OYSTER COVE
PETALUMA, CALIFORNIA

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Trece Herder
Brookfield Bay Area Holdings, LLC
12657 Alcosta Blvd, Suite 250
San Ramon, CA 94583

PREPARED BY
ENGEO Incorporated

January 29, 2024
Revised February 14, 2024

PROJECT NO.
15571.003.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES
COASTAL/MARINE GEOTECHNICS

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Ms. Trece Herder
Brookfield Bay Area Holdings, LLC
12657 Alcosta Blvd, Suite 250
San Ramon, CA 94583

Subject: Oyster Cove
100 East D Street
Petaluma, California

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

Dear Ms. Herder:

As requested, we completed a design-level geotechnical exploration for the proposed Oyster Cove project in Petaluma, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and design-level recommendations for the proposed project.

In our opinion, the site is suitable from a geotechnical standpoint for the proposed development, provided the recommendations and guidelines in this report are implemented during project planning, design, and construction. The main geotechnical considerations at the site include settlement of compressible soil layers, strong ground motions, liquefaction-induced settlement, slope stability, and the presence of non-engineered fill. Our recommendations to address these concerns are presented in the accompanying report.

We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

Kurt Katzenberger

Todd Bradford, GE

Jeff Fippin, GE

kk/tb/jaf/ar

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this design-level geotechnical exploration report is to provide recommendations for design and construction of the proposed Oyster Cove project in Petaluma, California. Brookfield Bay Area Holdings, LLC authorized us to conduct the following scope of services in accordance with our agreement dated December 19, 2023.

- Review of relevant background information, including available literature, geologic maps, and reports pertinent to the project site (site)
- Exploration of subsurface conditions
- Laboratory testing of select samples collected during the field exploration
- Evaluation of geotechnical conditions and perform analyses of collected data
- Preparation of this geotechnical report

Our conclusions and recommendations in this report are based, in part, on our review of the following plans and reports.

- Dailey, John H., Consulting Geotechnical Engineer. 1994. Geotechnical Investigation: Proposed Electric Equipment Company Building, 100 East D Street, Petaluma, California. June 20, 1994.
- Berlogar, Stevens, & Associates. 2018. Due Diligence Geotechnical Investigation, East D Street, Petaluma, California. December 19, 2018. Job No. 3995.100.
- KTGY. 2024. Schematic Design, Oyster Cove, Petaluma, California. January 8, 2024.
- ENGEO. 2021. Limited Geotechnical Exploration, Oyster Cove, Petaluma, California. July 19, 2021. Project No. 15571.001.000.
- Carlson, Barbee & Gibson, Inc. 2024. Preliminary Demolition, Surcharge & Site Leveling Plans, Oyster Cove, City of Petaluma, Sonoma County, California. January 11, 2024. Job No.: 2969-000.

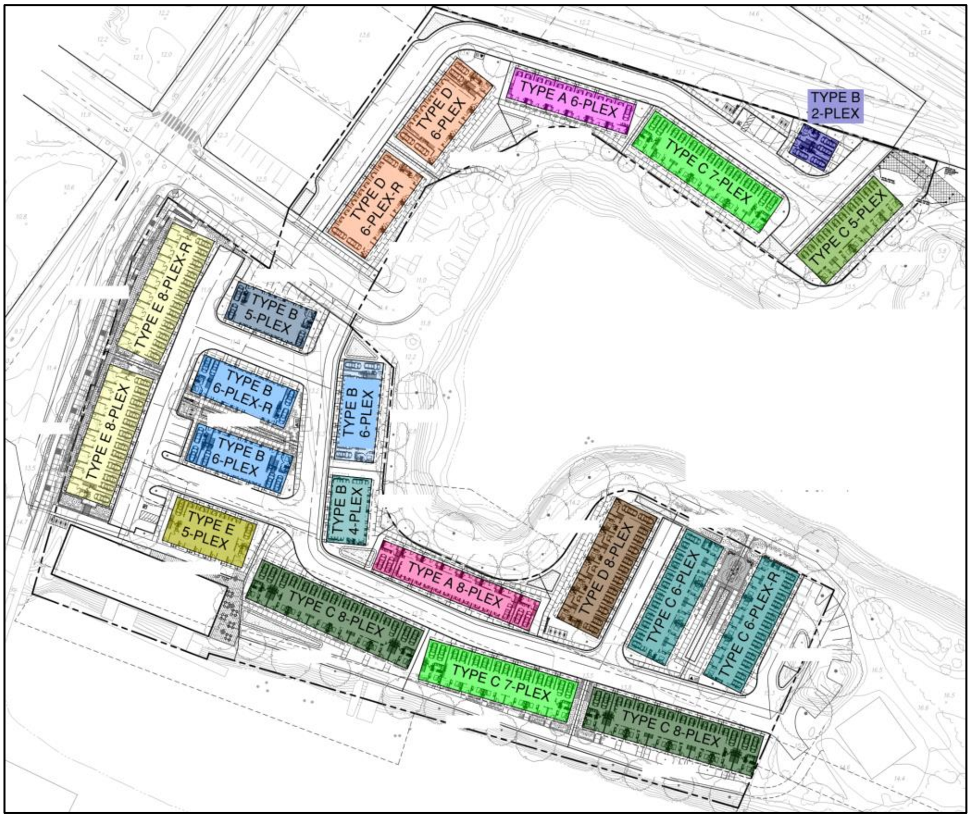
We prepared this report for the exclusive use of Brookfield Calwest Builders, LLC and its design team consultants. We should be engaged to review any changes made in the character, design, or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. 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 PROPOSED DEVELOPMENT

