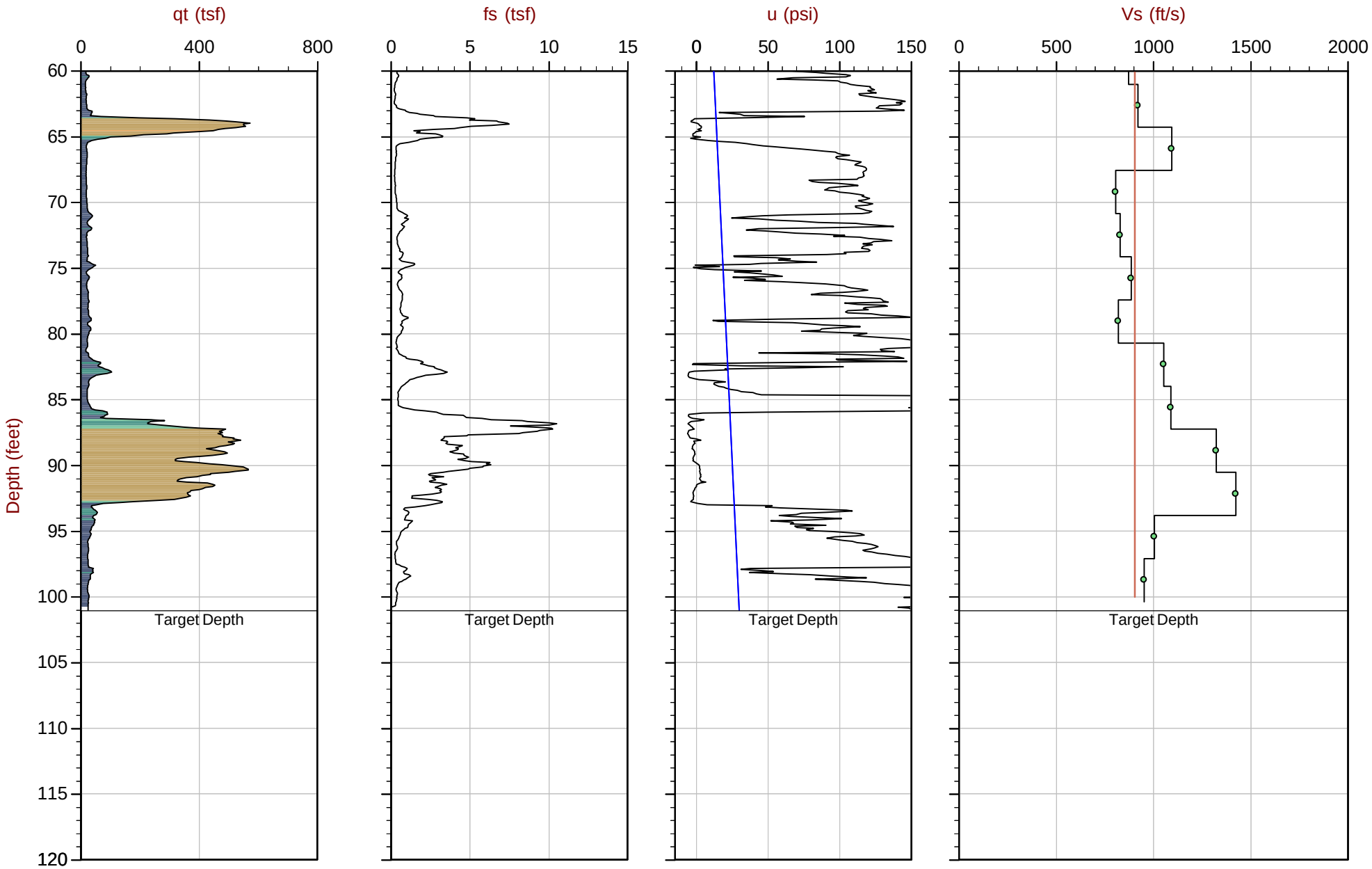
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

CONETEC

ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-56044_SP02.COR
Unit Wt: SBTQtn (PKR2009)

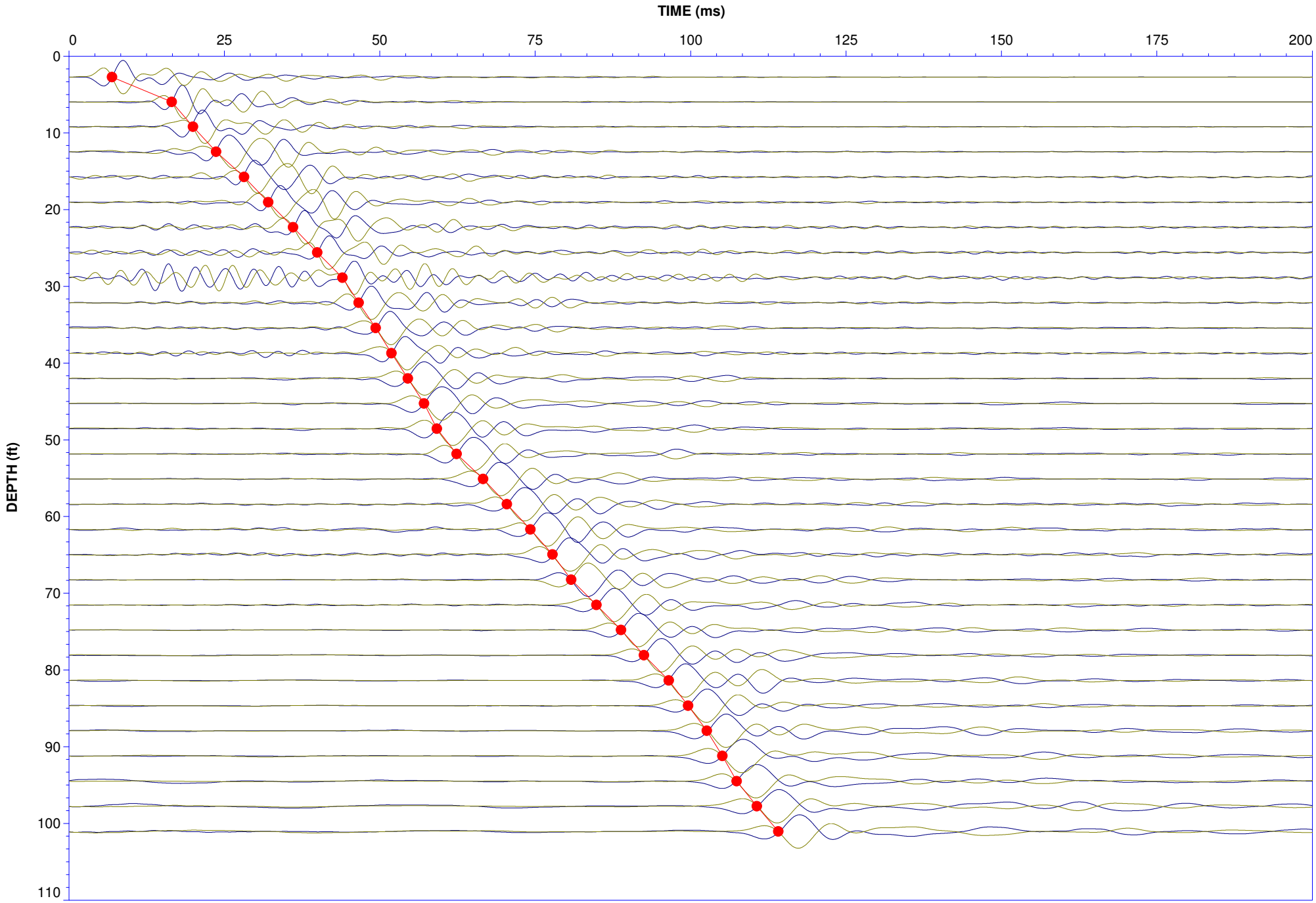
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4130599m E: 596466m
Page No: 2 of 2

Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

Seismic Cone Penetration Test Time Domain Traces

CONETEC

Job No: 19-56044 Client: ENGEO Inc. Project Title: Avenues Silicon Valley Hole: 1-SCPT02 Date: 03:29:19 12:39



Soil Behaviour Type (SBT) Scatter Plots

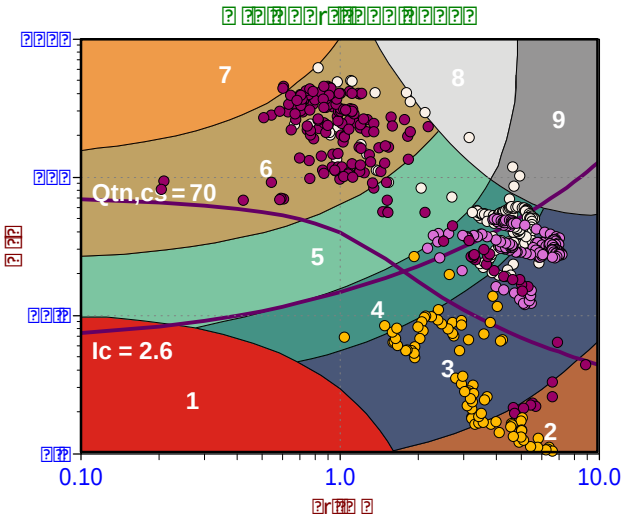
CONETEC

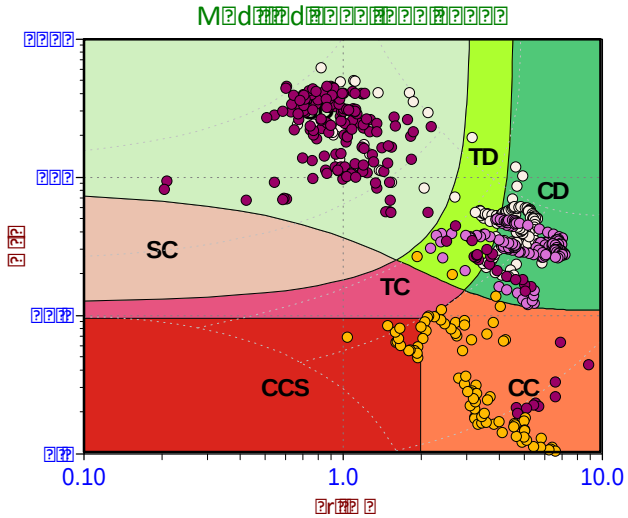
CONETEC

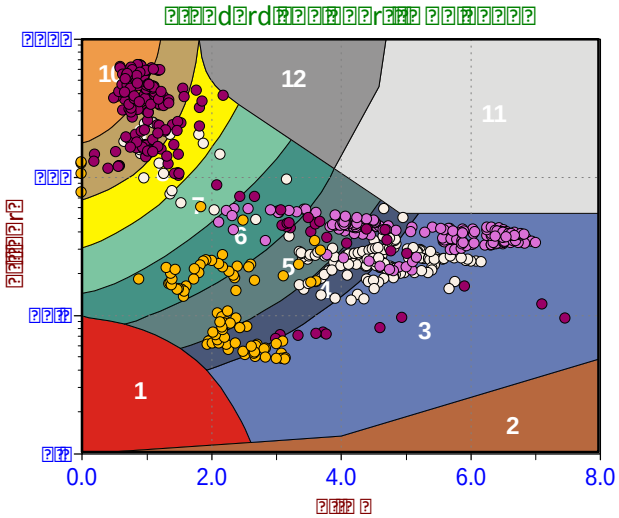
ENGEO

Job No: 19-56044
Date: 2019-03-29 11:28
Site: Avenues Silicon Valley

Sounding: 1-CPT01
Cone: 447:T1500F15U500







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 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
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- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
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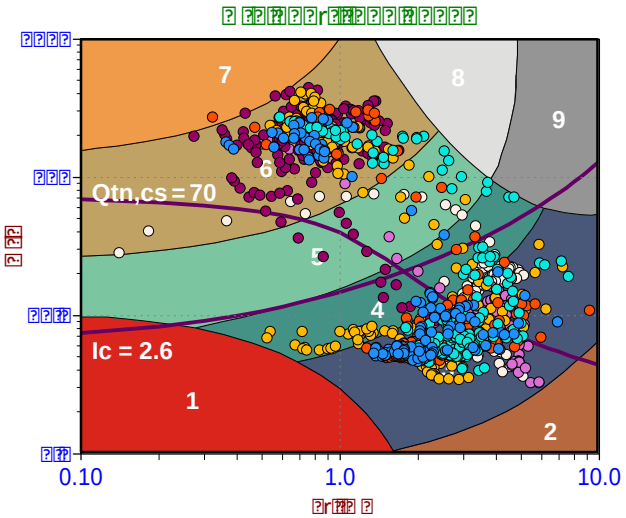
- Cemented Sand

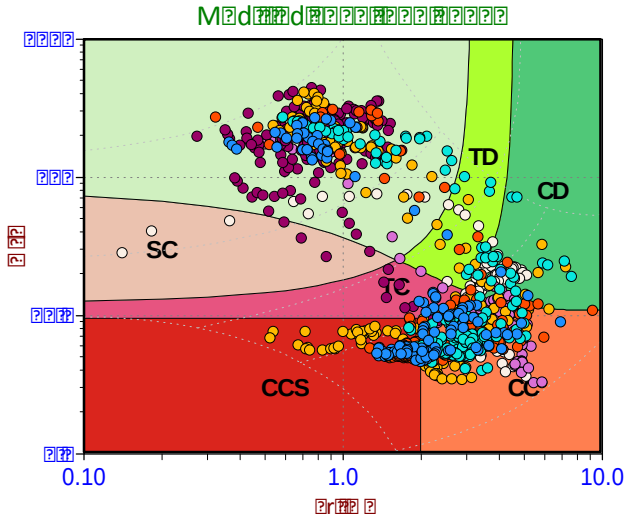
CONETEC

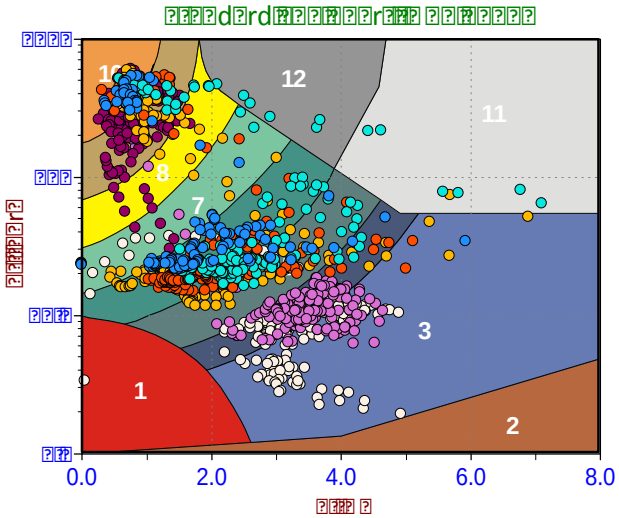
ENGEO

Job No: 19-56044
Date: 2019-03-29 12:39
Site: Avenues Silicon Valley

Sounding: 1-SCPT02
Cone: 447:T1500F15U500







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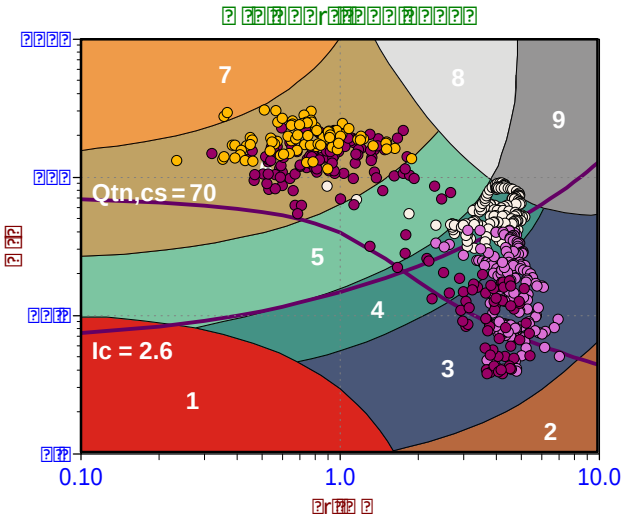
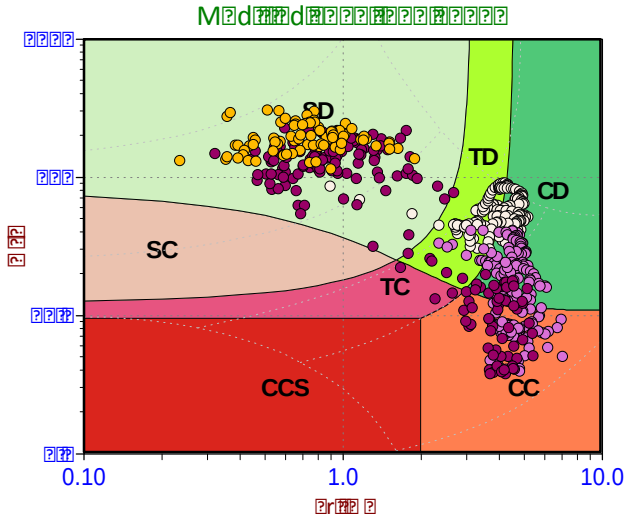
CONETEC

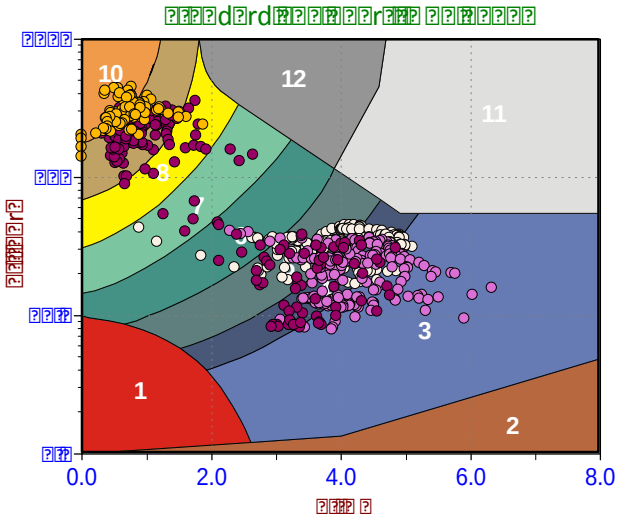
ENGEO

Job No: 19-56044
Date: 2019-03-29 10:14
Site: Avenues Silicon Valley

Sounding: 1-CPT03
Cone: 447:T1500F15U500







Depth Ranges
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Legend
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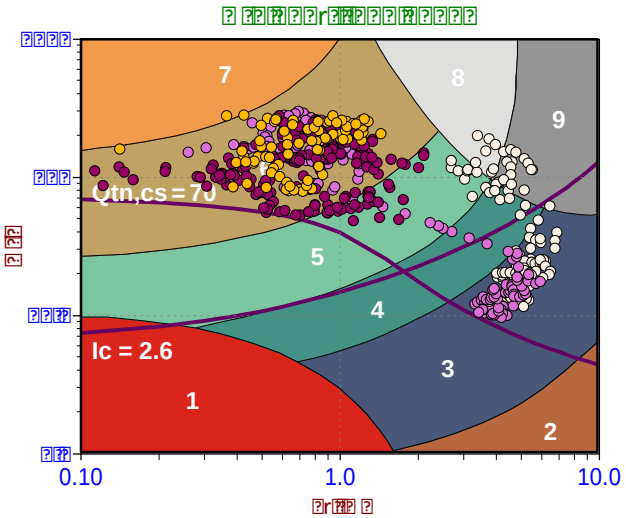
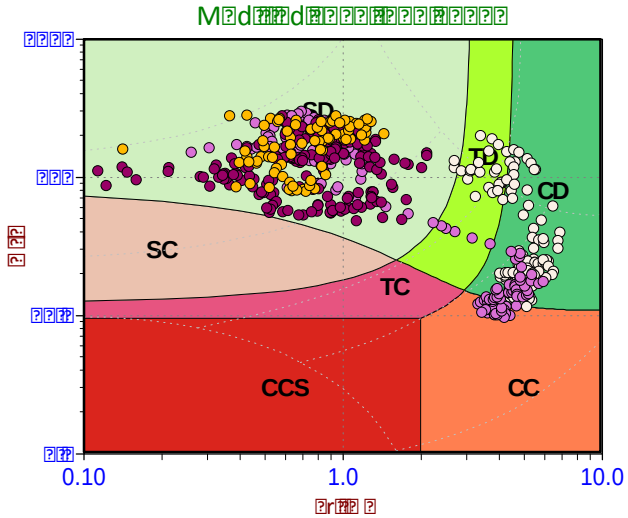
CONETEC

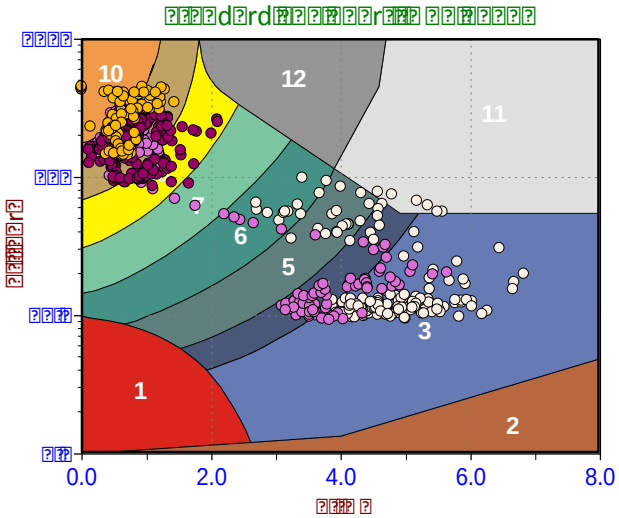
ENGEO

Job No: 19-56044
Date: 2019-03-29 14:17
Site: Avenues Silicon Valley

Sounding: 1-CPT04
Cone: 447:T1500F15U500







Depth Ranges
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Legend
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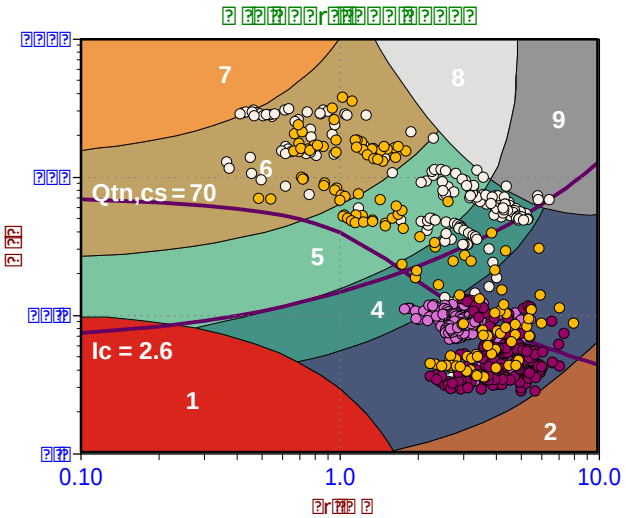
- Stiff Fine Grained
- Cemented Sand

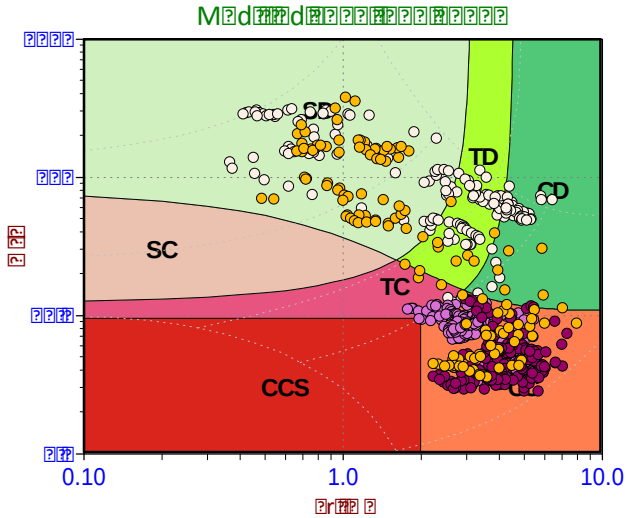
CONETEC

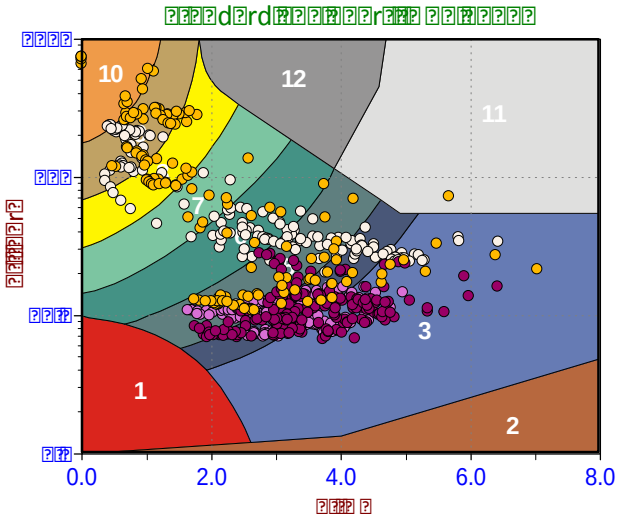
ENGEO

Job No: 19-56044
Date: 2019-03-29 07:38
Site: Avenues Silicon Valley

Sounding: 1-CPT05
Cone: 447:T1500F15U500







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Legend
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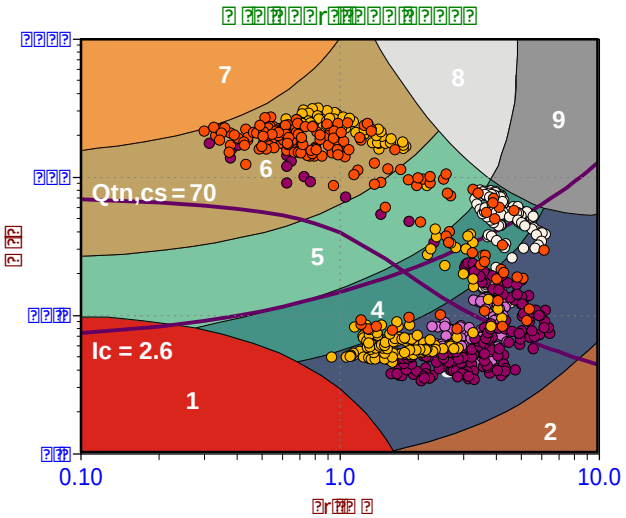
- Stiff Fine Grained
- Cemented Sand