We understand the proposed Oyster Cove development will consist of 131 townhome units within 21 residential structures. We anticipate that the development will also incorporate paved drive aisles and parking, underground utilities, and secondary slabs-on-grade such as sidewalks, ancillary structures, landscaping, and stormwater basins. The proposed development plan is shown in Exhibit 1.2-1. Structural loads are yet to be provided; however, we assume the structural loads will be consistent with similar construction.

According to the Preliminary Demolition, Surcharge & Site Leveling Plans prepared by Carlson, Barbee & Gibson, Inc. (CBG, 2024), earthwork will comprise minor excavations and fill of between 1 and 2 feet to achieve final building pad grades at approximately Elevation 13 to 16 feet (NAVD88).

EXHIBIT 1.2-1: Proposed Development Plan (ktgy, 2024)



1.3 SITE LOCATION AND DESCRIPTION

The property is located at 100 East 3rd Street and encompasses approximately 6 acres, identified as Assessor's Parcel Numbers (APNs) 007-700-003, 007-700-005, and 007-700-006. The project site is located along the eastern edge of D Street, adjacent to the D Street Drawbridge, and is bisected by Copeland Street and the manmade inlet McNear Channel. Lakeville Street and Hopper Street border the northern edge of the site, D Street borders the western edge, the McNear Channel and Steamer Landing Park border the eastern edge, and the Petaluma River borders the southern edge. We include a site vicinity map as Figure 1.

The northern portion of the site (APN 007-700-005) currently has no existing structures and consists of landscaped areas and a small, paved parking lot and previously contained railroad tracks through the parcel. The southern portion of the site (APNs 007-700-003 and 007-700-005) is currently occupied by several single-story commercial structures. Both the Petaluma River and McNear Channel lie downslope from the project site; the slope appears to reach a maximum height of approximately 13 feet with a gradient of approximately 1½:1 (horizontal:vertical).

The site is generally level with site grades ranging from approximately Elevation 11 to 16 feet in the relatively level portion of the site and down to approximately Elevation 3 feet along the waterline between the site and both the Petaluma River and McNear Channel.

2.0 FINDINGS

2.1 SITE HISTORY

We reviewed historical aerial photographs and topographic maps available on www.historicaerials.com, the University of California, Santa Barbara (UCSB) aerial photograph library, and Google Earth. We understand that the site was developed for commercial and industrial use before 1914, and the manmade McNear Channel was excavated before 1900. The original structures in the center and northern portions of the site were demolished between 1952 and 1968, and railway lines crossing through the northern portion of the site were abandoned between 1982 and 1993. Two structures were constructed in the southeastern portion of the site between 2002 and 2004, and the site has remained relatively unchanged since 2004. We observed large material stockpiles throughout the southern portion of the site in various photographs.

2.2 REGIONAL GEOLOGY

The site is located within the Coast Ranges geomorphic province of California. The Coast Ranges province is typified by a system of northwest-trending, fault-bounded mountain ranges, and intervening alluvial valleys.

Bedrock in the Coast Ranges consists of igneous, metamorphic, and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-known faults.

2.3 SITE GEOLOGY

According to published geologic mapping of the site by Wagner et al. (2002), the northern portion of the site is underlain by late Holocene terrace deposits (Qhty) that generally consist of silt and moderately to well-sorted sand and gravel, while the southern portion of the site is underlain by Holocene estuarine deposits (Qhbm). The surficial Holocene deposits are underlain by older Holocene alluvial deposits (Qha). We include a regional geologic map in Figure 3.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. An active fault is defined by the California Geologic Survey as one that has had surface displacement within Holocene time (about the last 11,700 years) (CGS, 2018).

We show the liquefaction susceptibility map in Figure 5. The project site has not been evaluated for seismic hazards in the Seismic Hazard Zone Map prepared by CGS.

2.4 REGIONAL FAULTING

The San Francisco Bay Area contains numerous active faults. Figure 4 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region.