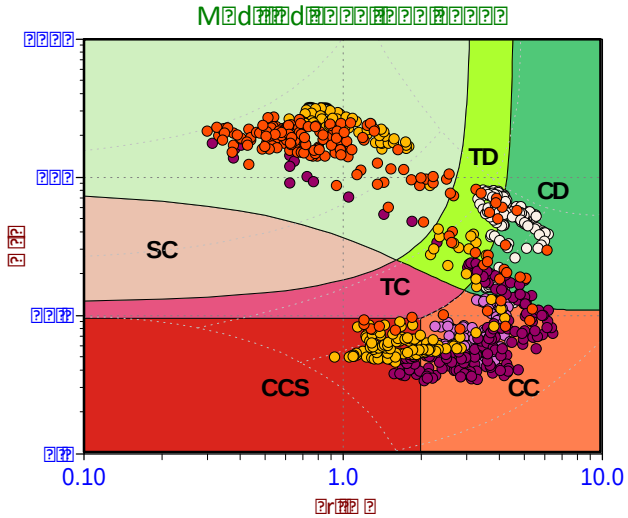
CONETEC

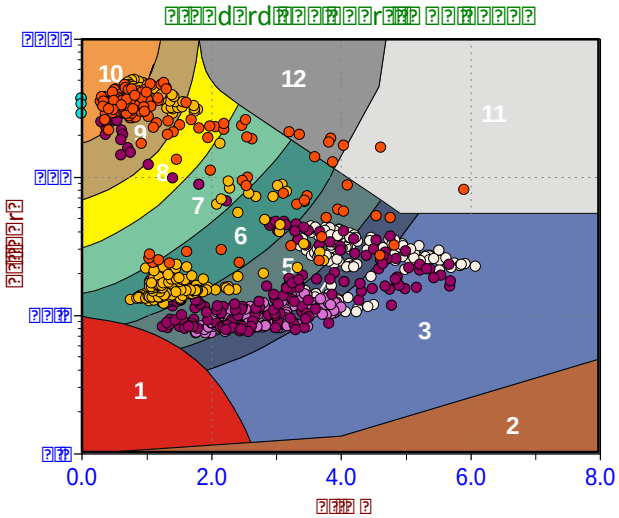
ENGEO

Job No: 19-56044
Date: 2019-03-29 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500







Depth Ranges
- >0.0 to 15.0 ft
- >15.0 to 30.0 ft
- >30.0 to 45.0 ft
- >45.0 to 60.0 ft
- >60.0 to 75.0 ft
- >75.0 to 90.0 ft
- >90.0 to 105.0 ft
- >105.0 to 120.0 ft
- >120.0 to 135.0 ft
- >135.0 to 150.0 ft
- >150.0 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 Summary and
Pore Pressure Dissipation Plots

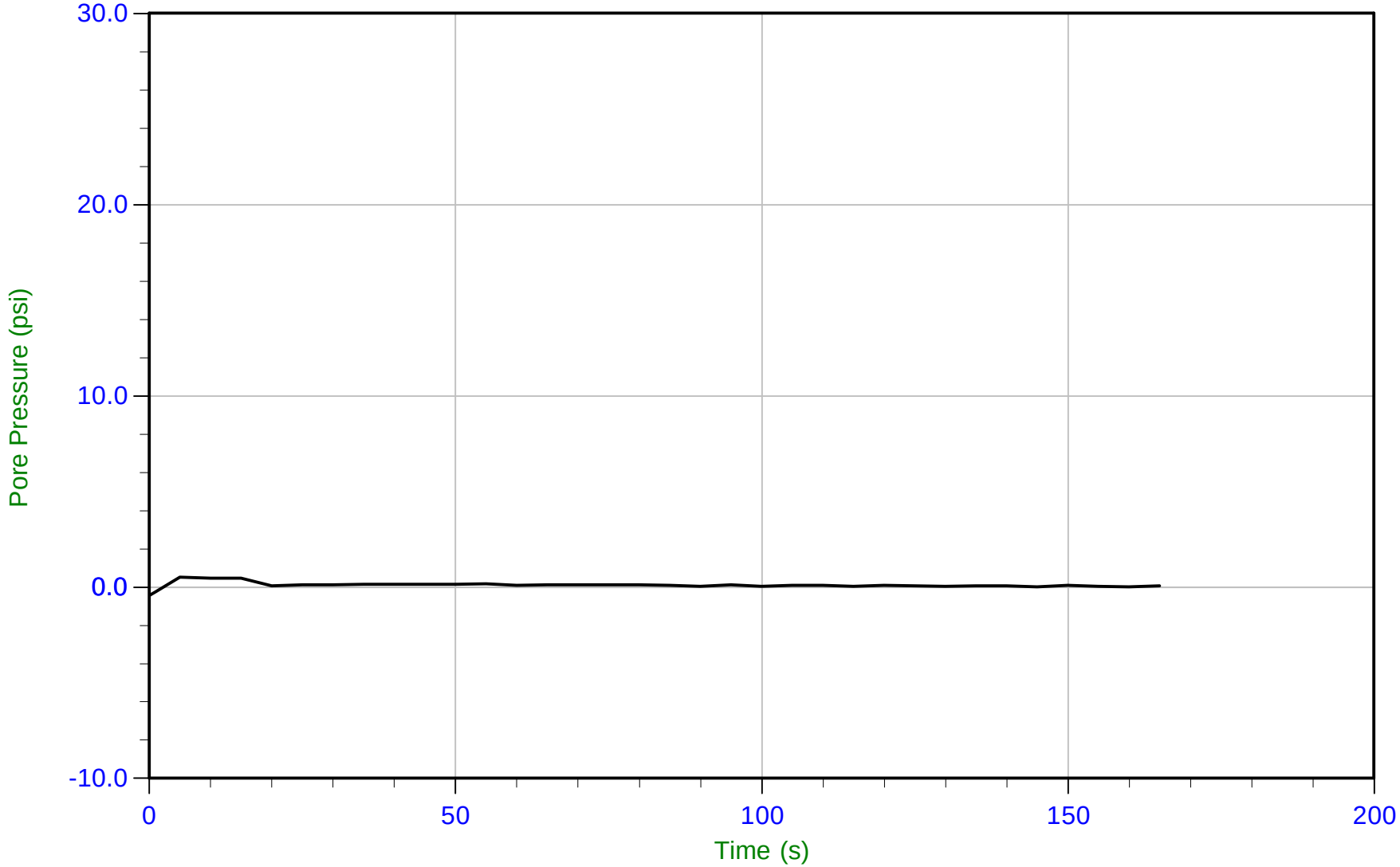
CONETEC

Job No:	19-56044
Client:	ENGEO Inc.
Project:	Avenues Silicon Valley
Start Date:	29-Mar-2019
End Date:	29-Mar-2019

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (psi)	Calculated Phreatic Surface (ft)
1-CPT01	19-56044_CP01	15	165	34.04	0.0	
1-CPT01	19-56044_CP01	15	245	52.00	7.6	34.4
1-CPT03	19-56044_CP03	15	500	40.60	3.2	33.2
1-CPT04	19-56044_CP04	15	425	42.16	3.8	33.4
1-CPT05	19-56044_CP05	15	80	48.72	7.2	32.1
1-CPT06	19-56044_CP06	15	255	75.21	17.8	34.1

ENGEO	Job No: 19-56044 Date: 03/29/2019 11:28 Site: Avenues Silicon Valley	Sounding: 1-CPT01 Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

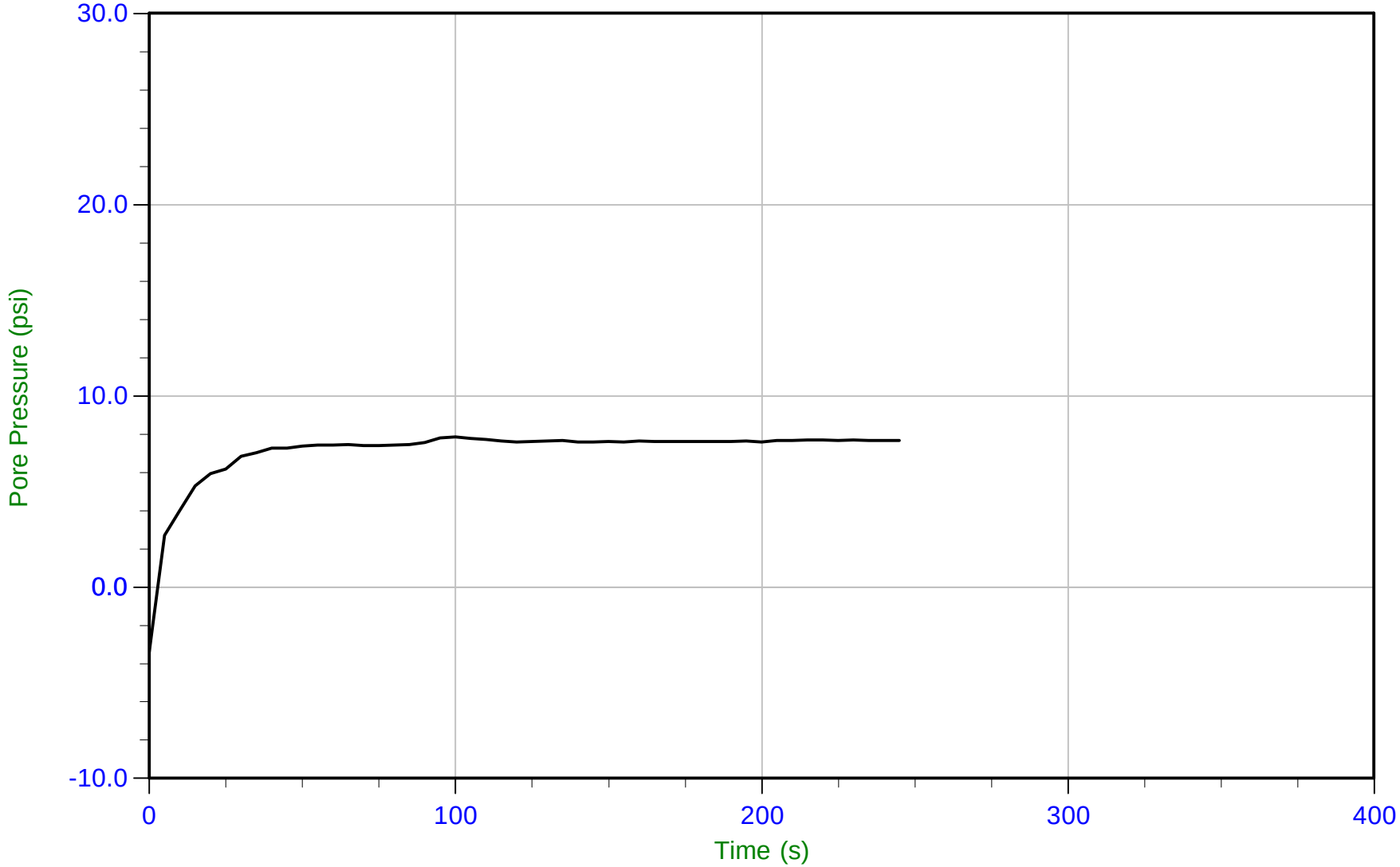
Filename: 19-56044_CP01.PPF
Depth: 10.375 m / 34.038 ft
Duration: 165.0 s

u Min: -0.4 psi
u Max: 0.5 psi
u Final: 0.1 psi

ENGEO

Job No: 19-56044
Date: 03/29/2019 11:28
Site: Avenues Silicon Valley

Sounding: 1-CPT01
Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56044_CP01.PPF
Depth: 15.850 m / 52.001 ft
Duration: 245.0 s

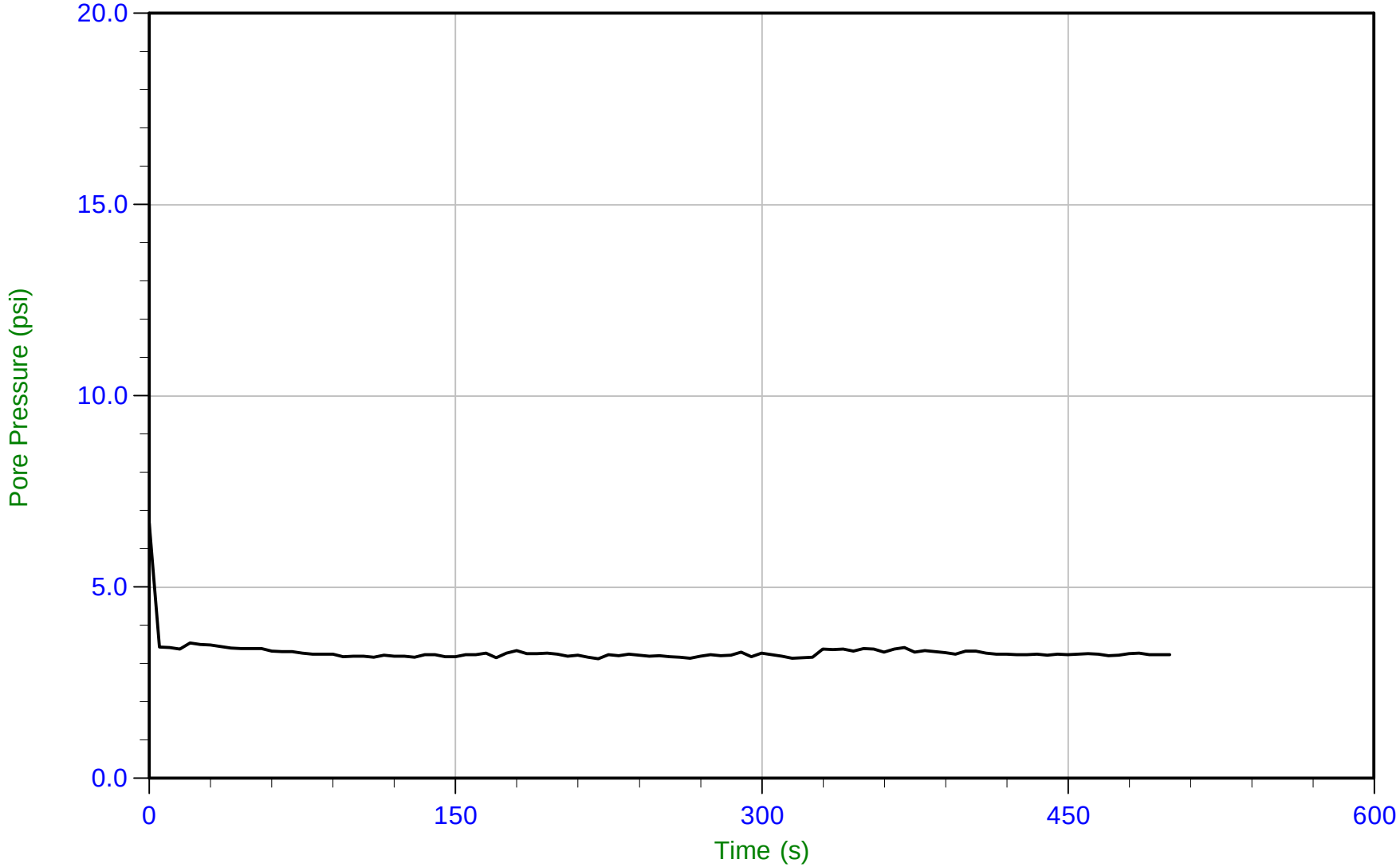
u Min: -3.4 psi
u Max: 7.9 psi
u Final: 7.7 psi

WT: 10.475 m / 34.367 ft
Ueq: 7.6 psi

ENGEO

Job No: 19-56044
Date: 03/29/2019 10:14
Site: Avenues Silicon Valley

Sounding: 1-CPT03
Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56044_CP03.PPF
Depth: 12.375 m / 40.600 ft
Duration: 500.0 s

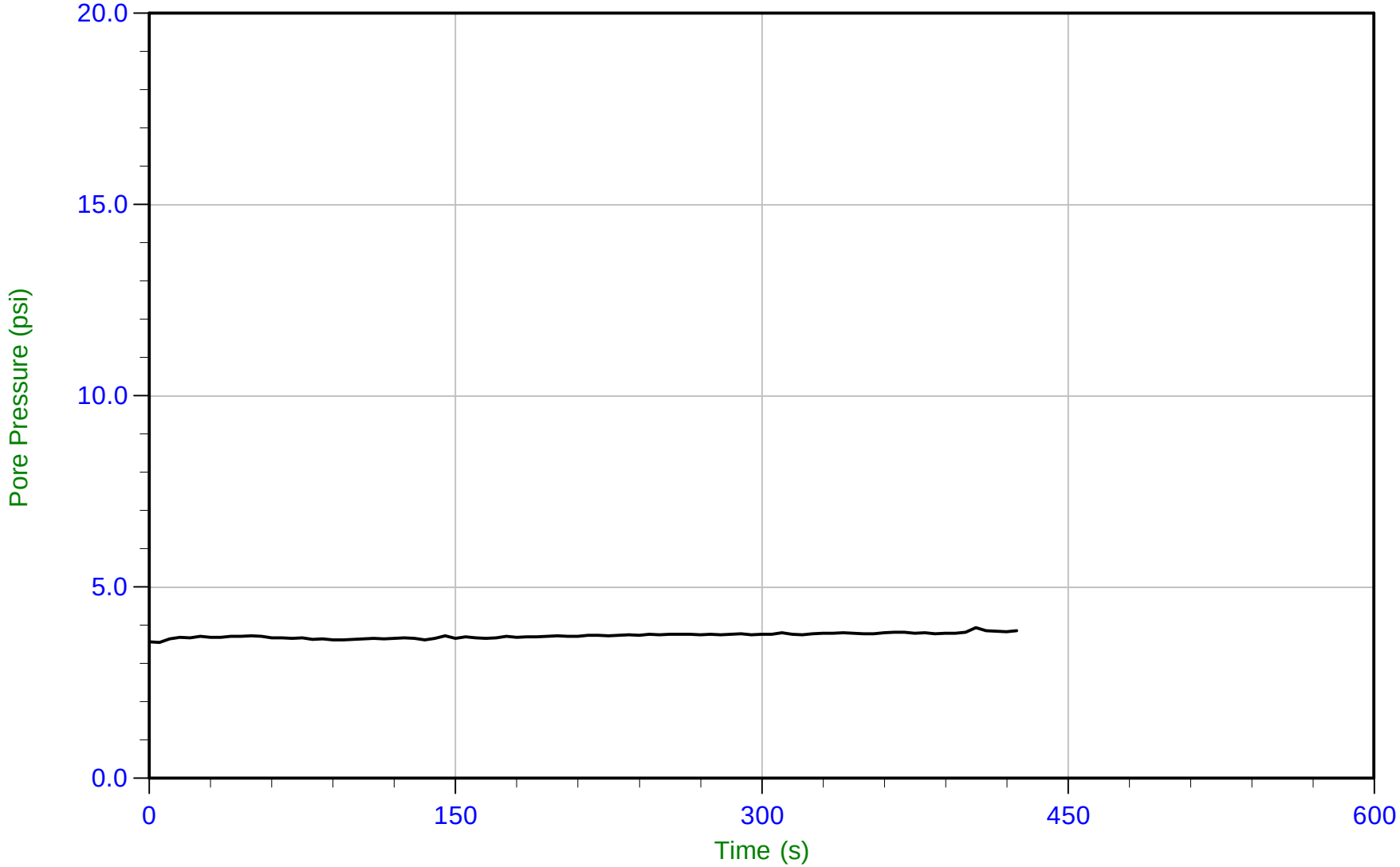
u Min: 3.1 psi
u Max: 6.7 psi
u Final: 3.2 psi

WT: 10.117 m / 33.191 ft
Ueq: 3.2 psi

ENGEO

Job No: 19-56044
Date: 03/29/2019 14:17
Site: Avenues Silicon Valley

Sounding: 1-CPT04
Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56044_CP04.PPF
Depth: 12.850 m / 42.158 ft
Duration: 425.0 s

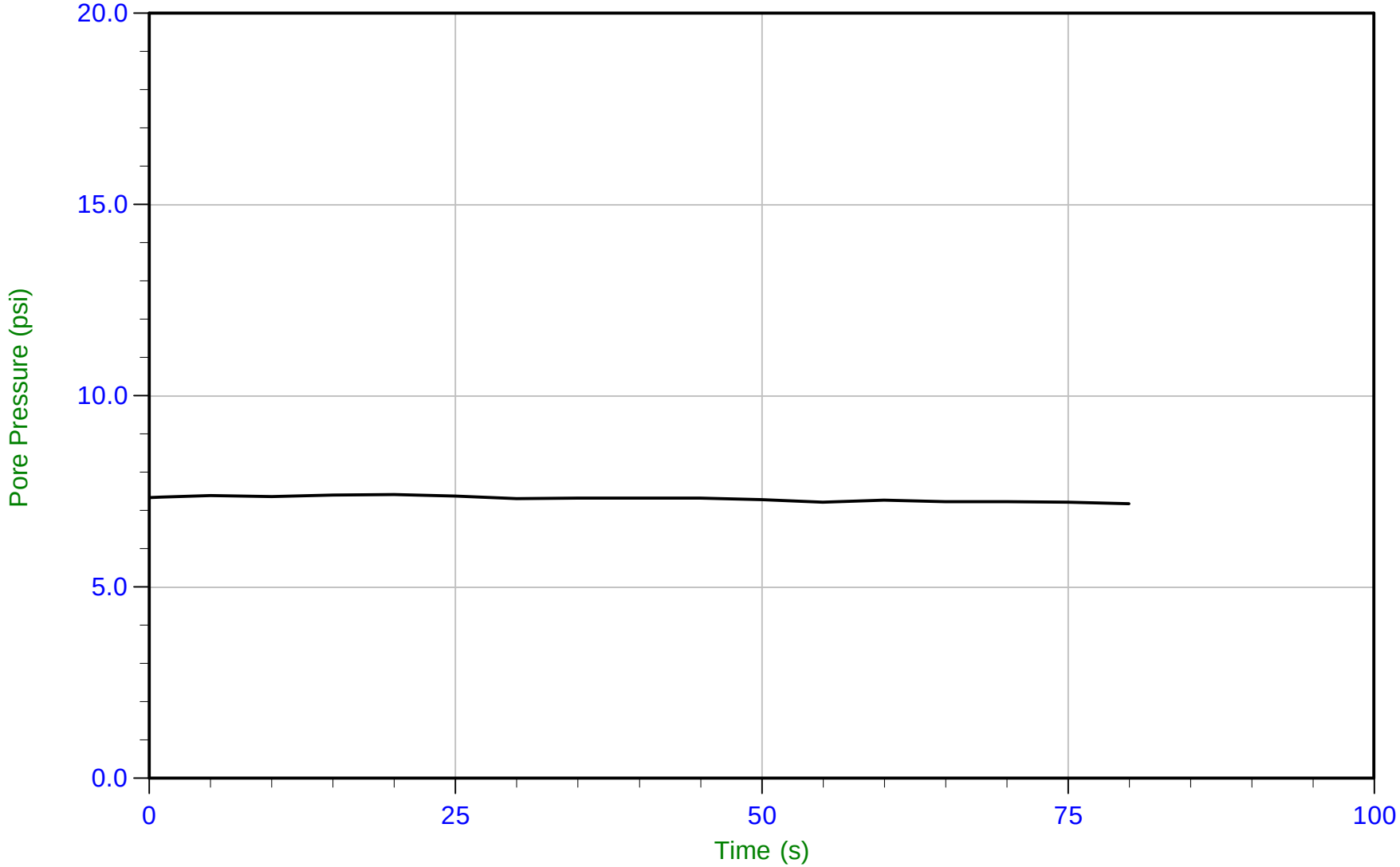
u Min: 3.5 psi
u Max: 3.9 psi
u Final: 3.9 psi

WT: 10.170 m / 33.366 ft
Ueq: 3.8 psi

ENGEO

Job No: 19-56044
Date: 03/29/2019 07:38
Site: Avenues Silicon Valley

Sounding: 1-CPT05
Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56044_CP05.PPF
Depth: 14.850 m / 48.720 ft
Duration: 80.0 s

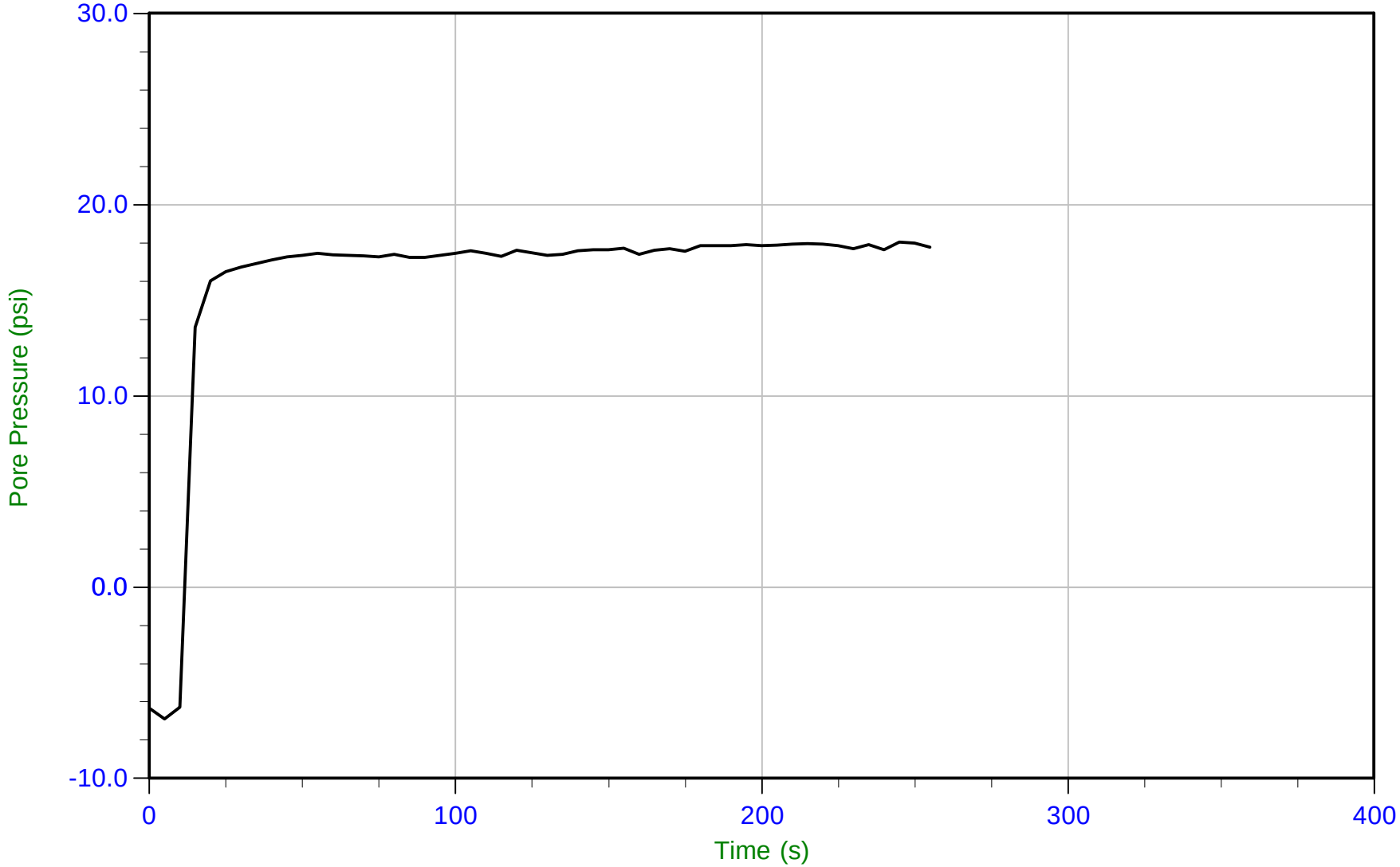
u Min: 7.2 psi
u Max: 7.4 psi
u Final: 7.2 psi