To identify nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Earthquake Hazard Toolbox and the 2018 National Seismic Hazard Model (NSHM) to perform a disaggregation of the seismic hazard at the peak ground acceleration (PGA) and at spectral periods up to 5 seconds for a return period of 2,475 years.

The nearest active fault with a significant contribution (greater than 1 percent) to the overall seismic hazard at the site is the Rodgers Creek trace of the Healdsburg fault, approximately 5.5 miles away. Other nearby faults capable of producing significant ground shaking at the site are shown in Table 2.4-1.

TABLE 2.4-1: Active Faults Capable of Producing Significant Ground Shaking at the Site Latitude: 38.2358 Longitude: -122.6346

SOURCE	R_{RUP}		MOMENT MAGNITUDE M_W
	(km)	(miles)	
Rodgers Creek - Healdsburg [3]	8.9	5.5	7.2
San Andreas (North Coast) [5]	24.1	15.0	7.9
Bennett Valley [1]	10.7	6.6	6.5

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

These results represent known fault sources contributing at least 1 percent to the seismic hazard at the site considering spectral periods ranging from the peak ground acceleration (PGA) to 5 seconds for the given return period. The rupture distances (R_{RUP}) and mean moment magnitudes (M_W) listed are based on values assigned according to the 2018 NSHM, and the numbers in parentheses after the fault names correspond to fault subsections assigned by the NSHM. Note that the above fault table is not an exhaustive list and other faults in the region may generate seismic shaking at the project site.

The Uniform California Earthquake Rupture Forecast (Field et al. 2015) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 generated an overall probability of 72 percent for the San Francisco Region.

2.5 PREVIOUS FIELD EXPLORATION

John H. Dailey Consulting Geotechnical Engineer prepared a geotechnical exploration report in 1994 that included two borings at the site. Berlogar, Stevens, & Associates (BSA) advanced eight cone penetration tests (CPTs) throughout the site for a geotechnical exploration in 2018. We conducted a geotechnical exploration in 2021 consisting of two borings and one direct-push continuous sampler. The approximate locations of the borings and CPTs from previous investigations are presented in Figure 2.

The previous borings ranged in depth between 11½ and 48 feet and the CPTs ranged in depth between 31 and 50 feet. The subsurface conditions noted in the previous explorations are generally consistent with our current findings presented in Section 2.8. The exploration logs and associated laboratory testing results from all previous explorations are included in Appendix D.

2.6 FIELD EXPLORATION

We performed a field exploration that included advancing four CPTs (including two seismic CPTs) and drilling four borings using a combination of solid-flight auger and mud-rotary methods. We performed our field exploration on the site on January 4 and January 5, 2024.

The approximate locations of our explorations are shown in Figure 2. We selected the exploration locations to supplement previous explorations and fill gaps in the data. The locations of our explorations are approximately located using consumer-grade global positioning system (GPS) and their proximity to existing site features; therefore, the locations shown should be considered accurate only to the degree implied by the method used. We permitted our explorations with Sonoma County.

2.6.1 Borings

A representative of our firm observed the drilling and logged the subsurface conditions at each location. We retained the services of a drilling subcontractor who provided a crew operating a truck-mounted drill rig to advance the borings using 4-inch-diameter solid-flight auger and mud-rotary drilling methods. We advanced the borings to depths ranging from 23 to 29½ feet below ground surface (bgs).

We obtained soil samples at various depth intervals using either standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D.) split-spoon sampler, California Modified samplers with 2½-inch inside diameter (I.D.) fitted with sampling liners, or Shelby tubes with a 3-inch I.D. We advanced the driven samplers with an automatic trip, 140-pound hammer with a 30-inch free fall and recorded the penetration of the sampler in the field as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring log shows the number of blows required for the last 1 foot of penetration. We did not adjust the blow counts shown on the boring logs using any energy correction or sample size factors. We containerized soil cuttings and excess fluids in 55-gallon steel drums.

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

2.6.2 Cone Penetration Tests

We retained the services of a subcontractor operating a 30-ton CPT rig to perform testing to a maximum depth of up to 55½ feet bgs in general accordance with ASTM D5778.

Measurements collected during the CPTs 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 performed shear-wave velocity (V_S) measurements in 2-SCPT3 and 2-SCPT4 using the downhole seismic method specified in ASTM D7400. We present the CPT report with logs in Appendix B.

2.7 LABORATORY TESTING

We performed laboratory tests on select soil samples to evaluate their engineering properties, which include the laboratory test and standard procedures shown in the following Table 2.7-1, including reference to the report appendix where results are provided.