WT: 9.791 m / 32.123 ft
Ueq: 7.2 psi

ENGEO

Job No: 19-56044
Date: 03/29/2019 08:49
Site: Avenues Silicon Valley

Sounding: 1-CPT06
Cone: 447:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56044_CP06.PPF
Depth: 22.925 m / 75.212 ft
Duration: 255.0 s

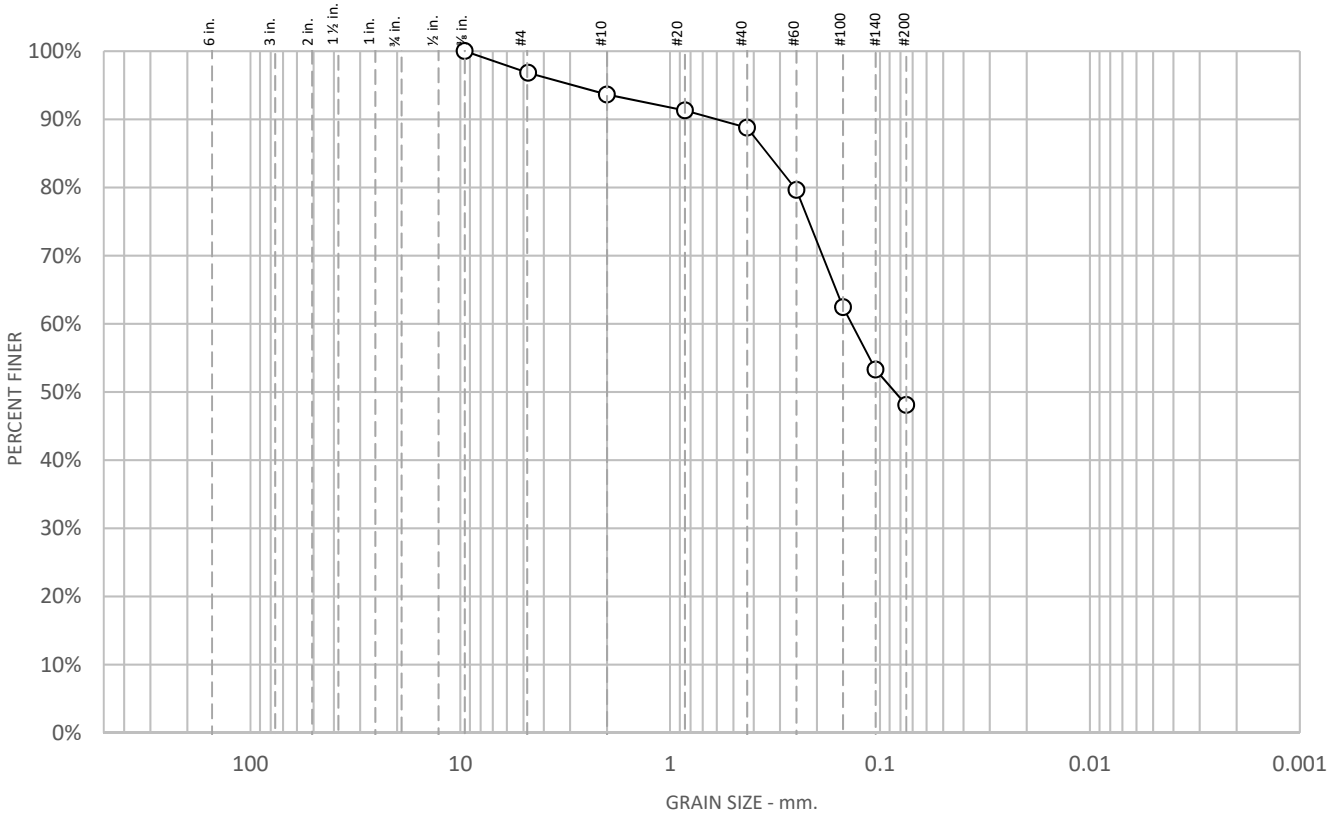
u Min: -6.9 psi
u Max: 18.0 psi
u Final: 17.8 psi

WT: 10.384 m / 34.067 ft
Ueq: 17.8 psi

APPENDIX C

LABORATORY TEST DATA

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		3.2	3.2	4.8	40.7	48.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100.0		
#4	96.8		
#10	93.6		
#20	91.3		
#40	88.8		
#60	79.6		
#100	62.4		
#140	53.3		
#200	48.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 0.6000 mm D_{85} = 0.3440 mm D_{60} = 0.1364 mm
D_{50} = 0.0849 mm D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D6913, Method B

Sample Number: 1-B1 @ 10

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, CA

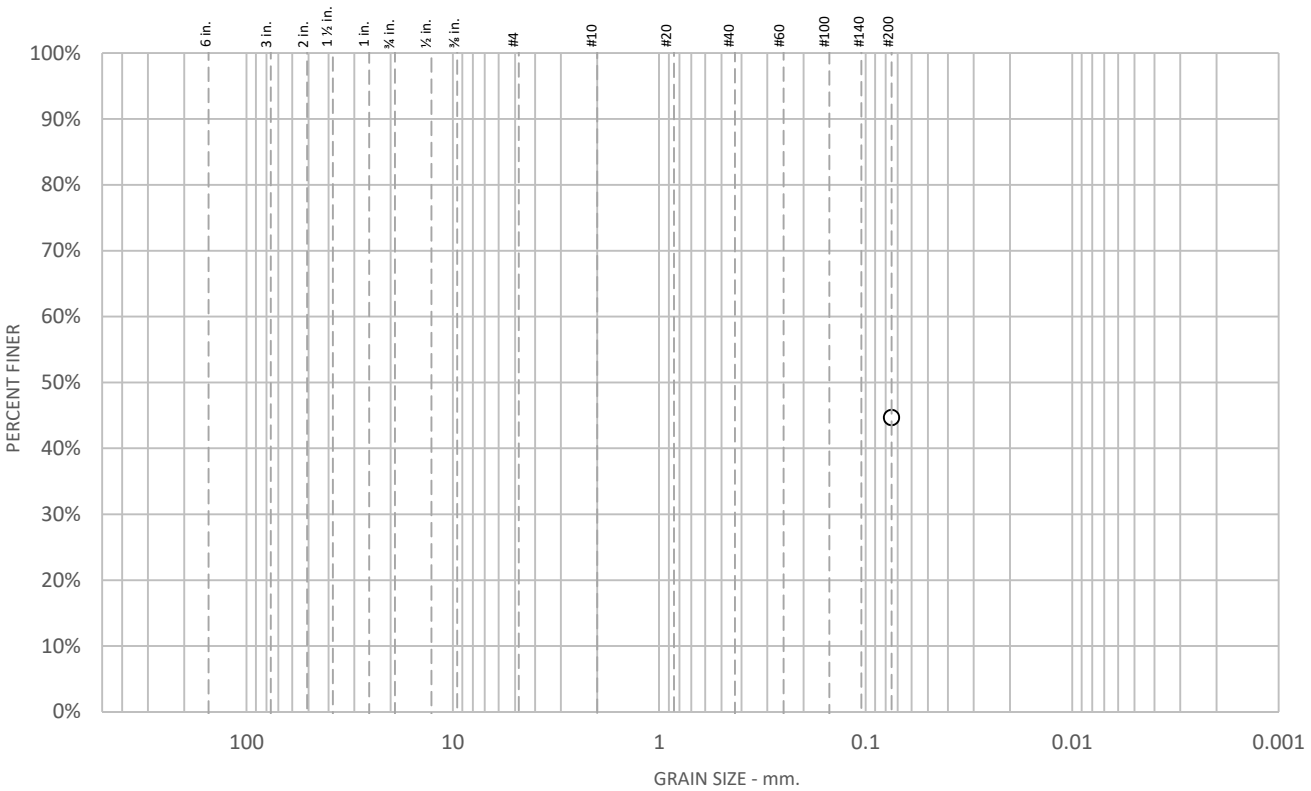
Project Number: 15929.000.000
Date: 4/26/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** W. Miller

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						44.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44.7		

* (no specification provided)

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

ASTM D1140, Method B
Soak time = 240 min
Dry sample weight = 92.74 g

Sample Number: 1-B1 @ 22.0-23.5 feet

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, California

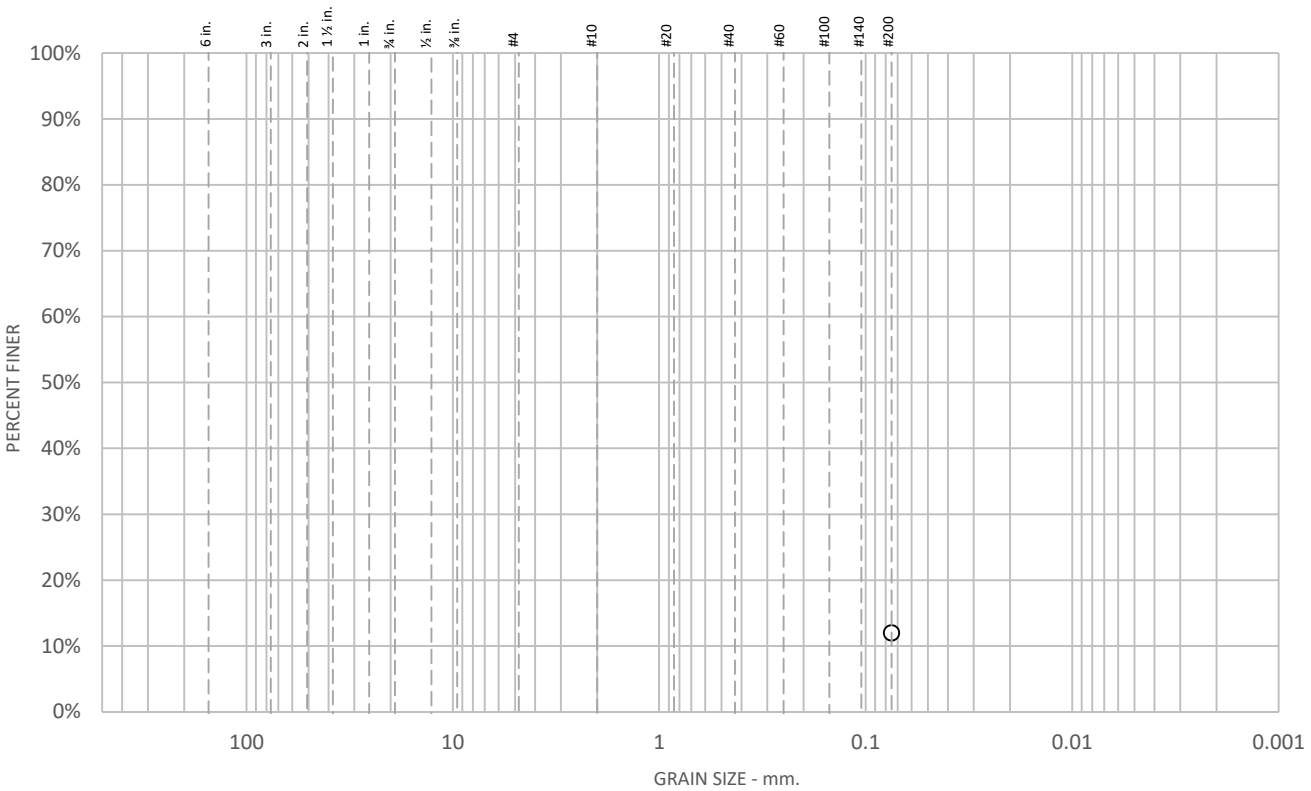
Project Number: 15929.000.000
Date: 4/24/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						12.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12.0		

* (no specification provided)

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

ASTM D1140, Method A
Soak time = 240 min
Dry sample weight = 285.97 g

Sample Number: 1-B1 @ 27.0-28.5 feet

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, California

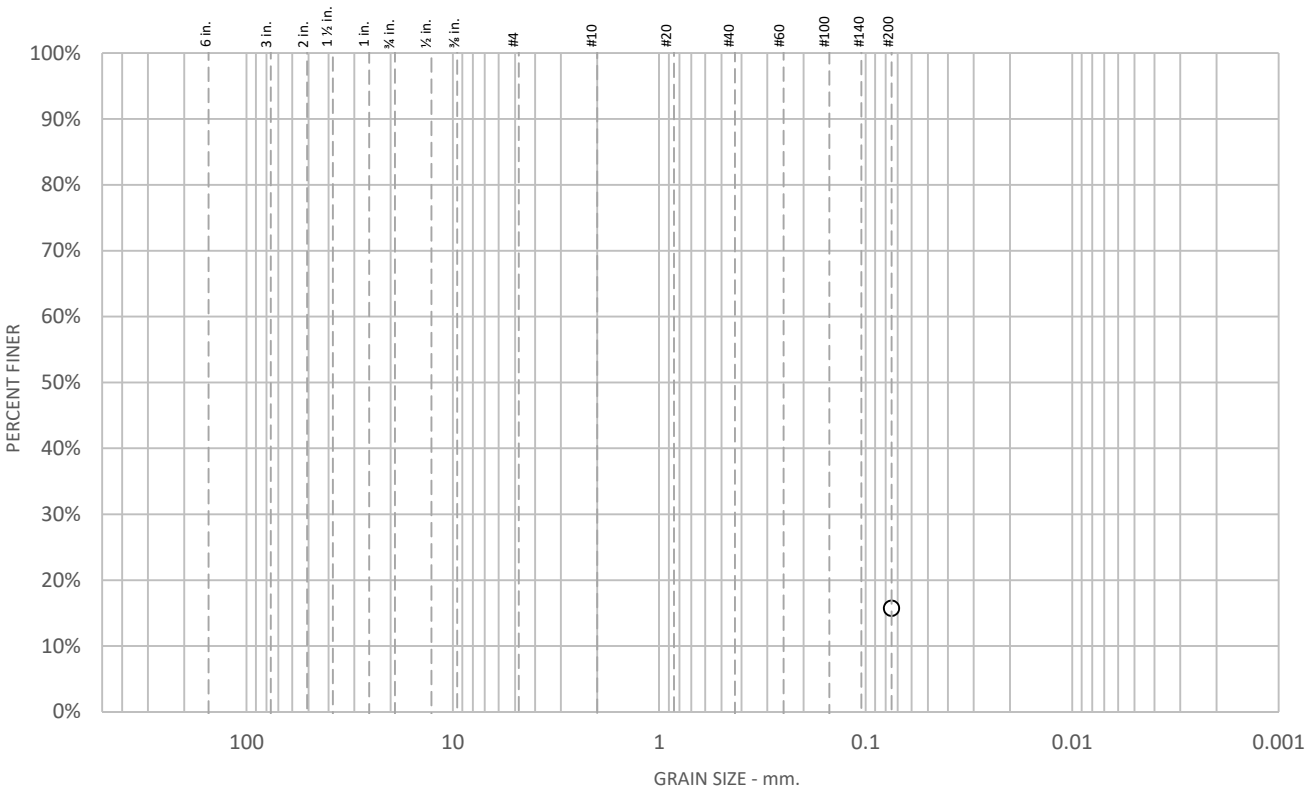
Project Number: 15929.000.000
Date: 4/24/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						15.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	15.7		

* (no specification provided)

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

ASTM D1140, Method A
Soak time = 240 min
Dry sample weight = 274.85 g

Sample Number: 1-B2 @ 30.0-30.5 feet

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, California

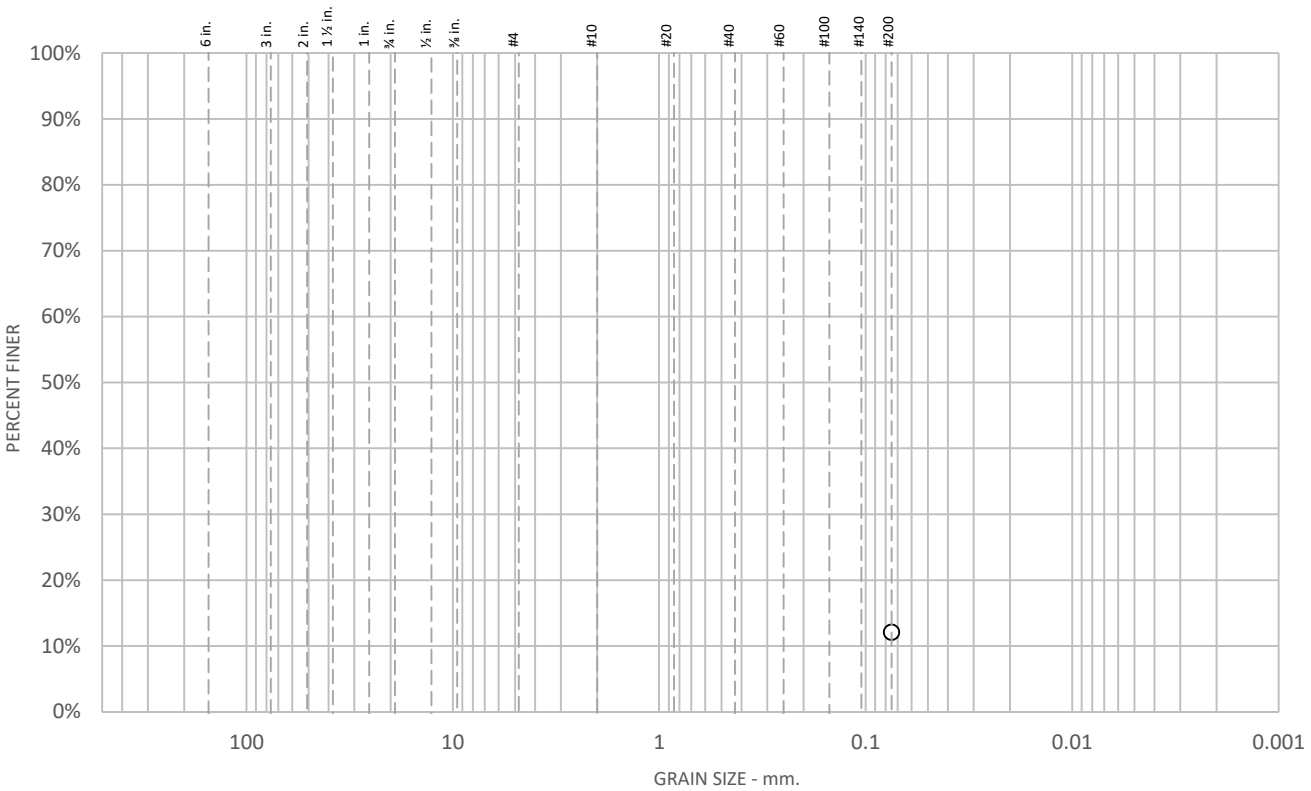
Project Number: 15929.000.000
Date: 4/24/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						12.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12.1		

* (no specification provided)

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

ASTM D1140, Method A
Soak time = 240 min
Dry sample weight = 347.55 g

Sample Number: 1-B3 @ 35.0-36.0 feet

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, California

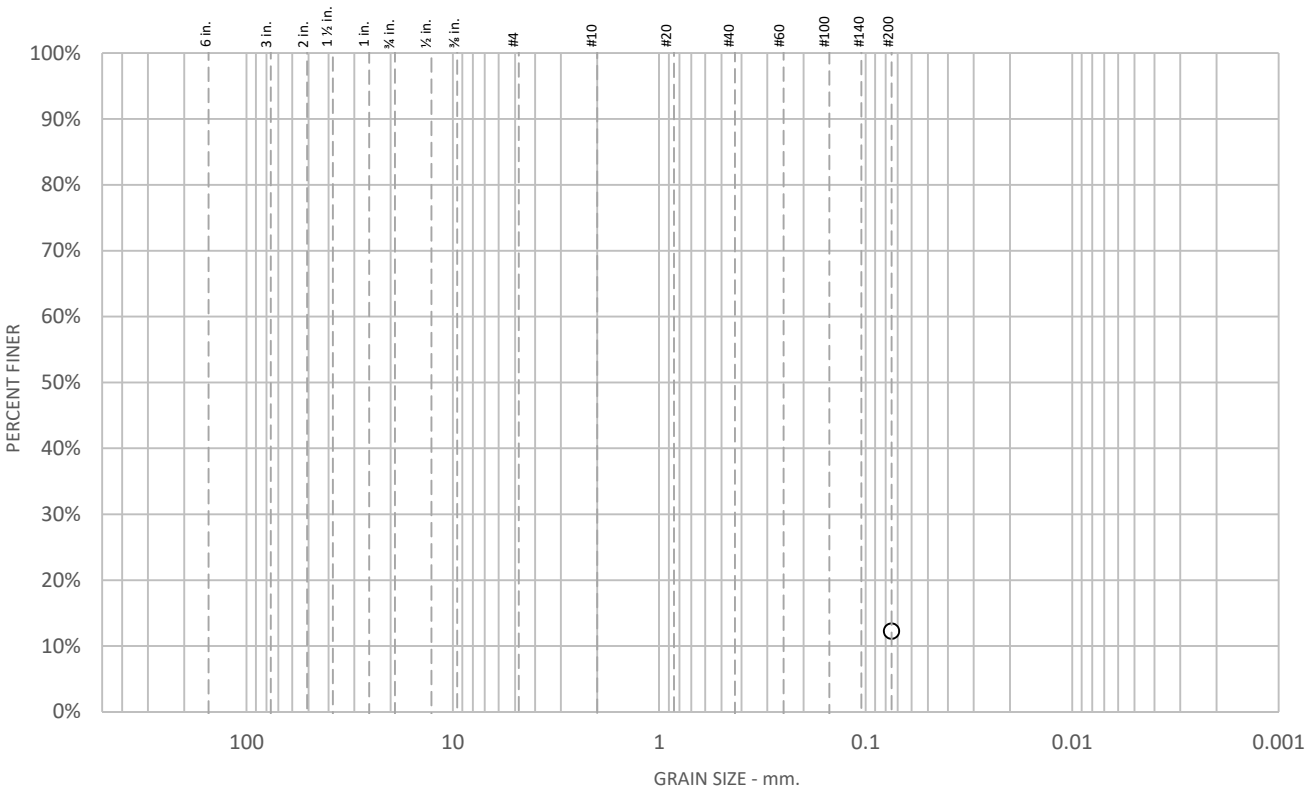
Project Number: 15929.000.000
Date: 4/24/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						12.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12.3		

* (no specification provided)

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

ASTM D1140, Method A
Soak time = 240 min
Dry sample weight = 302.57 g

Sample Number: 1-B4 @ 25.0-26.5 feet

Client: Avenues World Holdings, LLC
Project: Avenues SV
Project location: San Jose, California

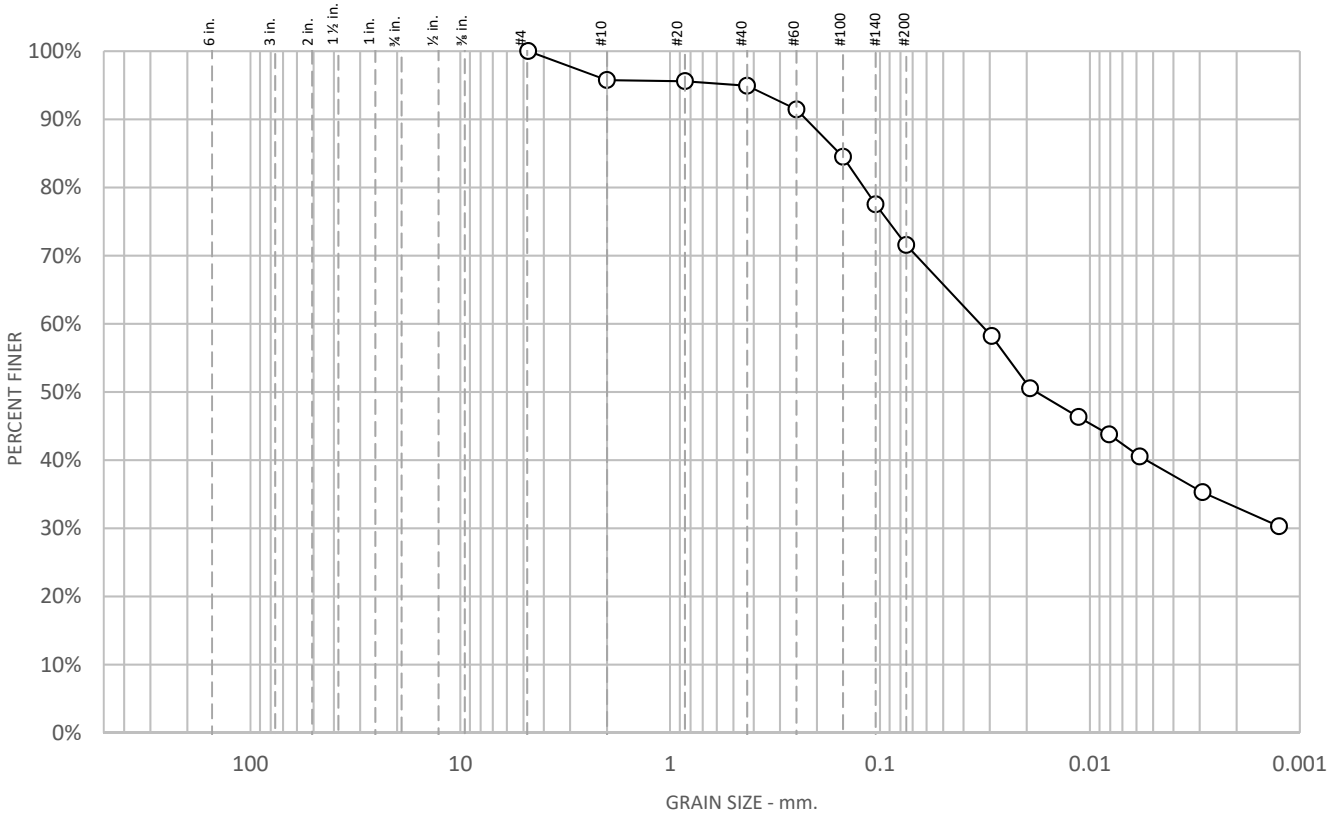
Project Number: 15929.000.000
Date: 4/24/2019

ENGEO Expect Excellence

Tested By: M. Quasem **Checked By:** G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			4.2	0.9	23.3	38.5	33.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	95.8		
#20	95.6		
#40	94.9		
#60	91.4		
#100	84.5		
#140	77.5		
#200	71.6		
0.0294 mm.	58.2		
0.0193 mm.	50.5		
0.0113 mm.	46.3		
0.0081 mm.	43.8		
0.0058 mm.	40.5		
0.0029 mm.	35.3		
0.0013 mm.	30.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = 0.2248 mm D_{85} = 0.1557 mm D_{60} = 0.0333 mm
D_{50} = 0.0180 mm D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

GS: ASTM D422 ASTM D422
Silt/clay division of 0.002mm used

Sample Number: 1-B5 @ 4.5-5

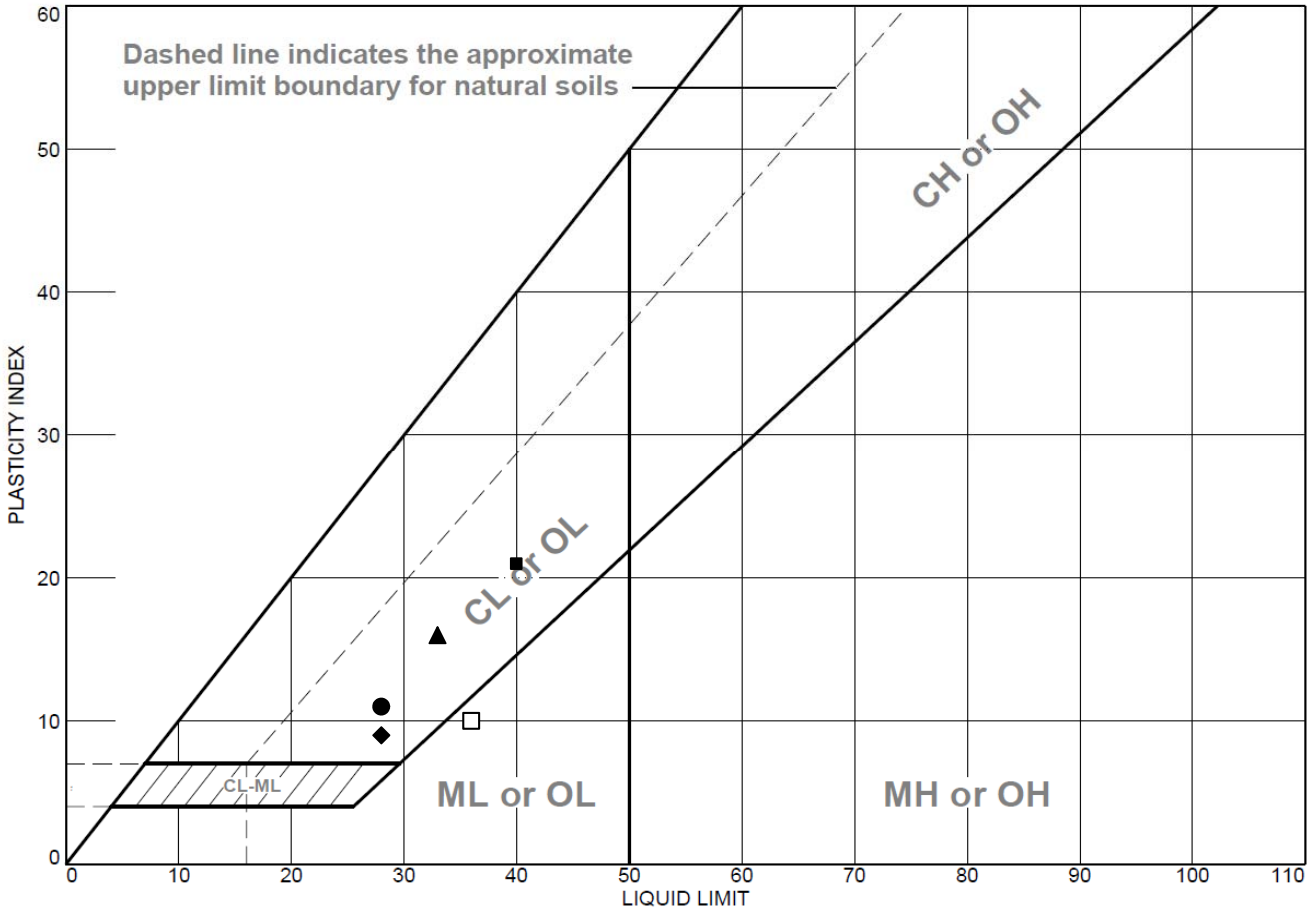
Client: Avenes World Holdings LLC **Project Number:** 15929.000.000
Project: Avenues SV **Date:** 4/26/2019
Project location: San Jose, CA

ENGEO Expect Excellence

Tested By: W. Miller **Checked By:** M. Quasem

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
▲	See exploration logs	33	17	16			
◆	See exploration logs	28	19	9		44.7	SC
□	See exploration logs	36	26	10			
●	See exploration logs	28	17	11			
■	See exploration logs	40	19	21			

Project No.: 15929.000.000 **Date:** 4/26/2019
Project Name: Avenues SV
Project location: San Jose, California
Client: Avenues World Holdings LLC

▲ **Sample Number:** 1-B1@4.5-5 **Depth:** 4.5-5.0'
◆ **Sample Number:** 1-B1@22-23.5 **Depth:** 22.0-23.5'
□ **Sample Number:** 1-B2@5.5-6 **Depth:** 5.5-6.0'
● **Sample Number:** 1-B2@21-21.5 **Depth:** 21.0-21.5'
■ **Sample Number:** 1-B3@4-4.5 **Depth:** 4.0-4.5'

Remarks:

▲ PI: ASTM D4318, Wet Method

◆ PI: ASTM D4318, Wet Method, GS: ASTM D1140, Method A, USCS: ASTM D2487

□ PI: ASTM D4318, Wet Method

● PI: ASTM D4318, Wet Method

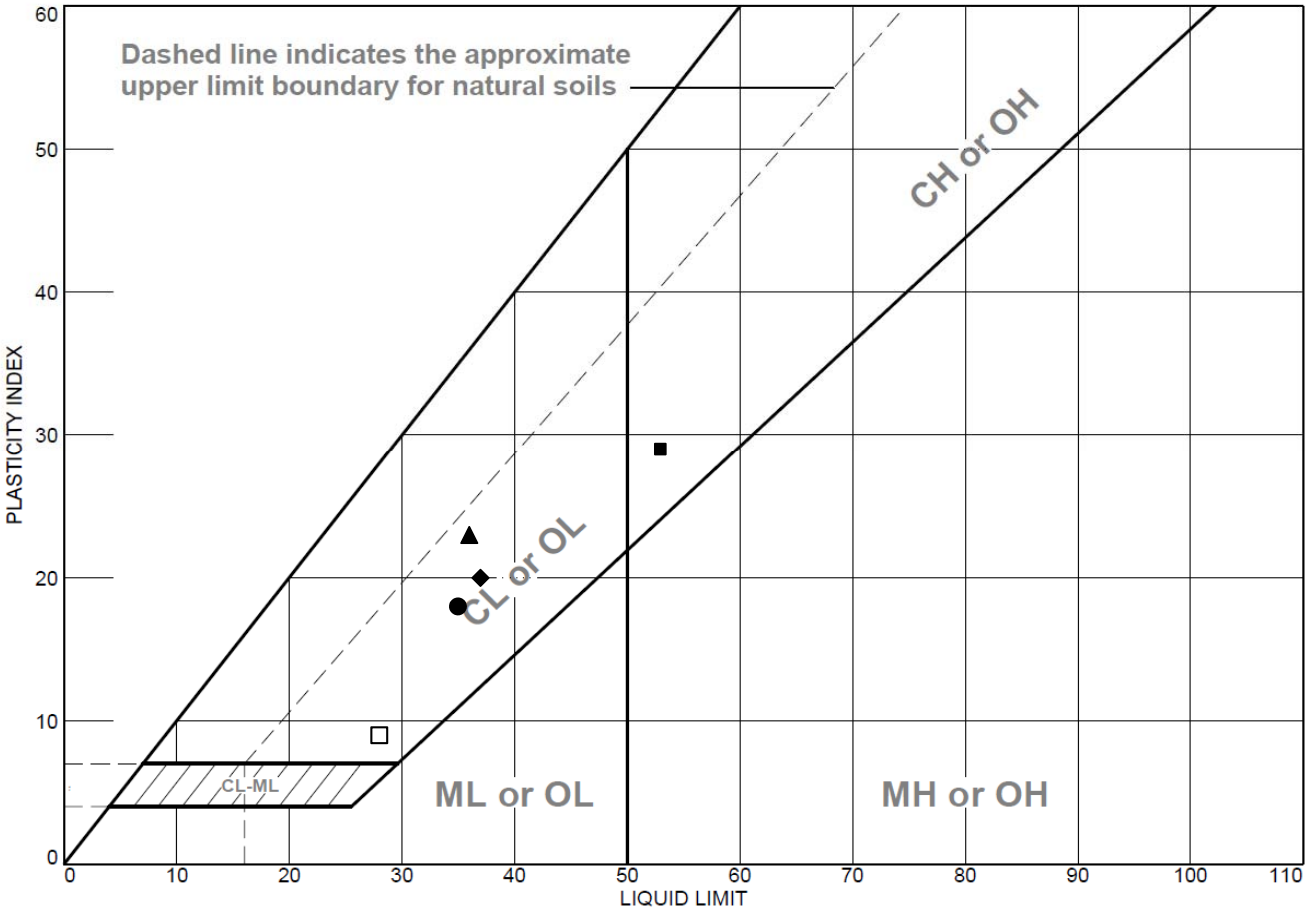
■ PI: ASTM D4318, Wet Method

ENGEO
Expect Excellence

Tested By: M. Quasem Checked By: G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
▲	See exploration logs	36	13	23			
◆	See exploration logs	37	17	20			
□	See exploration logs	28	19	9			
●	See exploration logs	35	17	18			
■	See exploration logs	53	24	29			

Project No.: 15929.000.000 **Date:** 4/26/2019
Project Name: Avenues SV
Project location: San Jose, California
Client: Avenues World Holdings LLC

▲ **Sample Number:** 1-B3@15.5-16 **Depth:** 15.5-16.0'
◆ **Sample Number:** 1-B4@3-3.5 **Depth:** 3.0-3.5'
□ **Sample Number:** 1-B4@30-31.5 **Depth:** 30.0-31.5'
● **Sample Number:** 1-B5@5-5.5 **Depth:** 5.0-5.5'
■ **Sample Number:** 1-B6@3.5-4 **Depth:** 3.5-4.0'

Remarks:

▲ PI: ASTM D4318, Wet Method
◆ PI: ASTM D4318, Wet Method
□ PI: ASTM D4318, Wet Method
● PI: ASTM D4318, Wet Method
■ PI: ASTM D4318, Wet Method

ENGEO
Expect Excellence

Tested By: M. Quasem Checked By: G. Criste

Test Location: 3420 Fostoria Way Ste. E, Danville, CA 94526

Isotropic Unconsolidated Undrained Triaxial Test

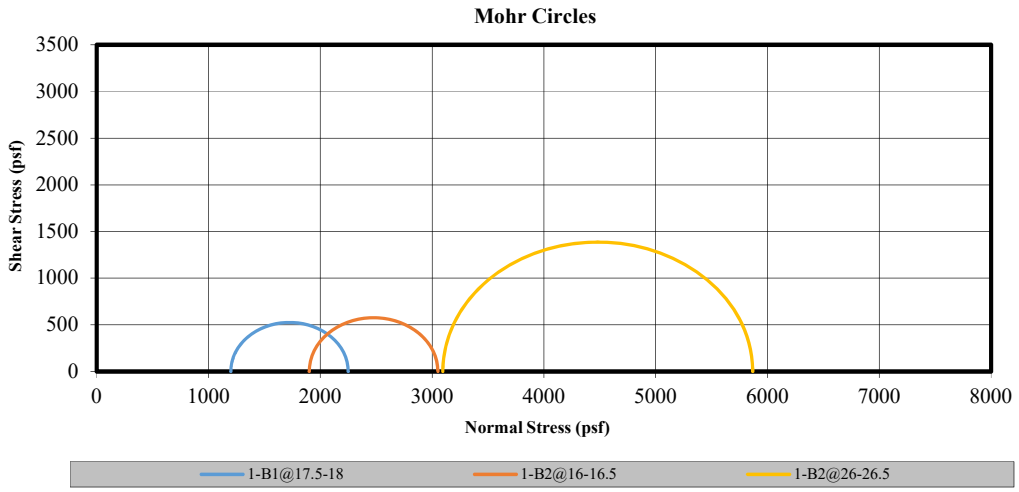
ASTM D2850

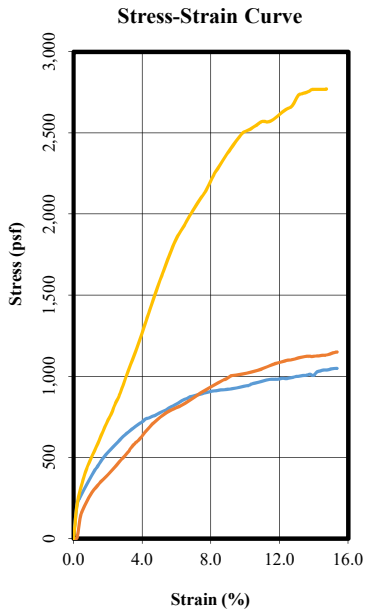
Date: 04/26/19

Checked By: G. Criste

Date: 4/26/2019

Tested By: M. Quasem





Before Test	Specimen 1-B1@17.5-18	1-B2@16-16.5	1-B2@26-26.5	
Water Content (%)	22.65	24.89	26.35	
Dry Density (pcf)	100.90	100.30	98.80	
Saturation (%)	90.29	97.66	99.60	
Void Ratio	0.68	0.69	0.72	
Diameter (in)	2.420	2.378	2.359	
Height (in)	5.009	5.038	4.965	
Height-to-Diameter Ratio	2.070	2.119	2.105	
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	
Plastic Limit	-	-	-	
ASTM D854 - Assumed				
Specific Gravity	2.720	2.720	2.720	
After Test	1-B1@17.5-18	1-B2@16-16.5	1-B2@26-26.5	
Water Content (%)	22.65	24.89	26.35	
Saturation (%)	90.29	97.66	99.59	
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	1049.8	1150.7	2771.6	
Axial Strain @ Failure (%)	15.347	15.351	14.734	
	Cell Pressure			
Cell (psf)	1199.5	1900.8	3096.0	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	2249.4	3051.5	5867.6	
σ3 (psf)	1199.5	1900.8	3096.0	
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		524.9	575.4	1385.8	
Friction Angle Ø	n/a		n/a	n/a	n/a	

Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Date: 04/26/19

Checked By: G. Criste

Date: 4/26/2019

Tested By: M. Quasem

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B1@17.5-18	SAMPLE NUMBER: 1-B2@16-16.5
	
SAMPLE NUMBER: 1-B2@26-26.5	**SAMPLE NUMBER:**
	

Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

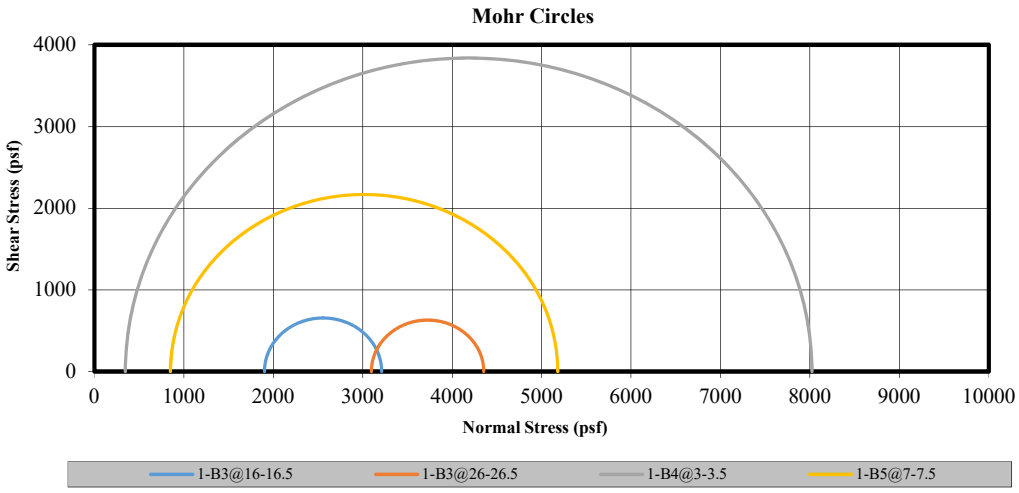
ASTM D2850

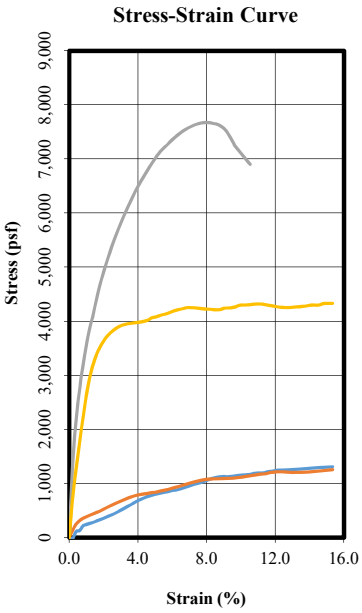
Date: 04/26/19

Checked By: G. Criste

Date: 4/26/2019

Tested By: M. Quasem





Before Test	Specimen 1-B3@16-16.5	1-B3@26-26.5	1-B4@3-3.5	1-B5@7-7.5
Water Content (%)	26.38	28.99	20.44	14.18
Dry Density (pcf)	97.40	95.10	108.20	107.80
Saturation (%)	96.56	100.29	97.63	67.03
Void Ratio	0.74	0.79	0.57	0.58
Diameter (in)	2.358	2.390	2.411	2.392
Height (in)	5.018	4.970	5.092	4.991
Height-to-Diameter Ratio	2.128	2.079	2.112	2.087
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	-
Plastic Limit	-	-	-	-
ASTM D854 - Measured				
Specific Gravity	2.720	2.720	2.720	2.720
After Test	1-B3@16-16.5	1-B3@26-26.5	1-B4@3-3.5	1-B5@7-7.5
Water Content (%)	26.38	28.99	20.49	14.18
Saturation (%)	96.56	100.00	97.88	67.03
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	1310.4	1257.2	7676.6	4330.8
Axial Strain @ Failure (%)	15.338	15.342	8.052	15.027
	Cell Pressure			
Cell (psf)	1900.8	3096.0	345.6	849.6
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	3211.2	4353.2	8022.2	5180.4
σ3 (psf)	1900.8	3096.0	345.6	849.6
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		655.2	628.6	3838.3	2165.4
Friction Angle Ø	n/a		n/a	n/a	n/a	n/a

Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Date: 04/26/19

Checked By: G. Criste

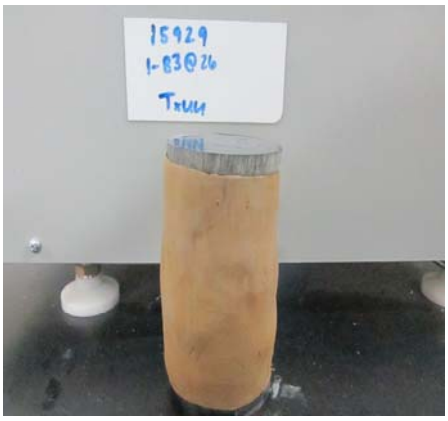
Date: 4/26/2019

Tested By: M. Quasem

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B3@16-16.5

SAMPLE NUMBER: 1-B3@26-26.5



SAMPLE NUMBER: 1-B4@3-3.5

SAMPLE NUMBER: 1-B5@7-7.5

Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

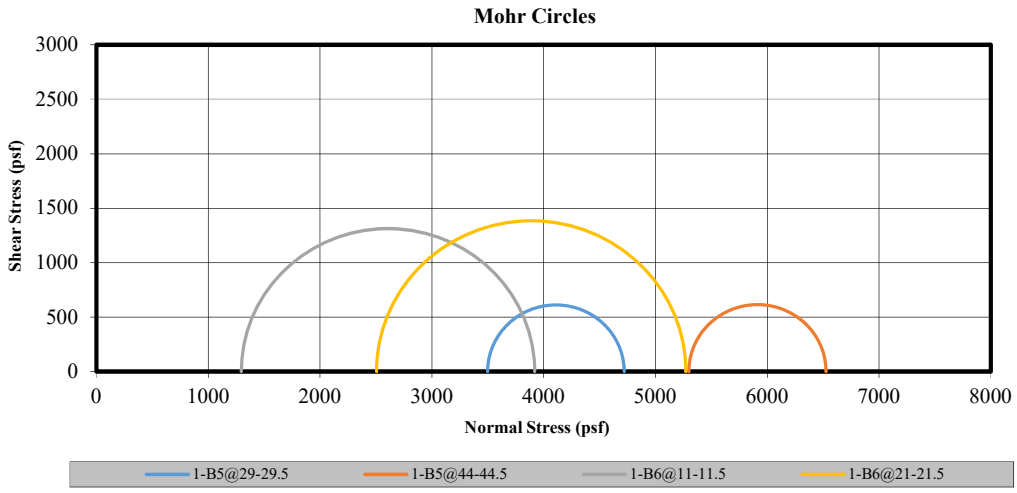
ASTM D2850

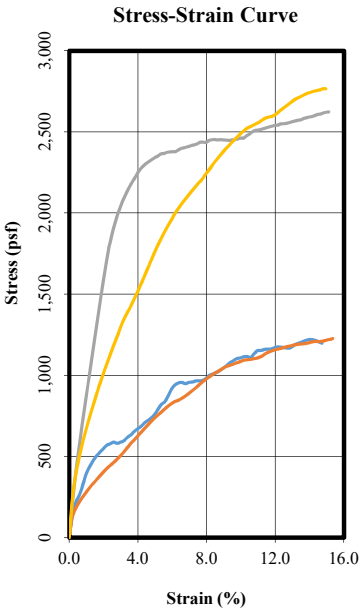
Date: 04/26/19

Checked By: G. Criste

Date: 4/26/2019

Tested By: M. Quasem





Before Test	Specimen 1-B5@29-29.5	1-B5@44-44.5	1-B6@11-11.5	1-B6@21-21.5
Water Content (%)	25.68	25.60	19.22	27.05
Dry Density (pcf)	93.90	96.10	112.50	99.20
Saturation (%)	86.46	90.71	100.40	99.90
Void Ratio	0.81	0.77	0.51	0.76
Diameter (in)	2.410	2.400	2.830	2.380
Height (in)	5.020	5.044	6.010	5.070
Height-to-Diameter Ratio	2.083	2.102	2.124	2.130
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	-
Plastic Limit	-	-	-	-
ASTM D854 - Measured				
Specific Gravity	2.720	2.720	2.750	2.790
After Test	1-B5@29-29.5	1-B5@44-44.5	1-B6@11-11.5	1-B6@21-21.5
Water Content (%)	25.68	25.60	19.22	27.05
Saturation (%)	86.46	90.71	100.00	99.90
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	1221.1	1227.1	2623.7	2766.9
Axial Strain @ Failure (%)	13.943	15.340	15.110	14.794
	Cell Pressure			
Cell (psf)	3499.2	5299.2	1296.0	2505.6
Back (psf)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psf)	4720.3	6526.3	3919.7	5272.5
σ3 (psf)	3499.2	5299.2	1296.0	2505.6
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		610.6	613.5	1311.9	1383.5
Friction Angle Ø	n/a		n/a	n/a	n/a	n/a

Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Date: 04/26/19

Checked By: G. Criste

Date: 4/26/2019

Tested By: M. Quasem

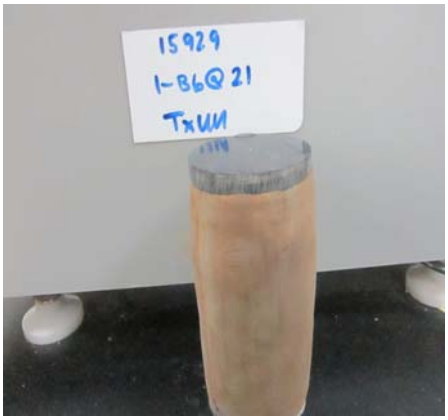
SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B5@29-29.5

SAMPLE NUMBER: 1-B5@44-44.5

SAMPLE NUMBER: 1-B6@11-11.5

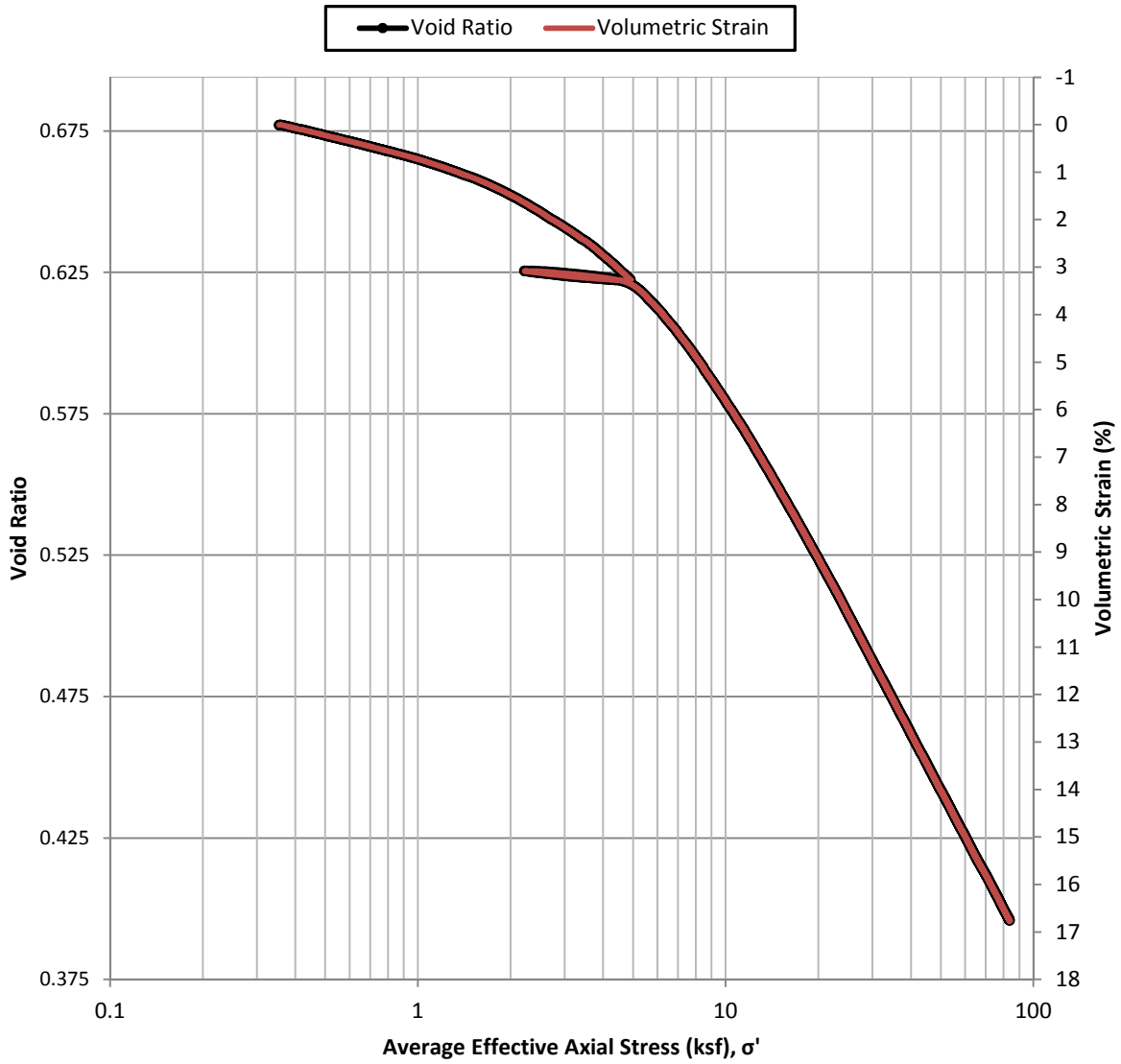
SAMPLE NUMBER: 1-B6@21-21.5



Project Information	
Project Name:	Avenues SV
Project Number:	15929.000.000
Project Location:	San Jose, CA
Client:	Avenues World Holdings, LLC
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



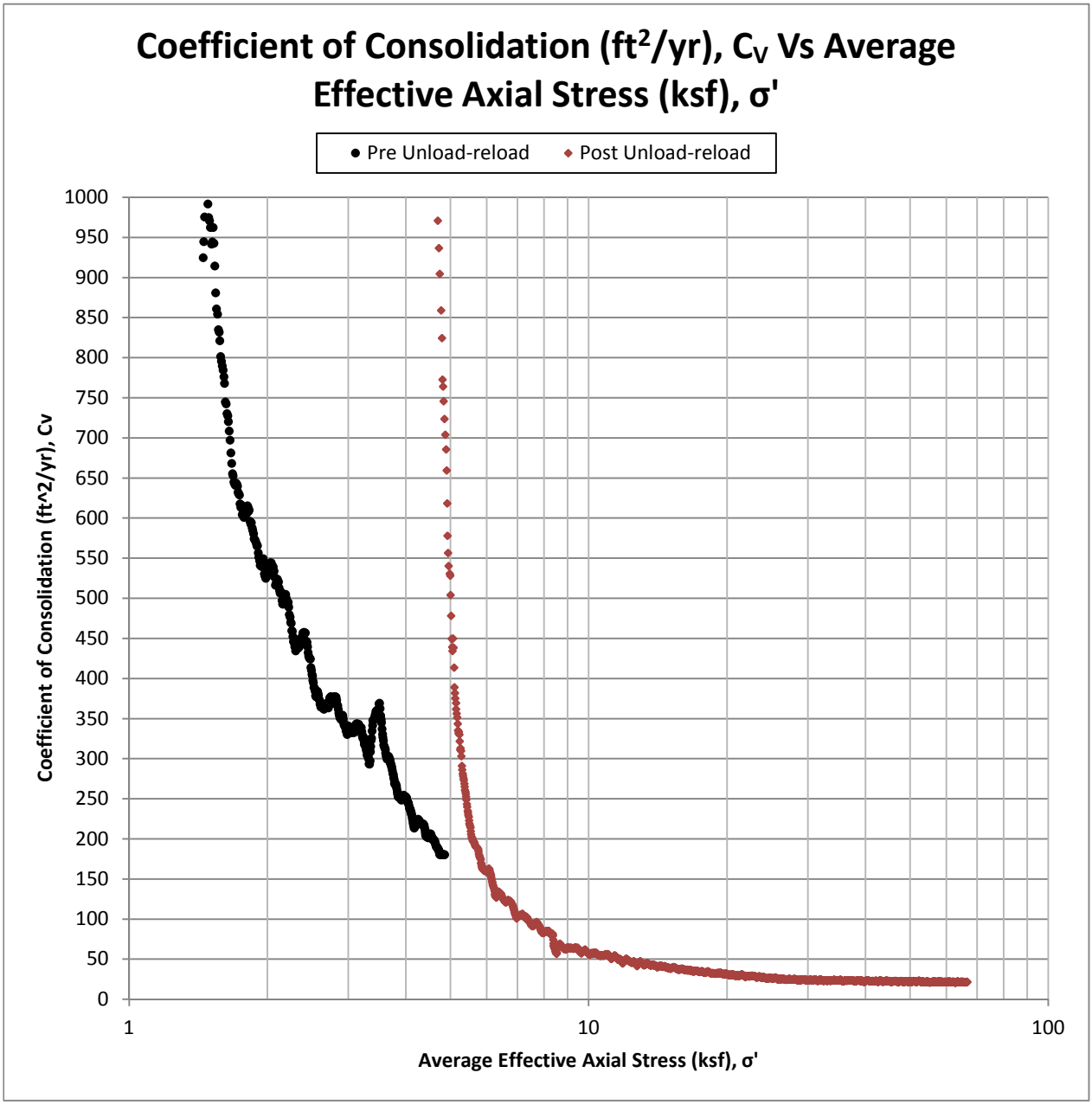
ASTM D2216					Report Date: 4/25/2019
	As Received	Final	ASTM D4318 - Wet Method		
Moisture (%):	21.66%	15.46%	Liquid Limit:		
Dry Density (pcf):	100.54	120.80	Plastic Limit:		
Saturation (%):	86.55%	100.00%	ASTM D854 - Measured		
Void Ratio:	0.6773	0.3959	Specific Gravity:	2.706	
			Soil Description:	See exploration logs	

Project Number:	15929.000.000	Depth:	21-21.5 ft
Sample Number:	1-B2 @ 20-22.5	Boring #:	1-B2
Project Name:	Avenues SV		
Client:	Avenues World Holdings LLC		
Location:	San Jose, California		
Tested By:	D. Seibold	Reviewed By:	Y. Zepeda

Remarks:

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2216				
	As Received	Final	ASTM D4318 - Wet Method	
Moisture (%):	21.66%	15.46%	Liquid Limit:	
Dry Density (pcf):	100.54	120.80	Plastic Limit:	
Saturation (%):	86.55%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.6773	0.3959	Specific Gravity:	2.706
			Soil Description:	See exploration logs

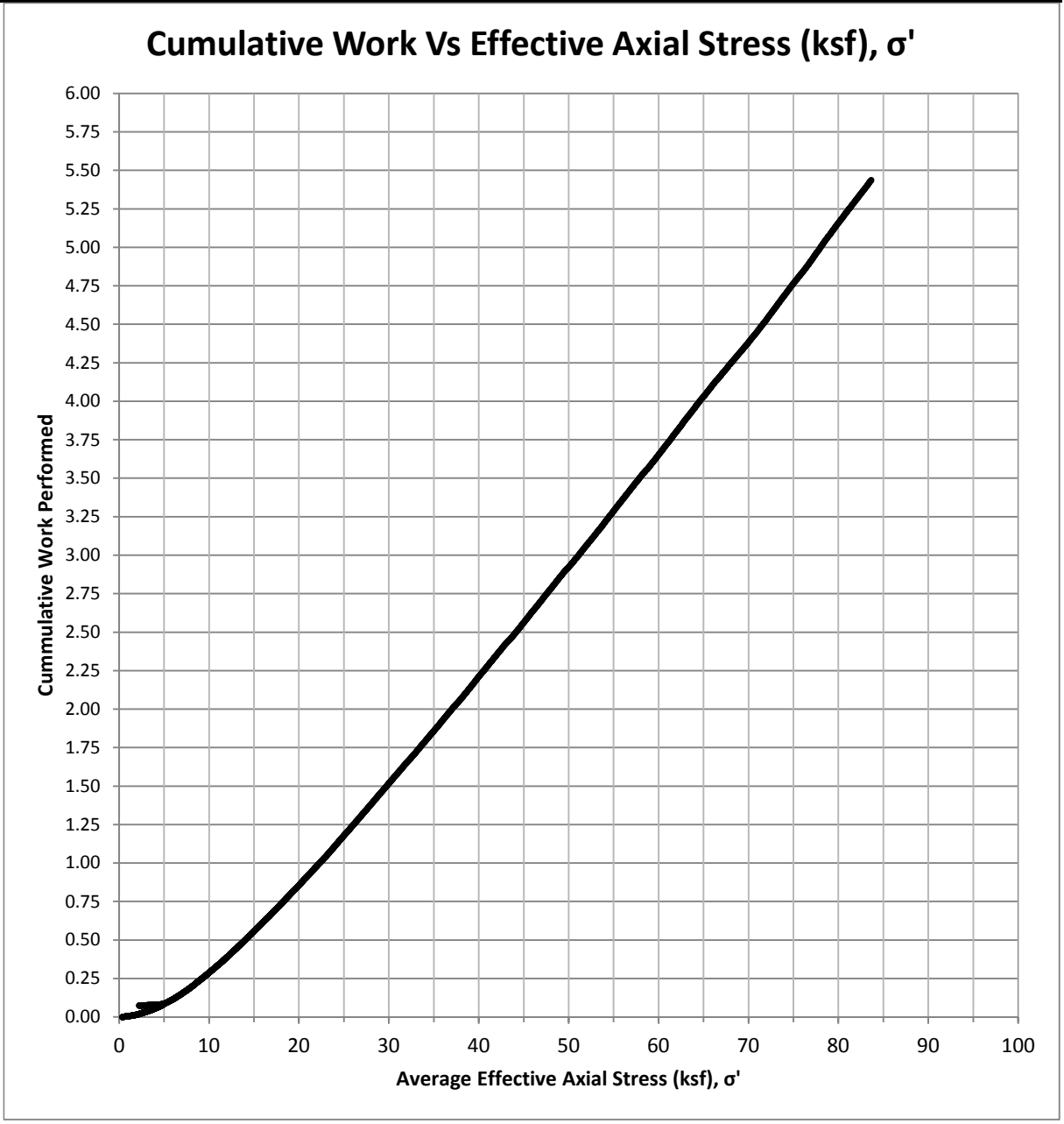
Report Date: 4/25/2019

Project Number:	15929.000.000	**Depth:**	21-21.5 ft
Sample Number:	1-B2 @ 20-22.5	**Boring #:**	1-B2
Project Name:	Avenues SV		
Client:	Avenues World Holdings LLC		
Location:	San Jose, California		
Tested By:	D. Seibold	**Reviewed By:**	Y. Zepeda

Remarks:

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2216				
	As Received	Final	ASTM D4318 - Wet Method	
Moisture (%):	21.66%	15.46%	Liquid Limit:	
Dry Density (pcf):	100.54	120.80	Plastic Limit:	
Saturation (%):	86.55%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.6773	0.3959	Specific Gravity:	2.706
			Soil Description:	See exploration logs

Report Date: 4/25/2019

Project Number:	15929.000.000	**Depth:**	21-21.5 ft
Sample Number:	1-B2 @ 20-22.5	**Boring #:**	1-B2
Project Name:	Avenues SV		
Client:	Avenues World Holdings LLC		
Location:	San Jose, California		
Tested By:	D. Seibold	**Reviewed By:**	Y. Zepeda

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Remarks:

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B2@8-8.5	soil	ND
2	1-B4@6-6.5	soil	ND
3	1-B6@6-6.5	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: Avenues SV
PROJECT NUMBER: 15929.000.000
CLIENT: Avenues World Holdings, LLC
PHASE NUMBER: 002

DATE: 04/24/19

ENGEO
Expect Excellence

Tested by: M. Quasem

Reviewed by: G. Criste

Lab Address: 3420 Fostoria Way Suite E, Danville, CA 94526. Phone No. (925) 355-9047.

APPENDIX D

LIQUEFACTION ANALYSIS

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

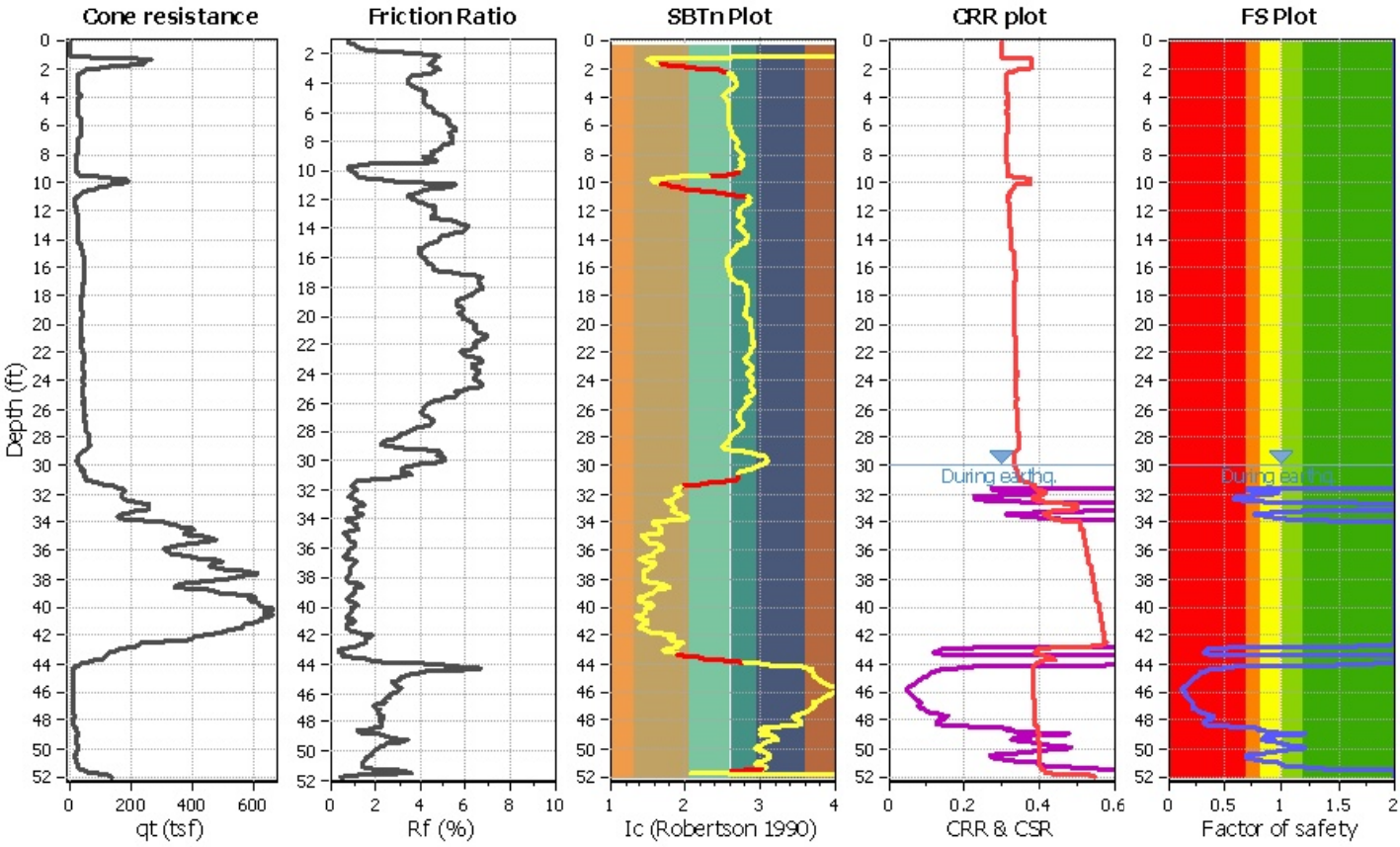
LIQUEFACTION ANALYSIS REPORT

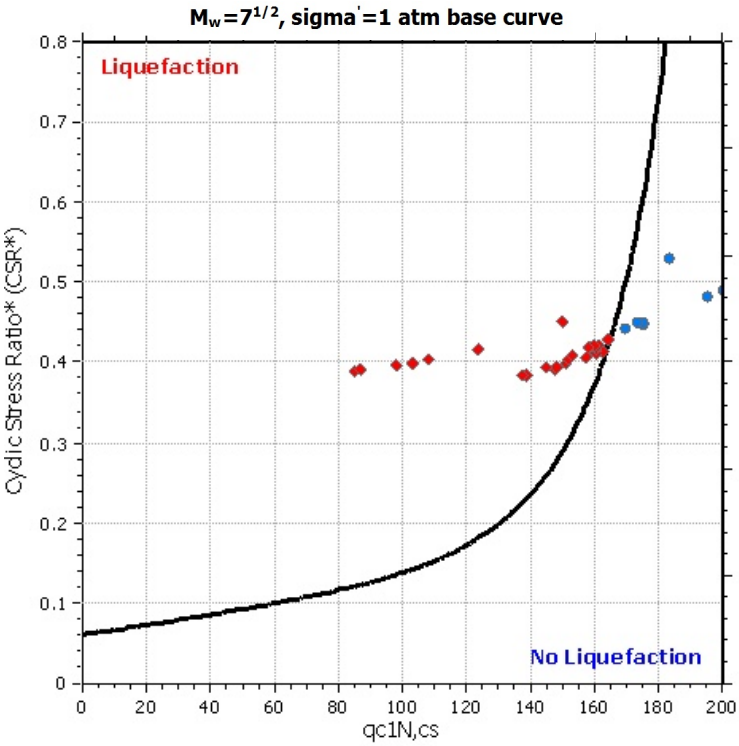
Project title : Avenues Silicon Valley **Location : San Jose, California**

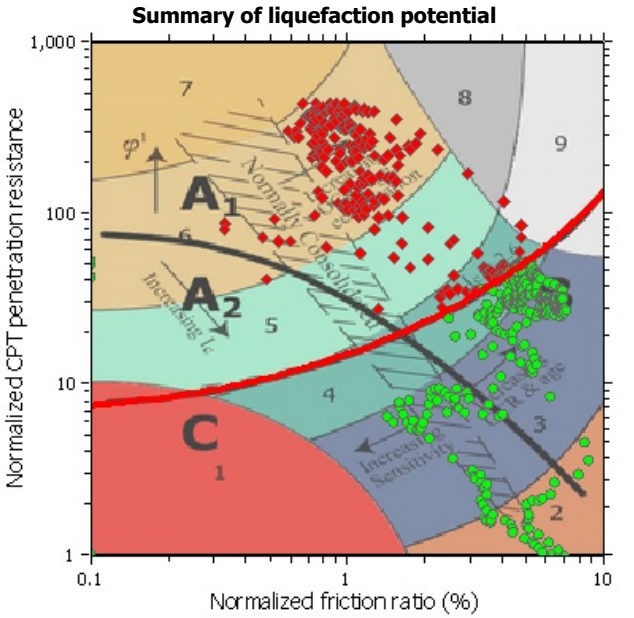
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

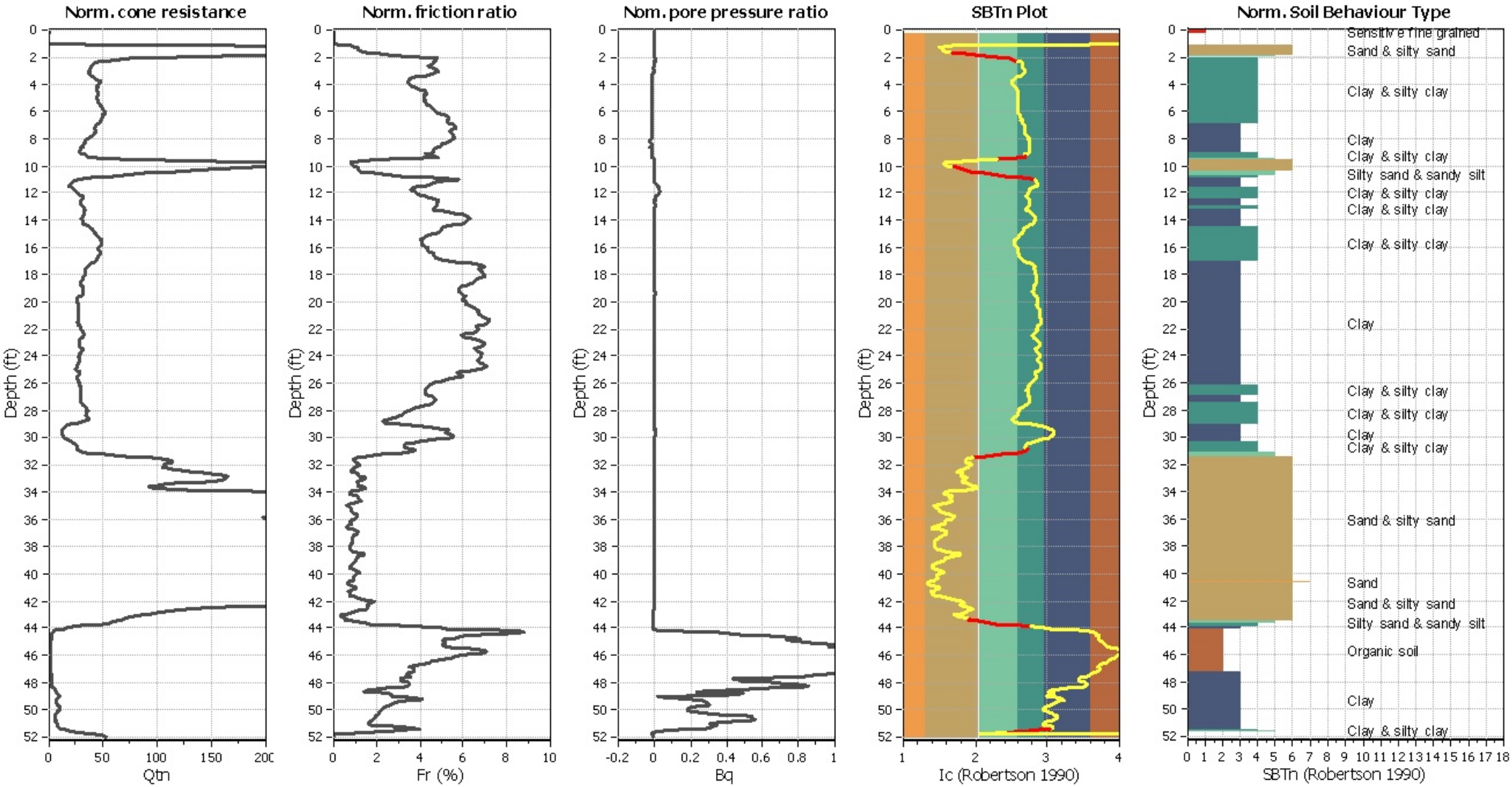






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



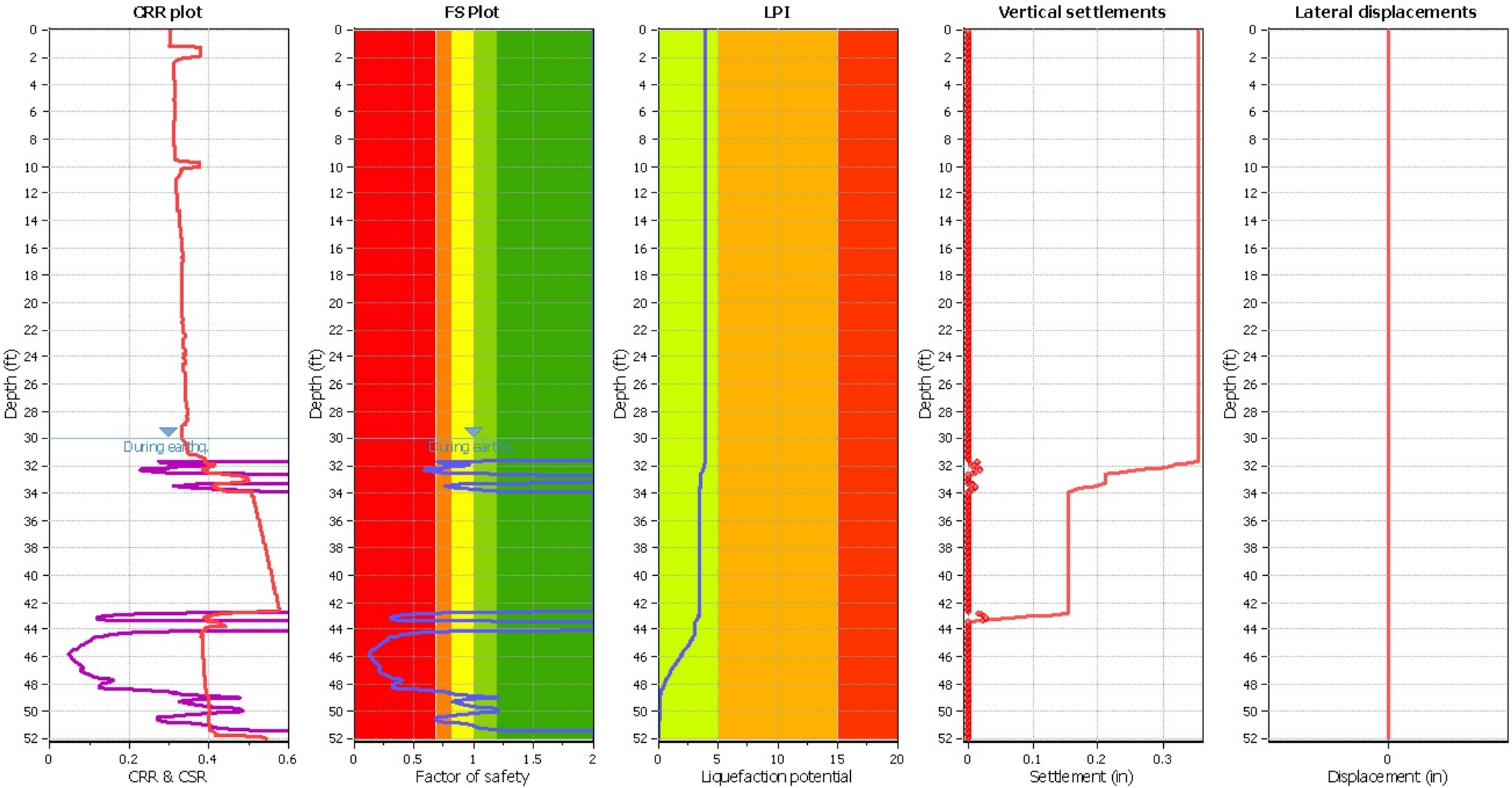
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

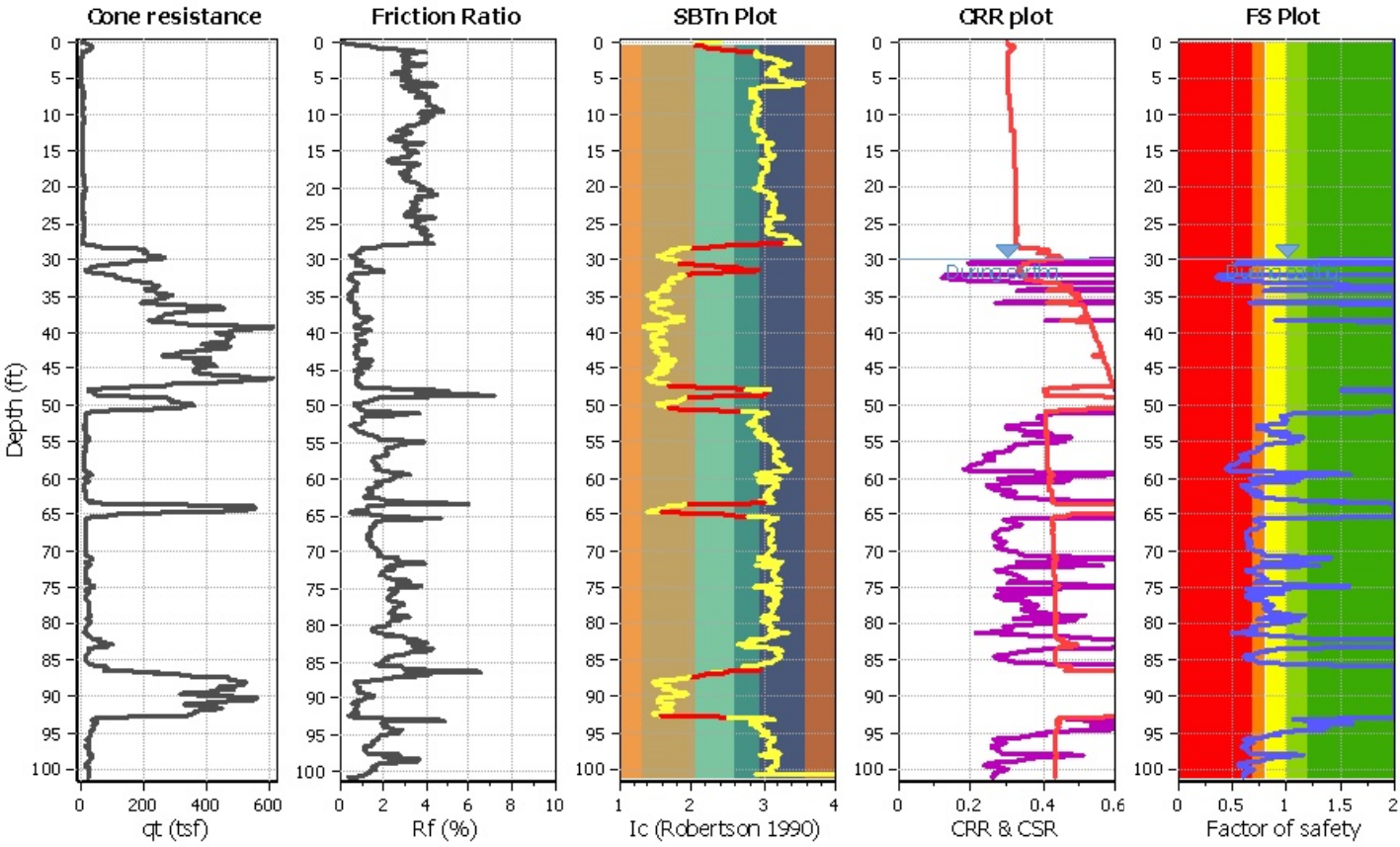
LIQUEFACTION ANALYSIS REPORT

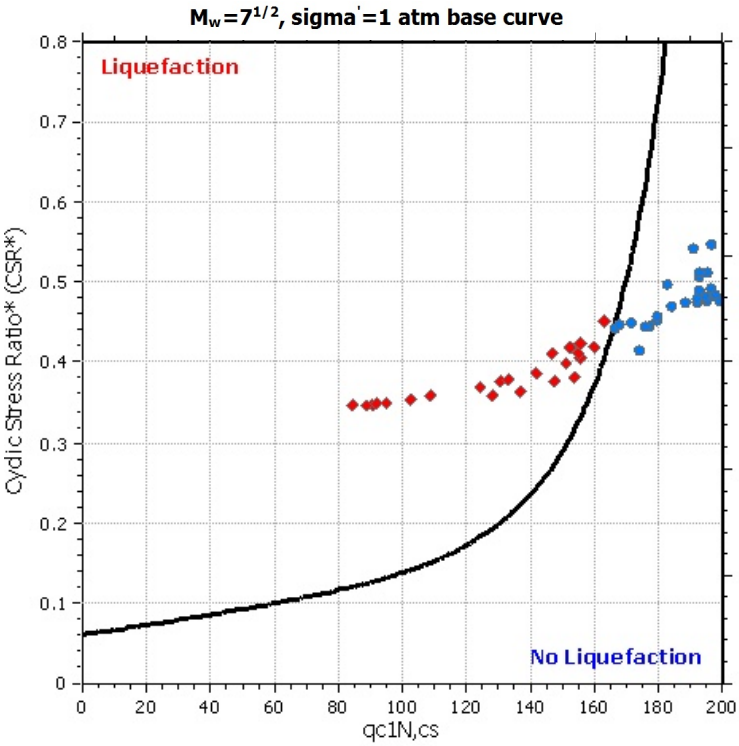
Project title : Avenues Silicon Valley **Location : San Jose, California**

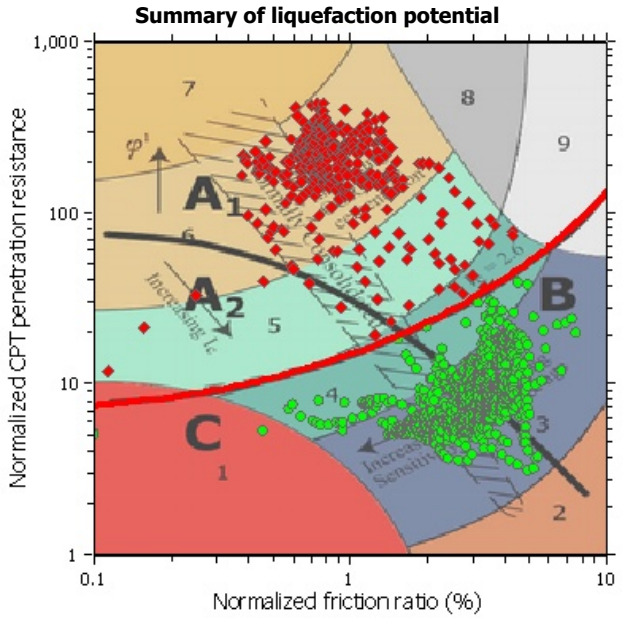
CPT file : 1-SCPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

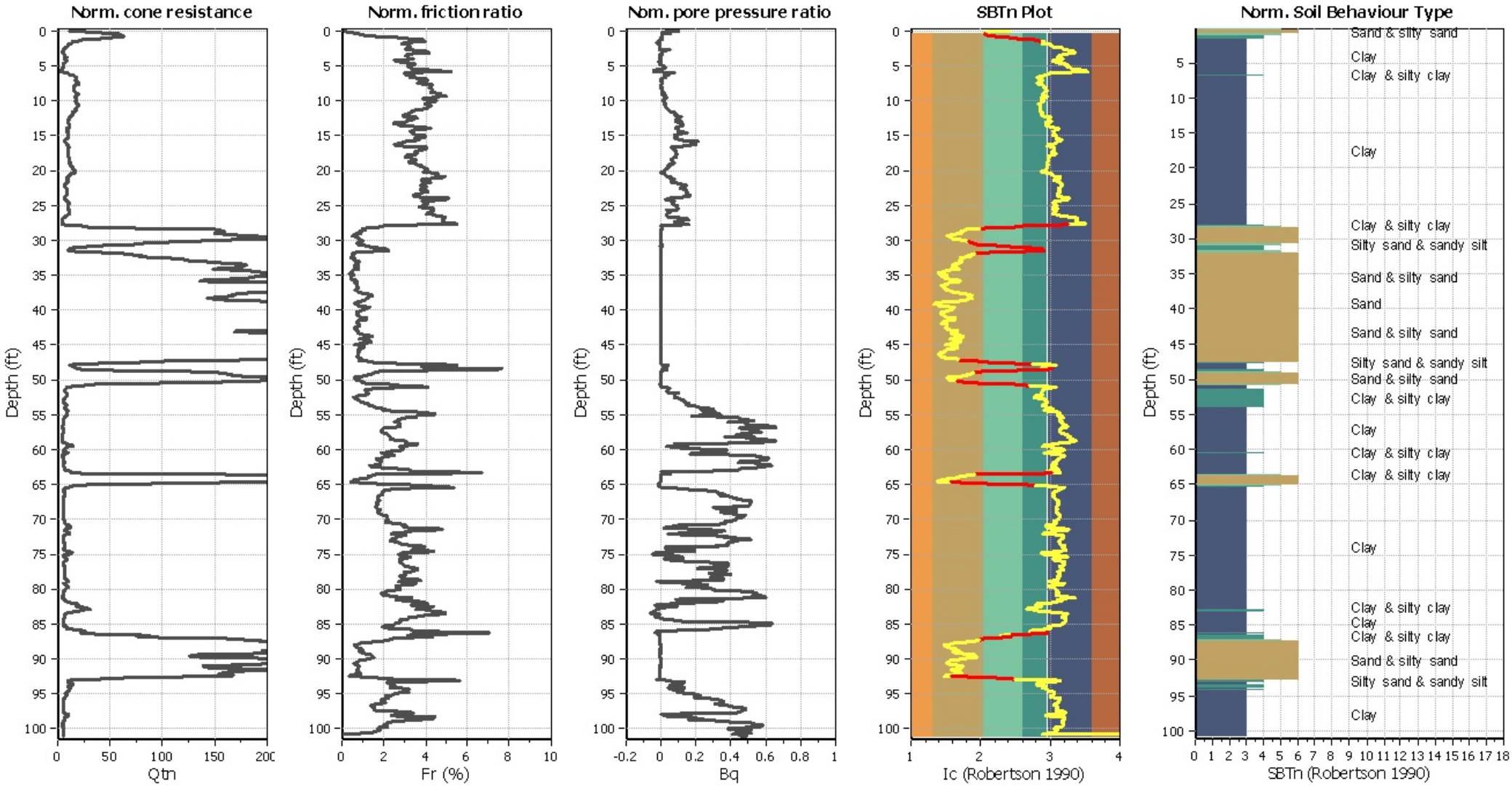






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



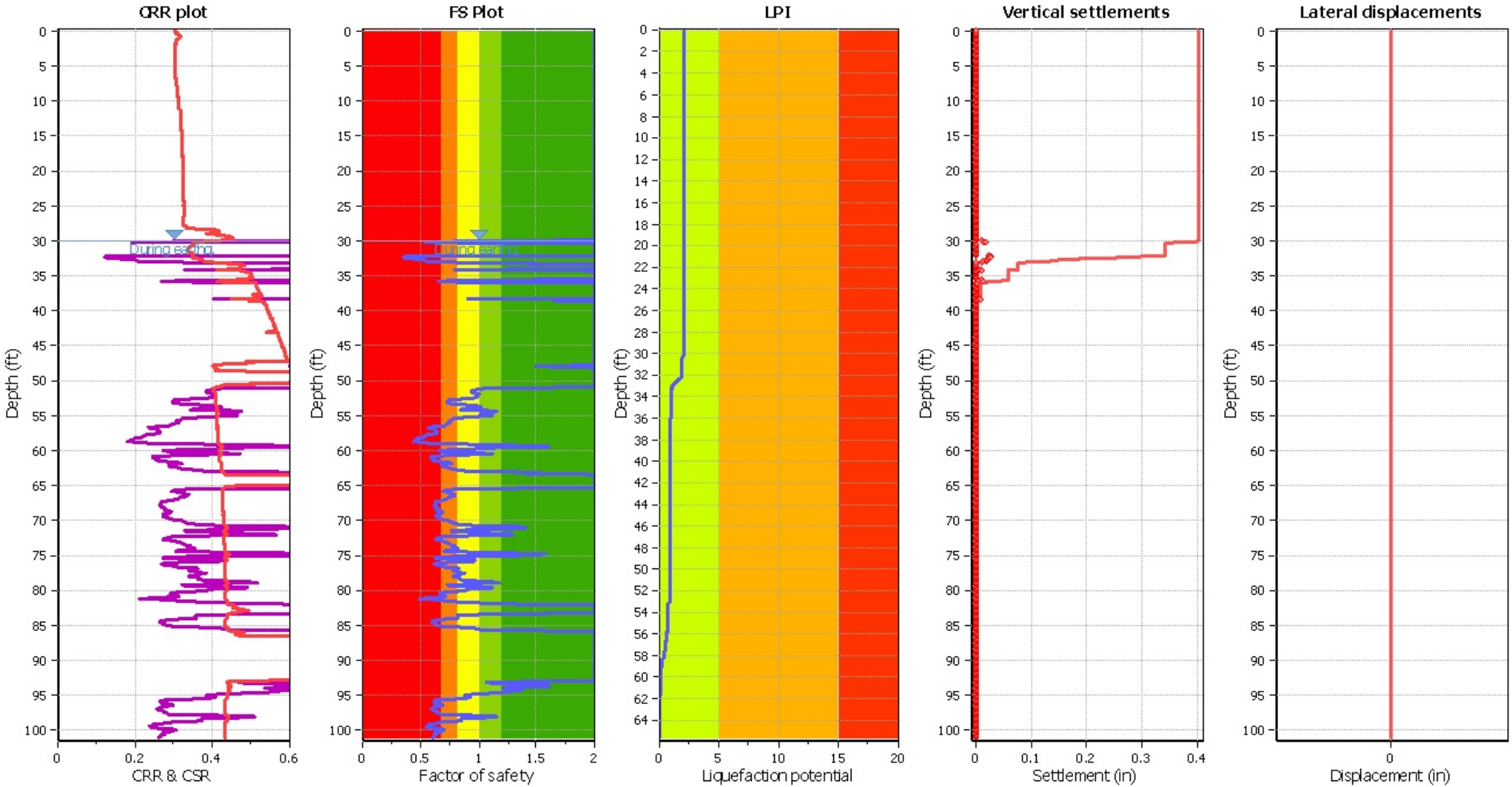
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

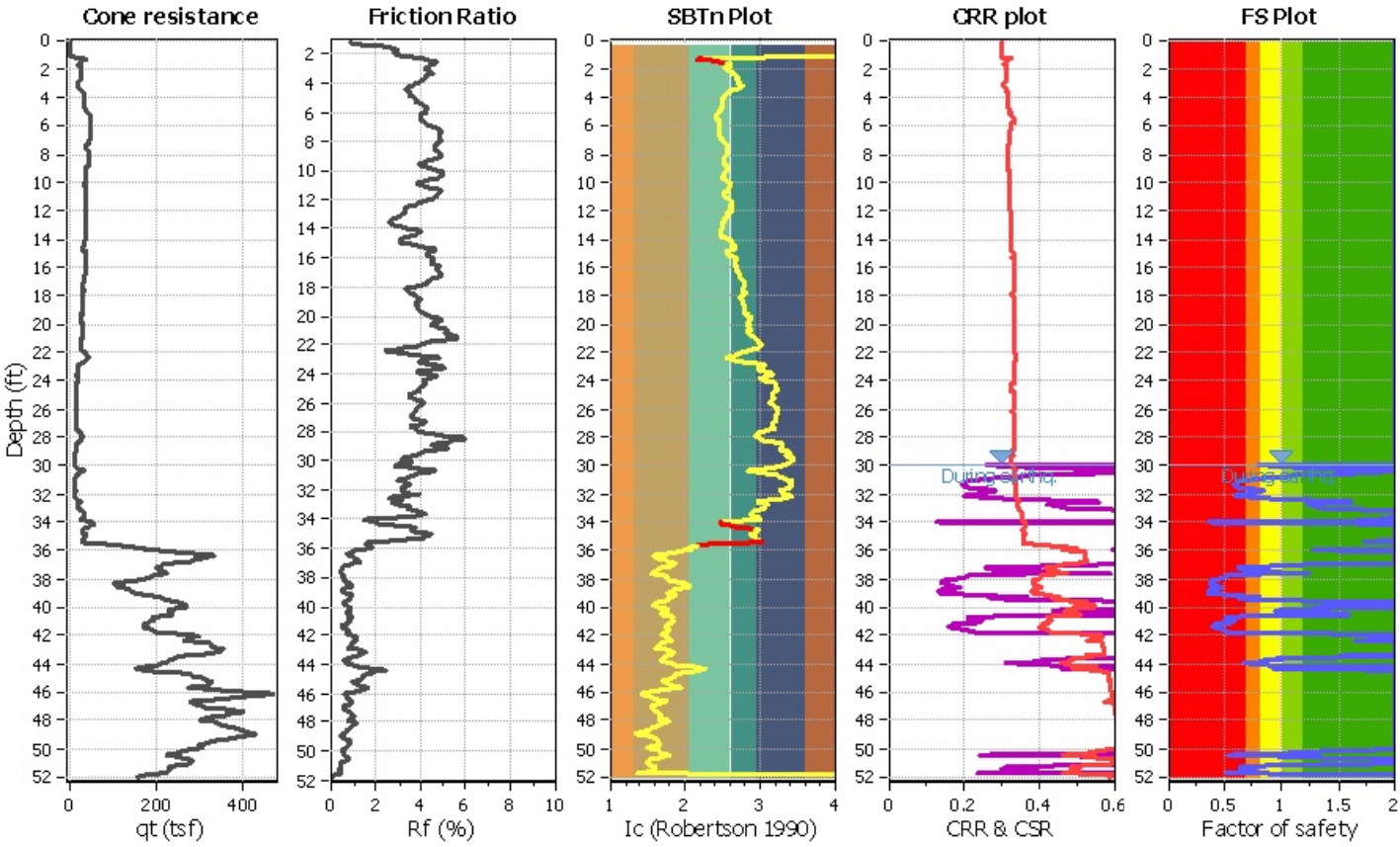
LIQUEFACTION ANALYSIS REPORT

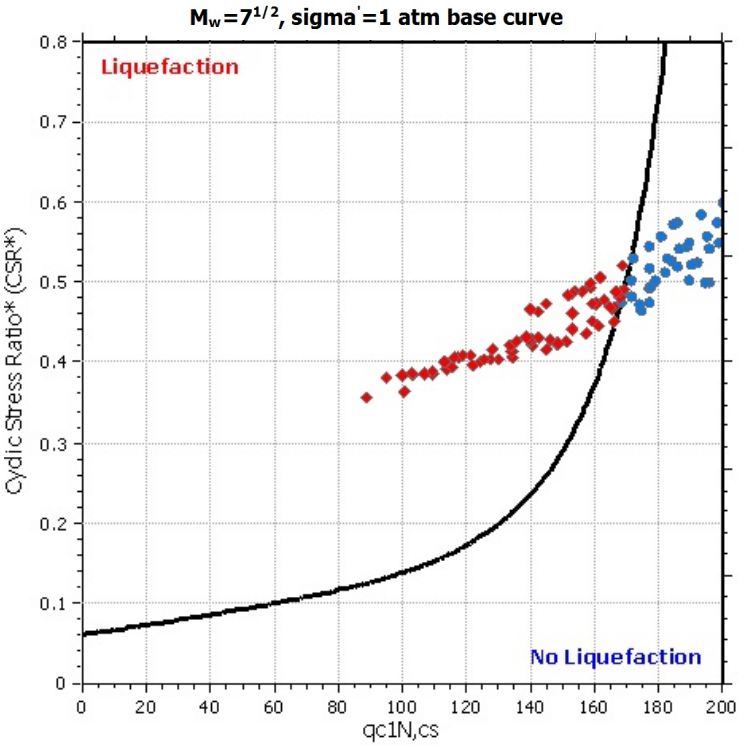
Project title : Avenues Silicon Valley **Location : San Jose, California**

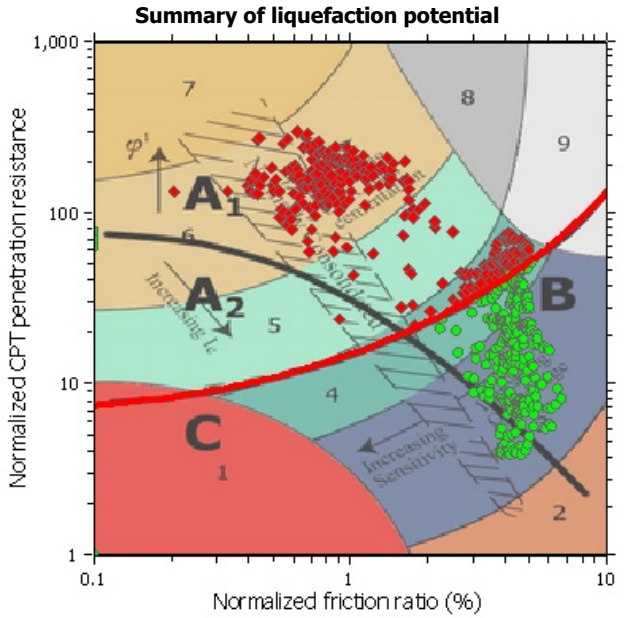
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

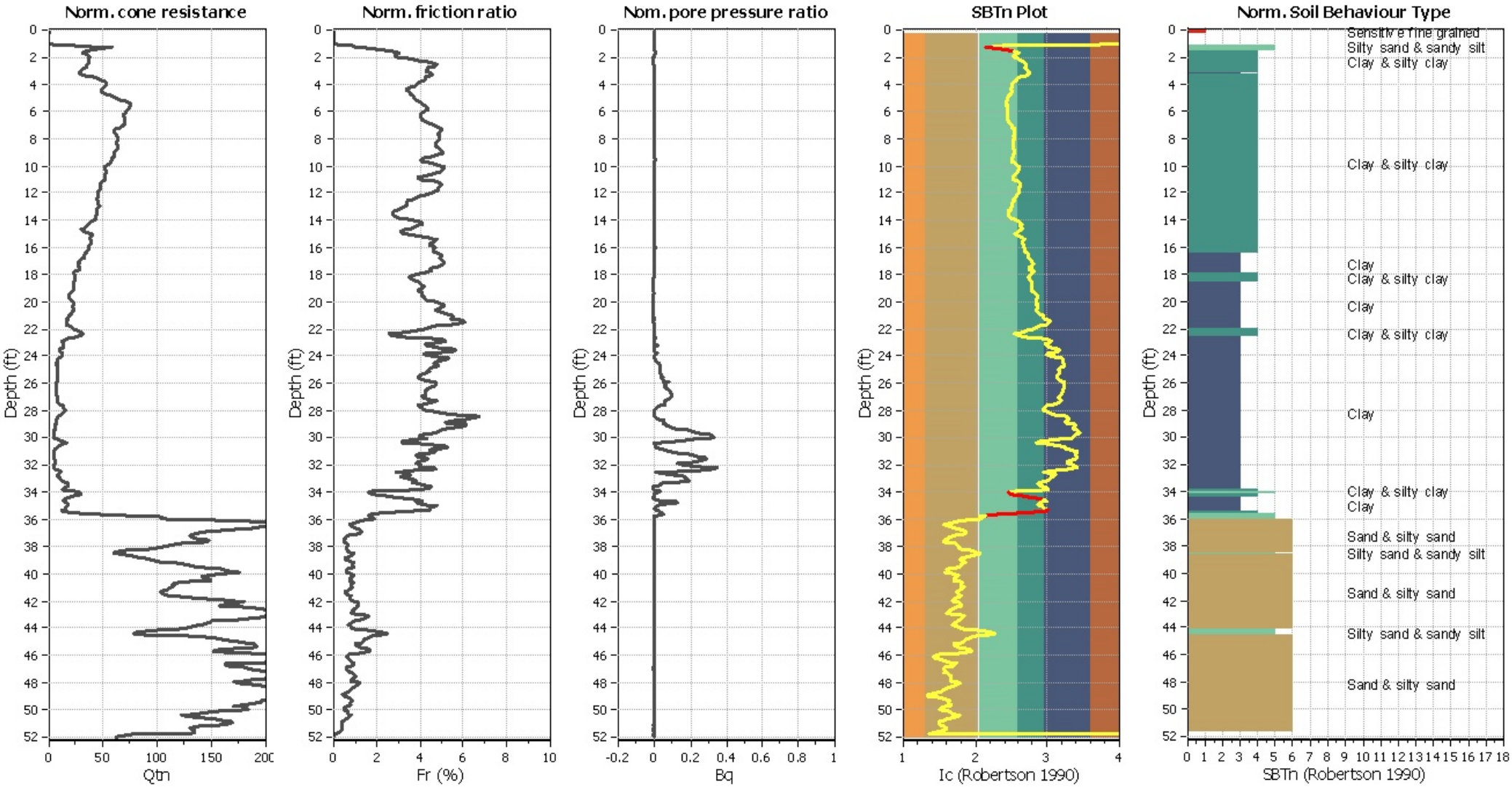






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



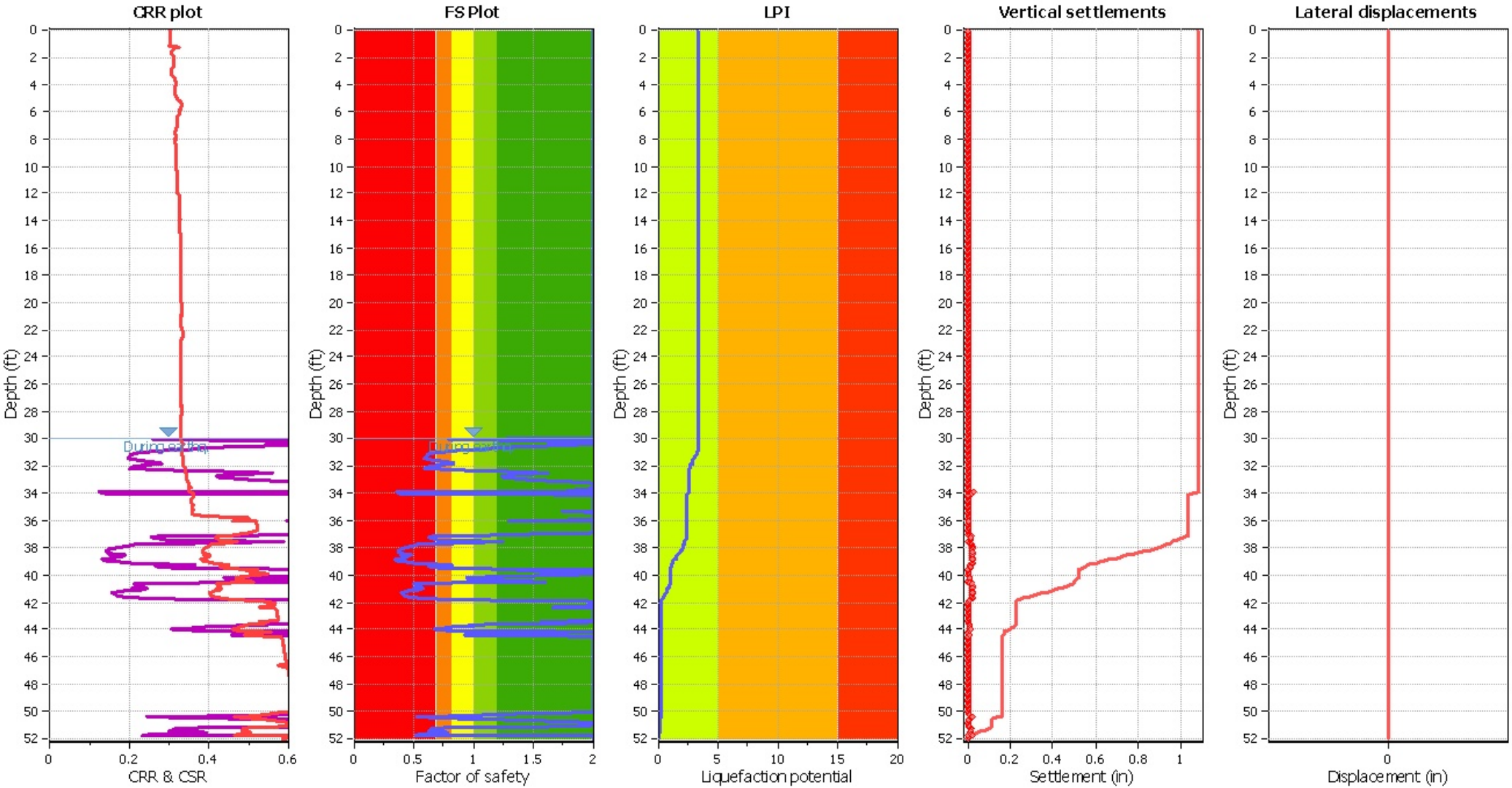
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

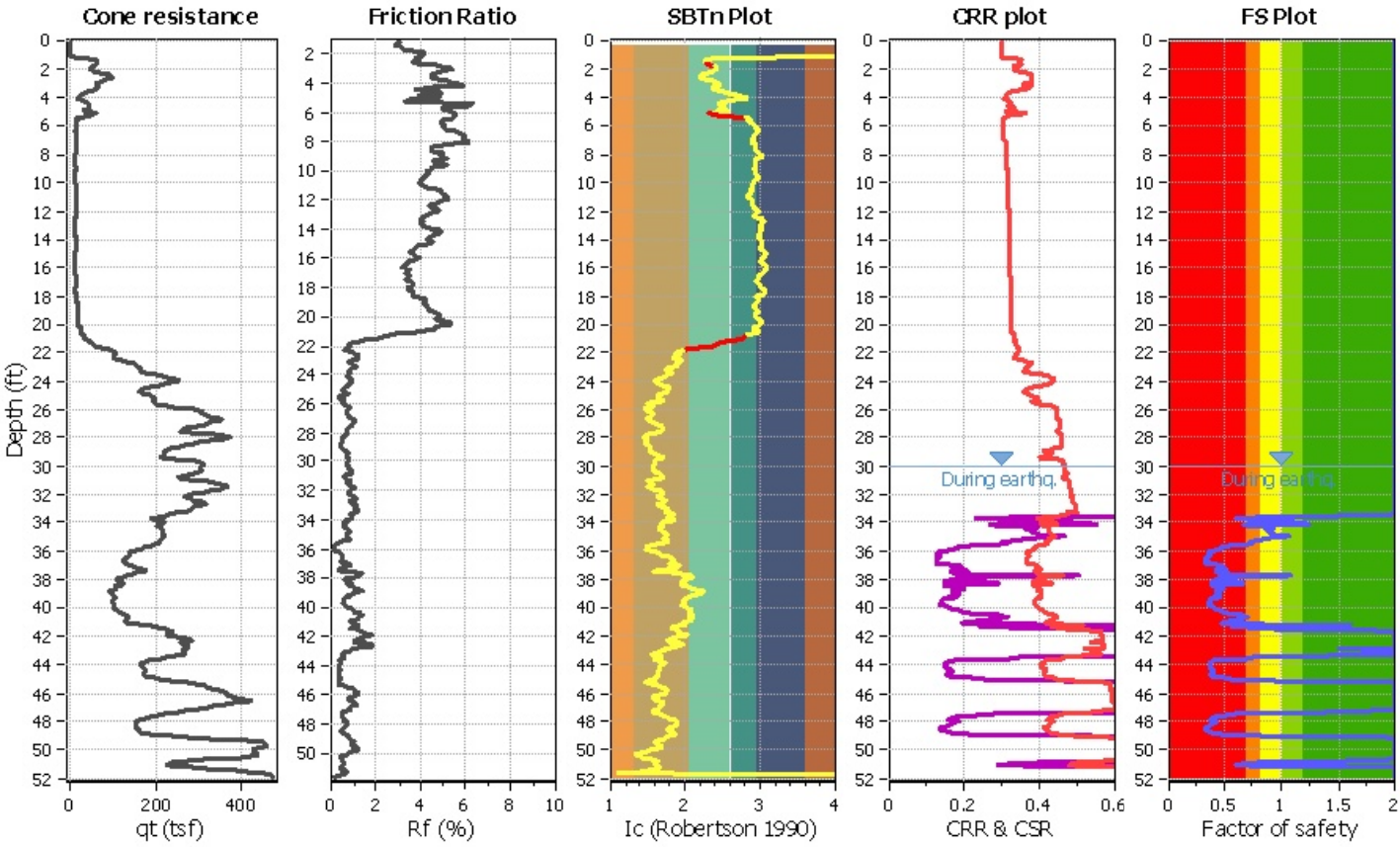
LIQUEFACTION ANALYSIS REPORT

Project title : Avenues Silicon Valley **Location : San Jose, California**

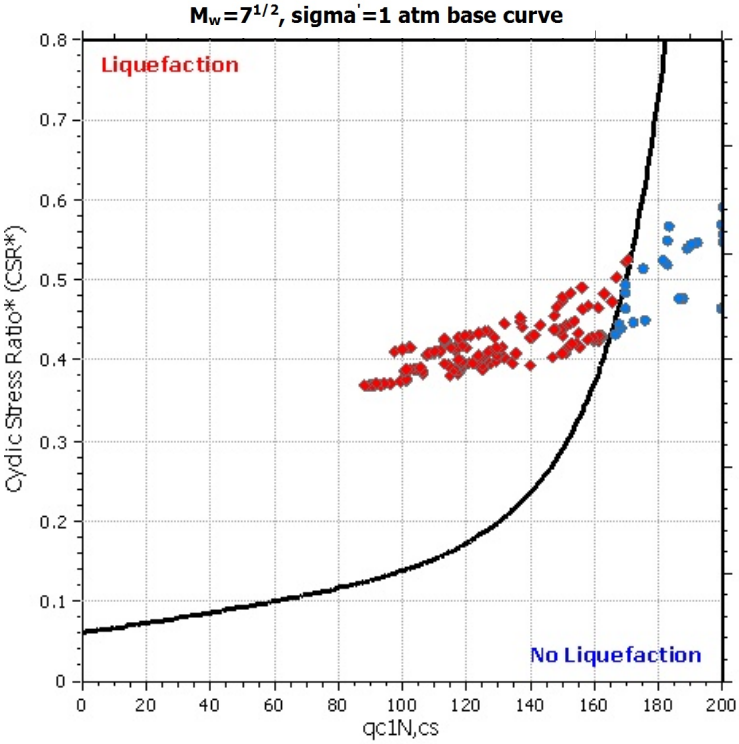
CPT file : 1-CPT4

Input parameters and analysis data

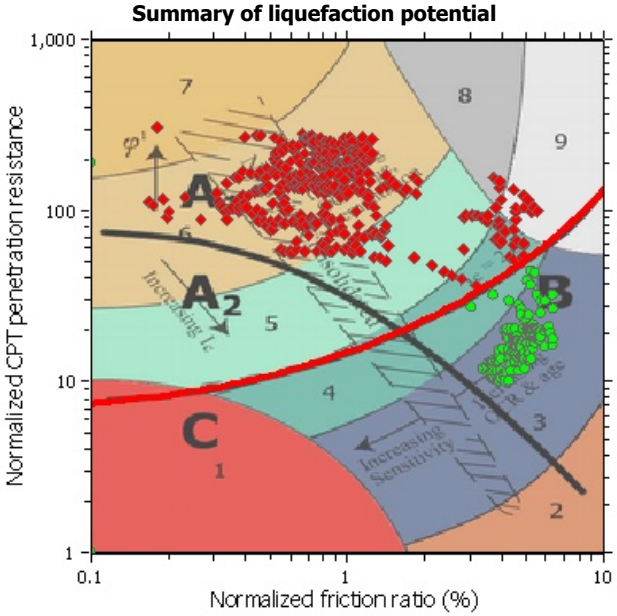
Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

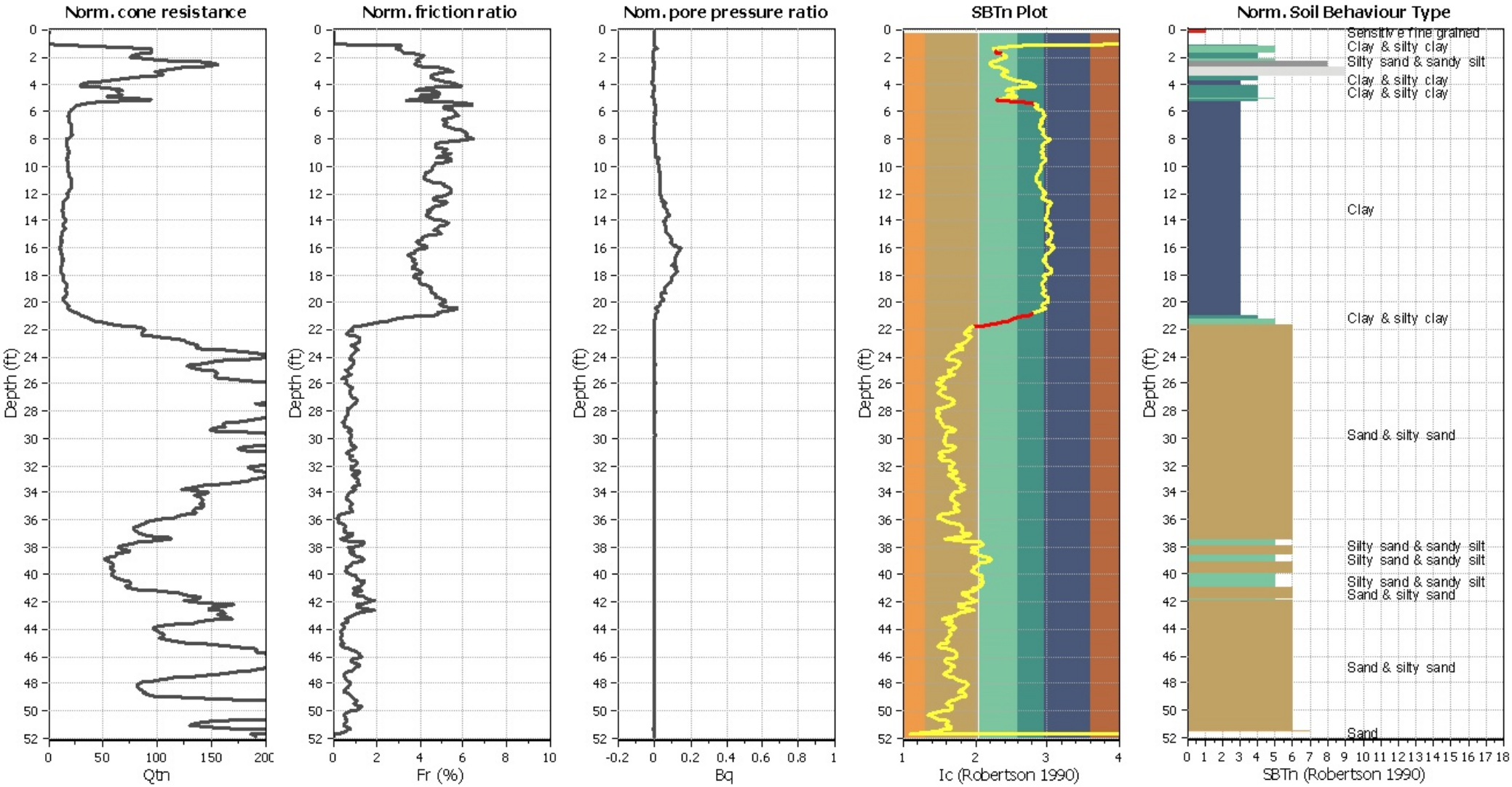


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



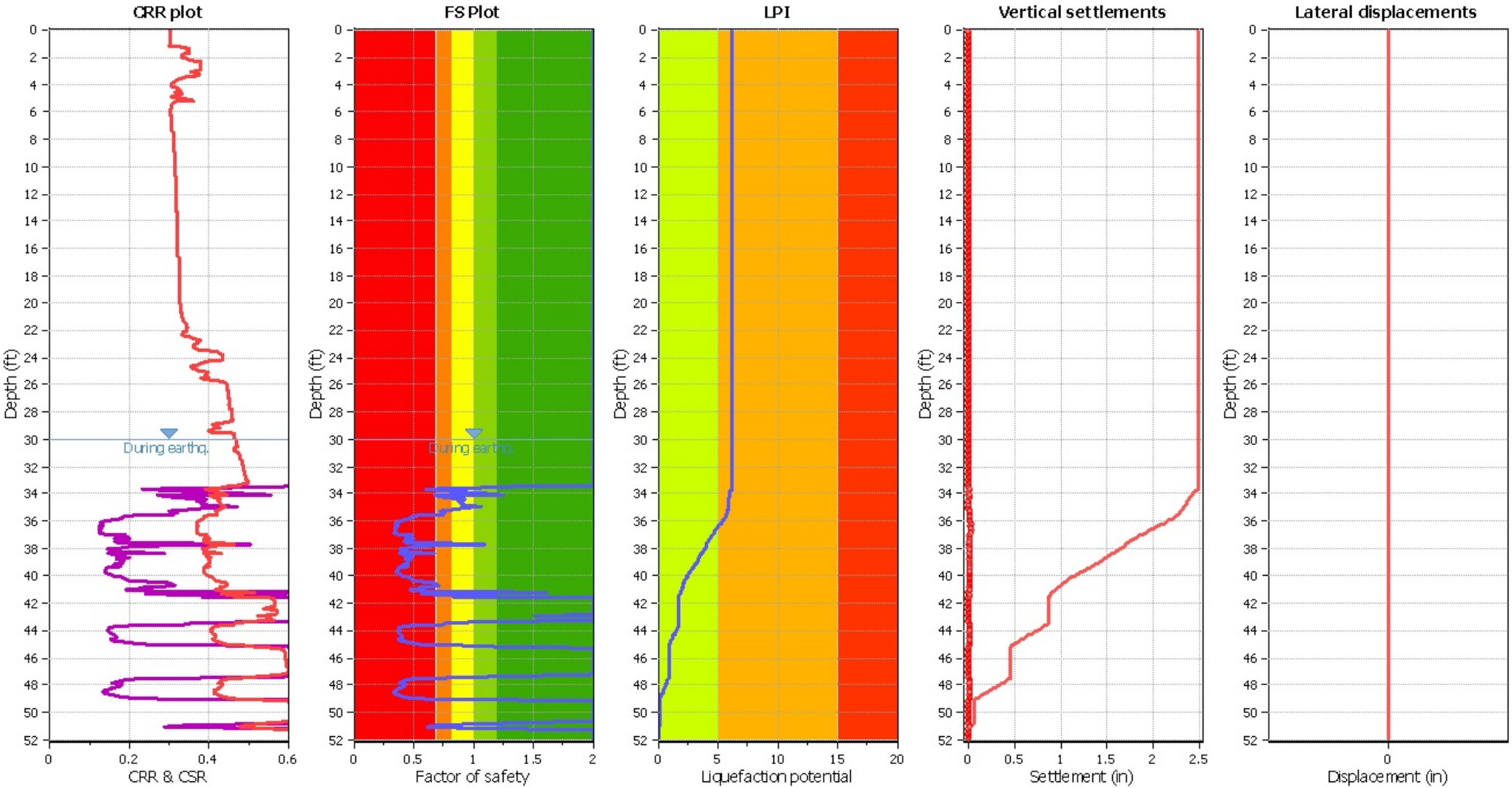
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

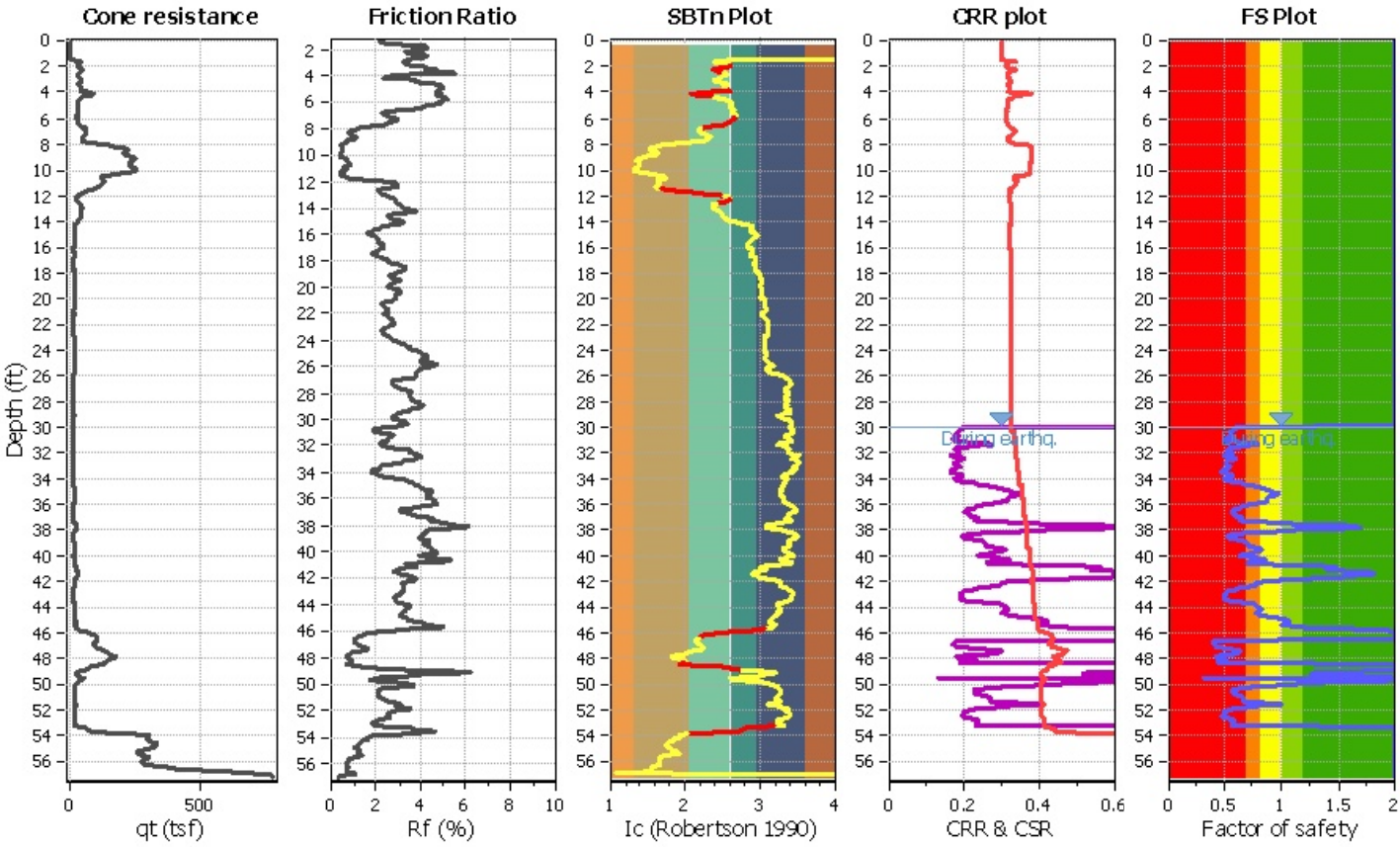
LIQUEFACTION ANALYSIS REPORT

Project title : Avenues Silicon Valley **Location : San Jose, California**

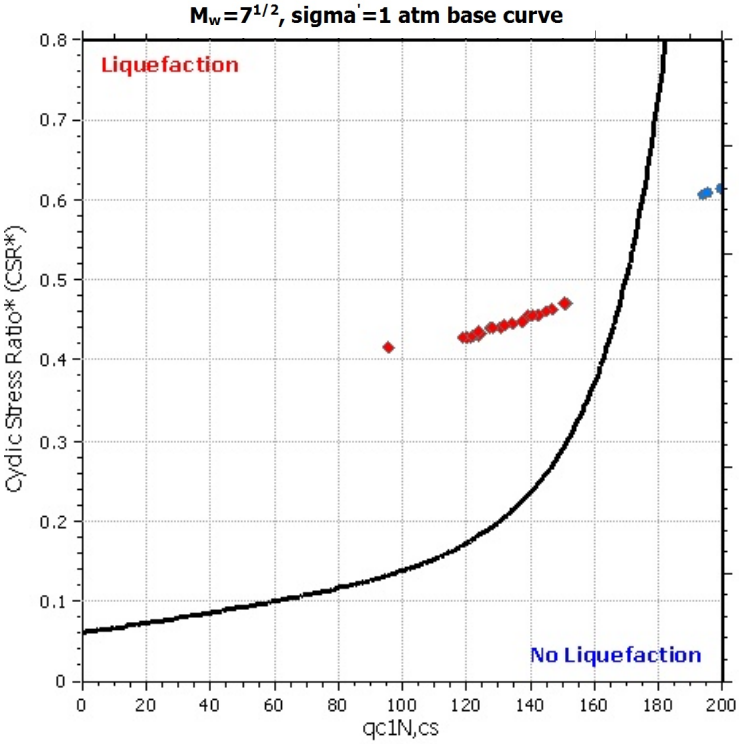
CPT file : 1-CPT5

Input parameters and analysis data

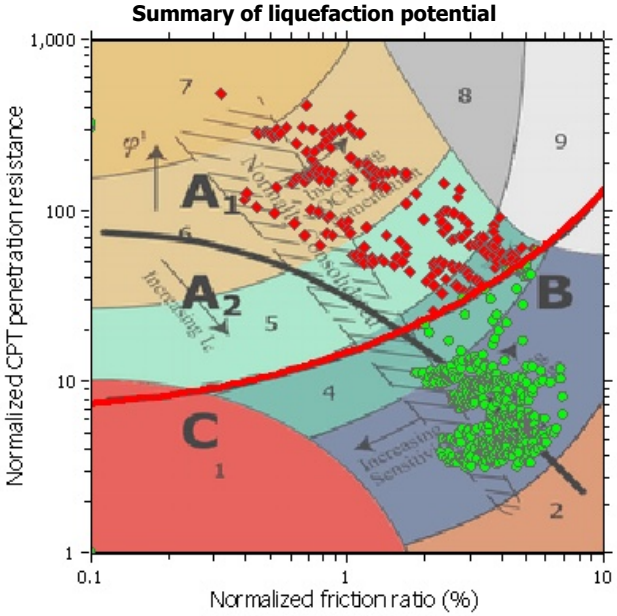
Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

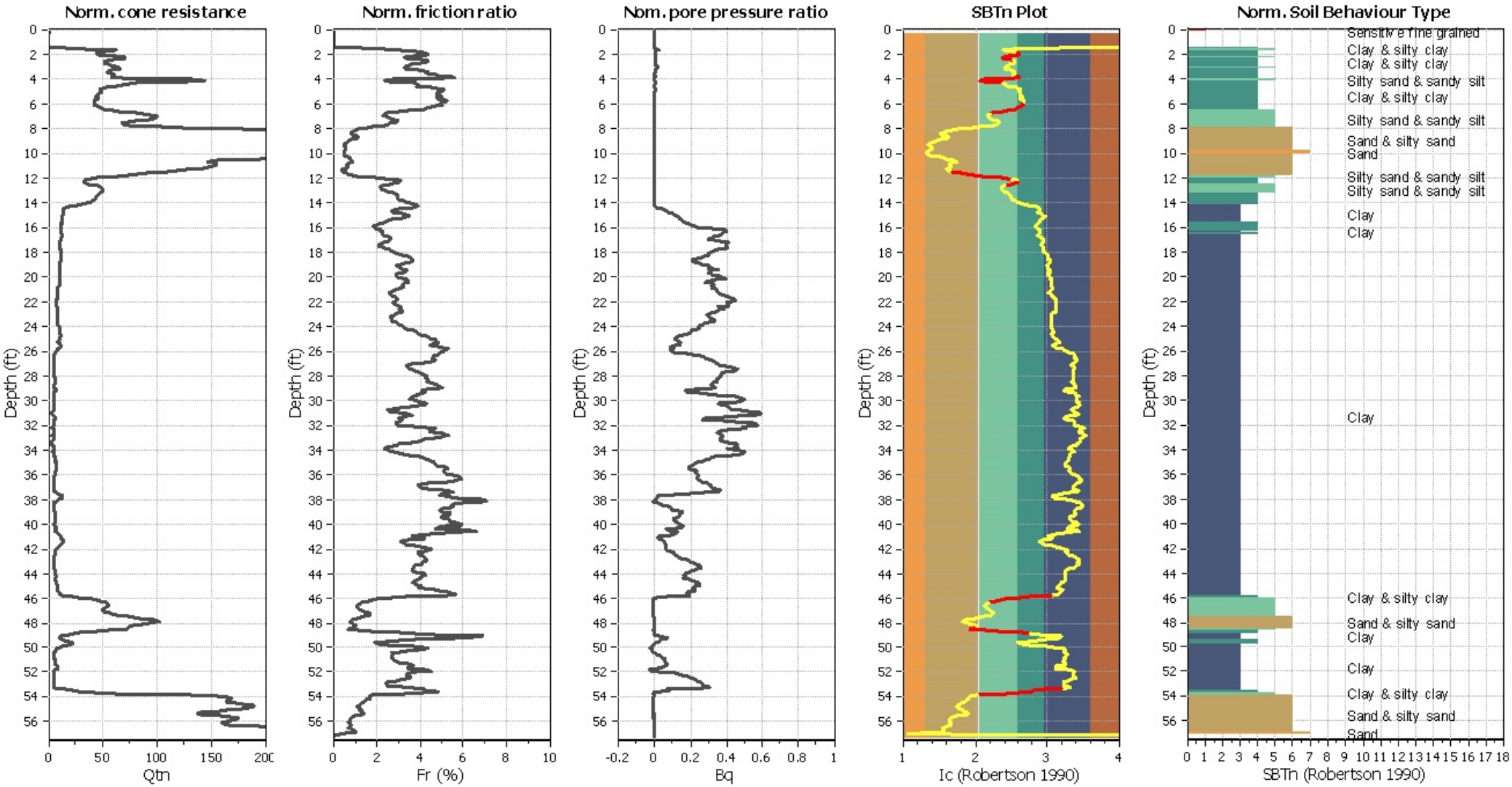


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



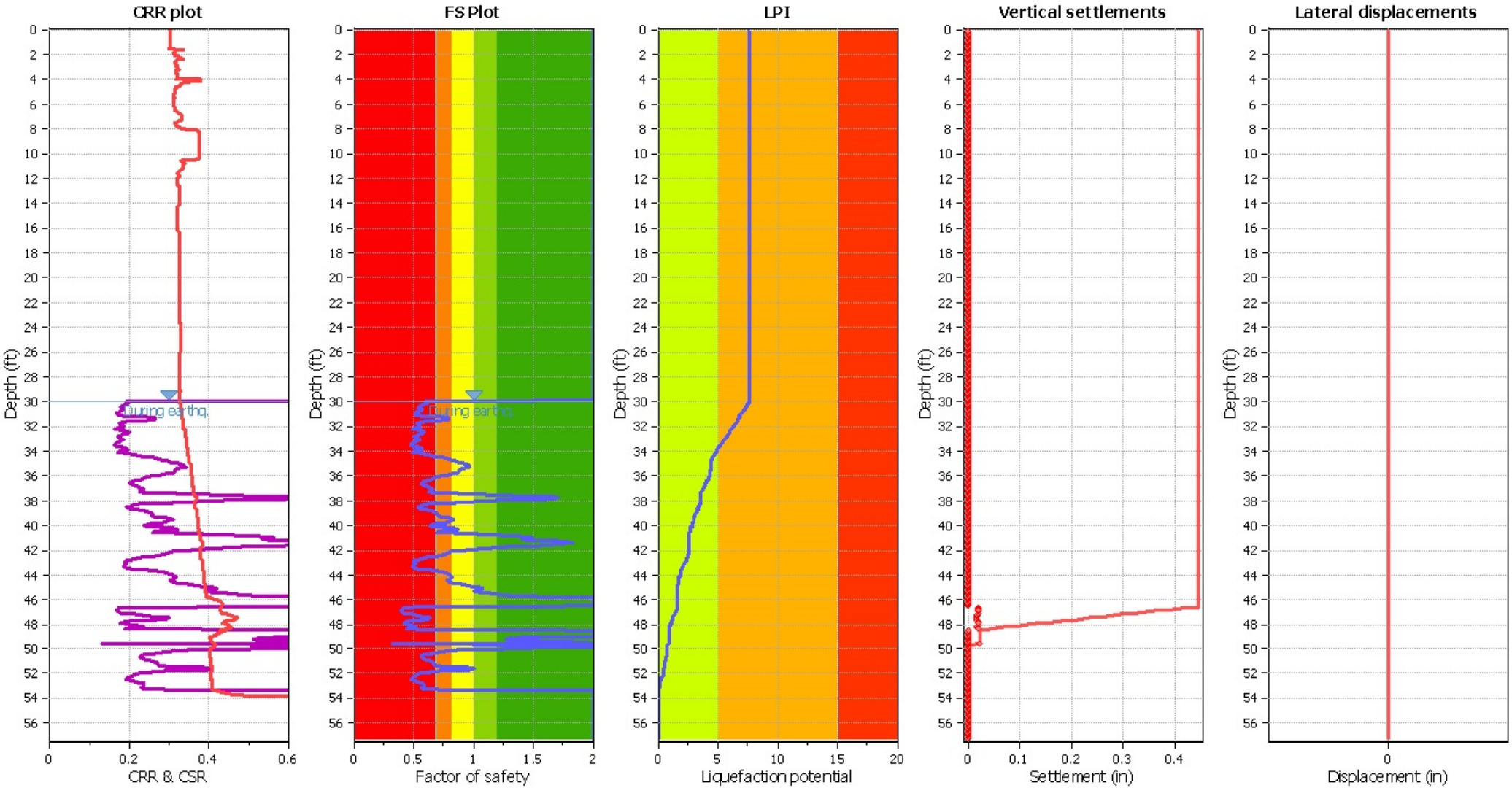
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
6399 San Ignacio Avenue
San Jose, CA
http://www.engeo.com

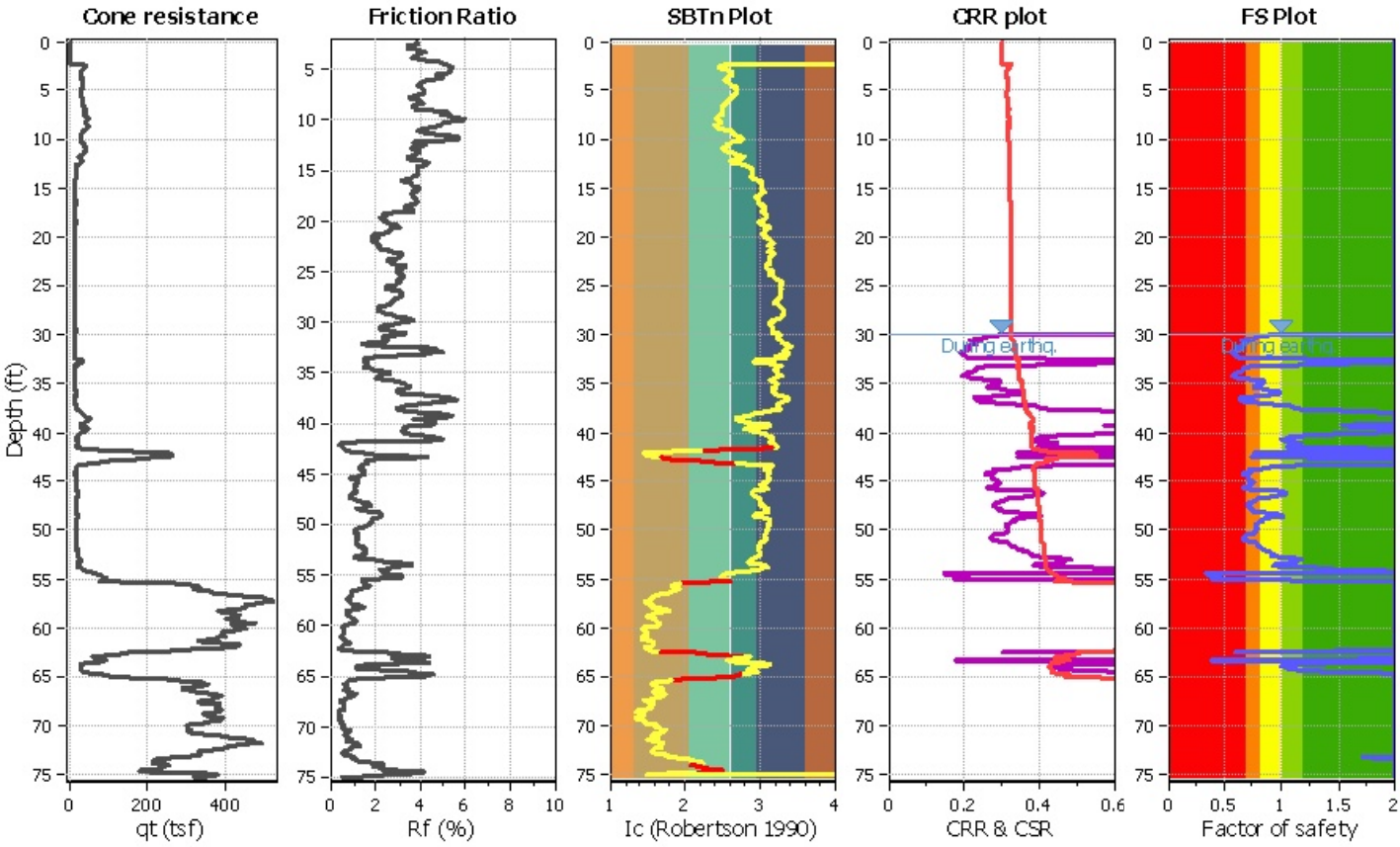
LIQUEFACTION ANALYSIS REPORT

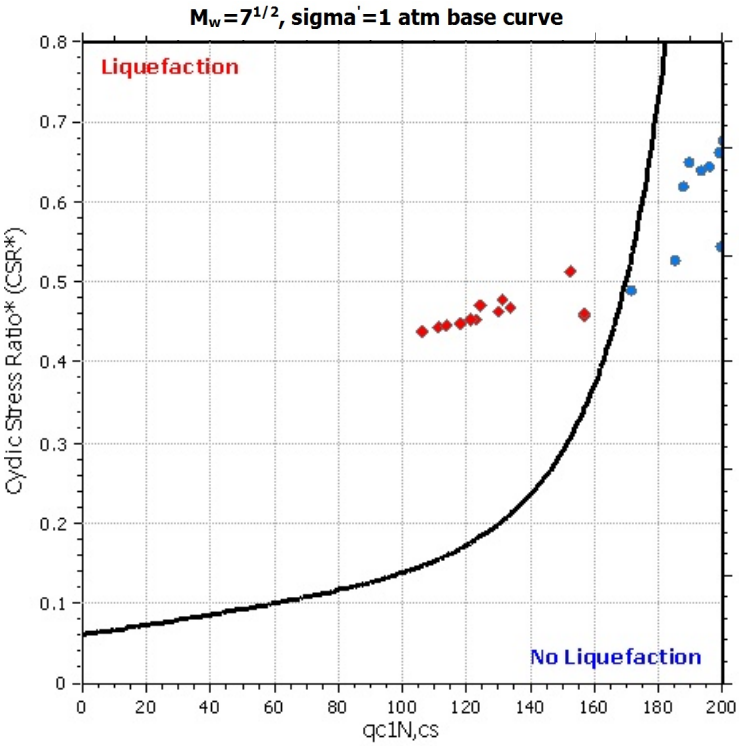
Project title : Avenues Silicon Valley **Location : San Jose, California**

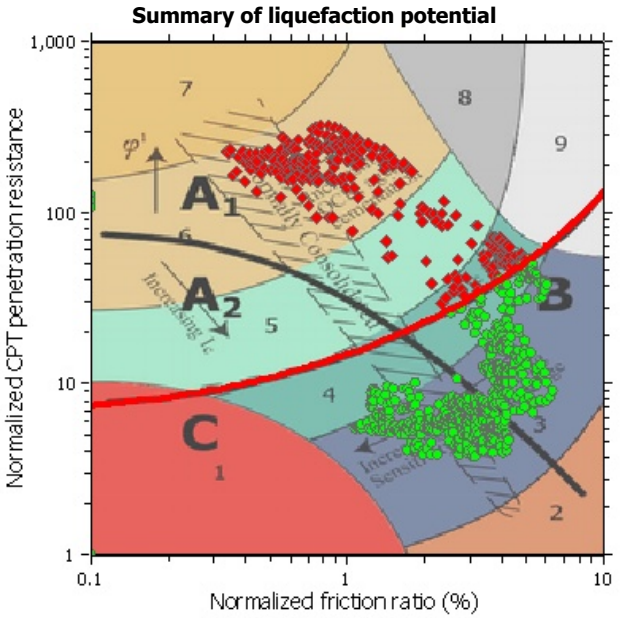
CPT file : 1-CPT6

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

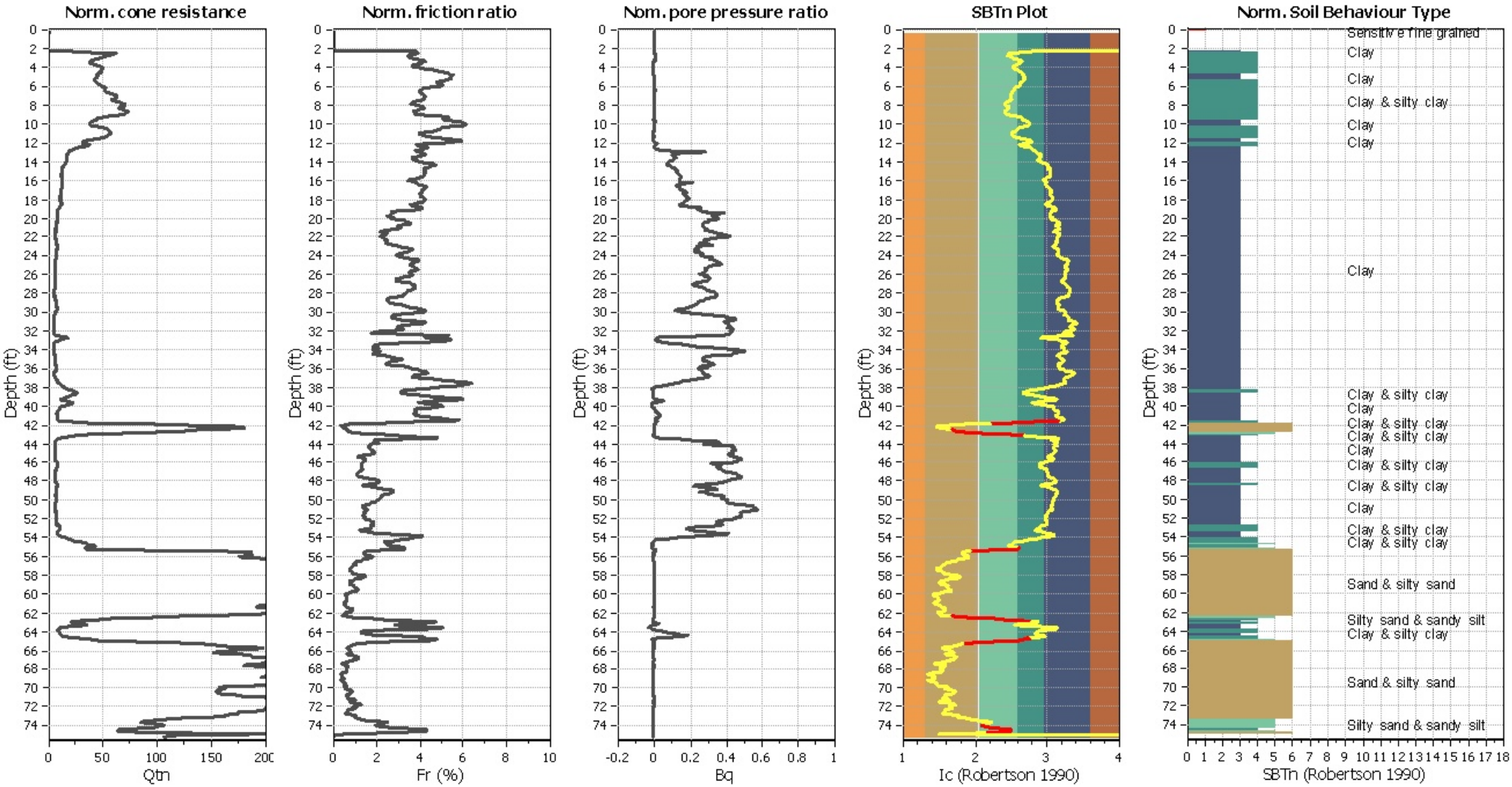






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



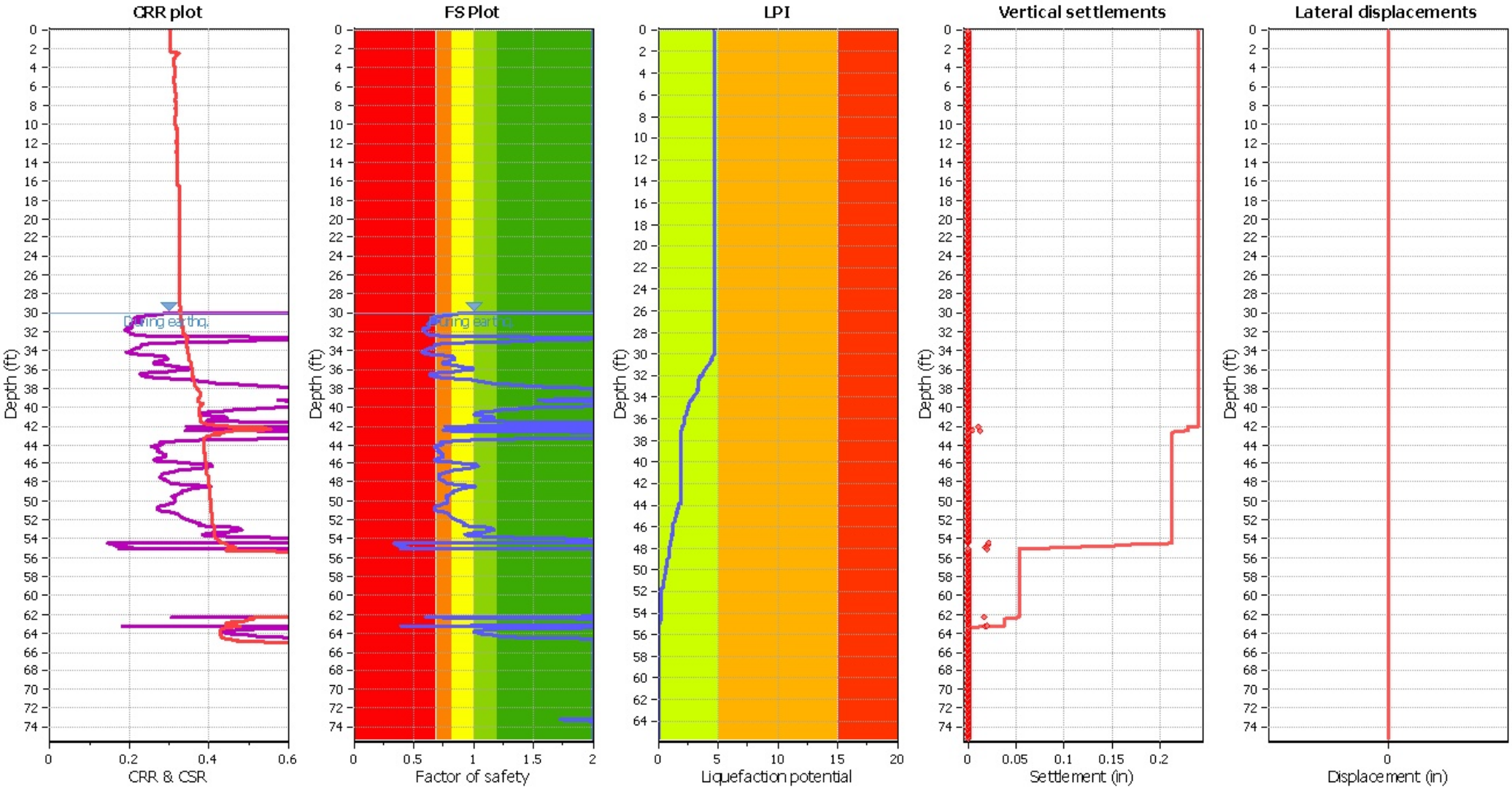
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX E

CORROSIVITY TEST RESULTS BY CERCO ANALYTICAL

California State Certified Laboratory No. 2153

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.: 15929.000.000
Client's Project Name: Avenues Silicon Valley
Date Sampled: 04/10 & 12/19
Date Received: 18-Apr-19
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 29-Apr-2019

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)*
1904152-001	1-B1 @ 4.5-5'	280	8.02	-	1,800	-	N.D.	28
1904152-002	1-B2 @ 8.5-9'	290	7.63	-	2,100	-	N.D.	N.D.
1904152-003	1-B4 @ 3-3.5'	270	8.23	-	1,200	-	N.D.	31

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	24-Apr-2019	24-Apr-2019	-	29-Apr-2019	-	24-Apr-2019	24-Apr-2019

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

1904152

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 15929.000.000

PROJECT NAME: Avenues Silicon Valley

SAMPLED BY: (PRINT) Jonas Bauer/Yanet Zepeda

PROJECT MANAGER: Yanet Zepeda 408-215-7009

ROUTING: E-MAIL vzepeda@engeo.com

Hard Copy NA

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride												REMARKS REQUIRED DETECTION LIMITS
001	1-B1 @ 4.5-5'	4/10/2019	—	Soil	1	liner	N/A	X	X	X	X	X												ASTM Test Methods
002	1-B2 @ 8.5-9	4/12/2019	—	Soil	1	liner	N/A	X	X	X	X	X												ASTM Test Methods
003	1-B4 @ 3-3.5	4/10/2019	—	Soil	1	liner	N/A	X	X	X	X	X												ASTM Test Methods

TUBES

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
	4/16/19 5:00pm			4/18 1:50p	
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
	4/17/19 2:30PM				
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME	REMARKS	
	4/18/19 1400			STANDARD TAT	

ENGEO INCORPORATED

6399 SAN IGNACIO AVENUE, SUITE 150
SAN JOSE, CALIFORNIA 95119
(408) 574-4900 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

APPENDIX F

SUPPLEMENTAL RECOMMENDATIONS

SUPPLEMENTAL RECOMMENDATIONS

Prepared by
ENGEO Incorporated

ENGEO
Expect Excellence

TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

BACKFILL	Soil, rock or soil-rock material used to fill excavations and trenches.
DRAWINGS	Documents approved for construction which describe the work.
THE GEOTECHNICAL ENGINEER	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
ENGINEERED FILL	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
FILL	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
IMPORTED MATERIAL	Soil and/or rock material which is brought to the site from offsite areas.
ONSITE MATERIAL	Soil and/or rock material which is obtained from the site.
OPTIMUM MOISTURE	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
RELATIVE COMPACTION	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
SELECT MATERIAL	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.0 GENERAL

1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

2.0 MATERIALS

2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 2.2-1: Engineered Fill and Backfill Requirements

US STANDARD SIEVE	PERCENTAGE PASSING
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 2.2-2: Imported Fill Material Requirements

	SIEVE SIZE	PERCENT PASSING
GRADATION (ASTM D-421)	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing

water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

2.4 PIPE

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 2.4-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZES (INCHES)	PIPE STIFFNESS (PSI)
PIPE STIFFNESS ABOVE 200 PSI (BELOW 50 FEET OF FINISHED GRADE)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
PIPE STIFFNESS BETWEEN 100 PSI AND 150 PSI (BETWEEN 15 AND 50 FEET OF FINISHED GRADE)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
PIPE STIFFNESS BETWEEN 45 PSI AND 50 PSI* (BETWEEN 0 TO 15 FEET OF FINISHED GRADE)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

***Pipe with a stiffness less than 45 psi should not be used.**

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

2.5 OUTLETS AND RISERS

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

2.6 PERMEABLE MATERIAL

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 2.6-1: Class 2 Permeable Material Grading Requirements

SIEVE SIZES	PERCENTAGE PASSING
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

2.7 FILTER FABRIC

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) ..... 180 lbs
Mass per Unit Area (ASTM D-4751) ..... 6 oz/yd^2
Apparent Opening Size (ASTM D-4751) .....70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)..... 80 gal/min/ft^2
Puncture Strength (ASTM D-4833) ..... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer’s recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

2.8 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the

core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the

geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1: Geotextile Soil Reinforcements

PROPERTY	TEST
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12-inch length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

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