TABLE 2.7-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Natural Unit Weight	ASTM D7263	Appendix A & C
Natural Moisture Content	ASTM D2216	Appendix A & C
Plasticity Index (PI) (Wet Method)	ASTM D4318	Appendix A & C
Grain Size Distribution	ASTM D6913	Appendix A & C
	ASTM D422	Appendix A & C
Consolidation – Constant Rate of Strain	ASTM D4186	Appendix C
Triaxial Compression – Unconsolidated, Undrained	ASTM D2850	Appendix A & C
Unconfined Compression	ASTM D2166	Appendix A & C

2.8 SURFACE AND SUBSURFACE CONDITIONS

The ground surface of the site is relatively flat and ranges from approximately Elevation 11 to 16 feet. The stratigraphy can generally be defined by two locations as being the northern and southern portions of the site. In the northern portion, we encountered fill over alluvial deposits consisting of lean clay and silt interbedded with sand layers and occasional gravel. In the southern portion of the site, we encountered a soft fat clay layer between the fill and the alluvial deposits previously described. The following sections further describe soil layers based on the historical borings and our supplemental explorations.

2.8.1 Artificial Fill (Qaf), Northern and Southern Portions of the Site

Our explorations generally encountered artificial fill in thicknesses up to 4 feet in the northern portion of the site and up to 6½ feet in the southern portion of the site. The fill we encountered in the northern area of the site generally consisted of dark grayish brown to reddish brown, medium stiff, lean clay with sand. The southern area of the site had a surficial layer of white to pale olive, silty sand with gravel to a depth up to 3½ feet bgs with a substantial amount of crushed and fragmented shells. Beneath the surficial, pale olive silty sand in the southern area of the site we encountered artificial fill generally consisting of dark grayish brown to olive gray, loose, clayey sand. Thicker fill may be present in areas where former excavations were located and then backfilled.

2.8.2 Holocene Terrace Deposits (Qhty), Northern and Southern Portions of the Site

Holocene terrace alluvial deposits encountered at the site generally consist of yellowish brown to yellowish red, medium stiff to very stiff lean clay and low-plasticity silt and dark yellowish brown medium dense to dense poorly graded sand with clay and clayey sand.

2.8.3 Holocene Estuary Deposits (Qhbm), Southern Portion of the Site

Holocene estuary deposits encountered at the site generally consists of dark greenish gray to olive gray, soft to medium stiff fat clay that is highly compressible when subjected to new loads.

2.9 GROUNDWATER CONDITIONS

We observed static groundwater in our borings and estimated the depth to groundwater in CPTs based on pore pressure dissipation tests. We summarize our observations in the table below.

TABLE 2.9-1: Groundwater Observations During Exploration

EXPLORATION ID	INTERPRETED GROUNDWATER DEPTH (feet, bgs)	INTERPRETED GROUNDWATER ELEVATION (feet, NAVD88)
2-B1	N/A[2]	N/A[2]
2-B2	12.0[1]	0.5[1]
2-B3	6.0	5.5
2-B4	6.0	6.5
2-CPT1	N/A[3]	N/A[3]
2-CPT2	5.6	6.4
2-SCPT3	N/A[3]	N/A[3]
2-SCPT4	N/A[3]	N/A[3]

[1] Potential outlier compared to other GWT data.
[2] Not observed during drilling due to drilling method.
[3] Equilibrium pore pressure not achieved.

We also reviewed the historical reports for groundwater information. These reports generally encountered groundwater between 5 and 12 feet bgs at the time of exploration. As a conservative estimate, we used a design groundwater table of 5 feet bgs in our analyses based on our explorations, historical groundwater data, and previous explorations. Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, and changes in irrigation and drainage patterns.

3.0 DISCUSSION AND CONCLUSIONS

Based on the exploration and laboratory test results, the project site is feasible for the proposed development, provided the recommendations contained in this report are properly incorporated into the design plans and specifications. The primary geotechnical concerns for the proposed site redevelopment include the following.

- Settlement of compressible layers due to new fill placement and new building loads
- Strong ground motions
- Liquefaction and liquefaction-induced settlement
- Presence of existing non-engineered fill
- Expansive soil
- Shallow groundwater
- Slope stability of site adjacent to Petaluma River

These and other pertinent design issues are discussed in the following sections.

3.1 STATIC SETTLEMENT AND CONSOLIDATION OF ESTUARY DEPOSITS

The ground surface can experience settlement through short-term elastic compression and long-term consolidation when a new loading scenario is introduced by structures, earthwork, and/or equipment if the site is underlain by compressible soil. The amount of settlement is dependent on the magnitude and duration of the applied load, the shape and size of the applied load area, and the depth, thickness, and stress history of the compressible soil. In the northern portion of the site, our field explorations generally encountered material susceptible to short-term elastic compression, including clay and loose sand. Such settlements are typically exhibited during construction but can continue past construction if not mitigated.

In the southeastern portion of the site, our field explorations encountered compressible Holocene alluvium (Qhaf) soil susceptible to long-term consolidation settlement; the explorations encountered this soil ranging from 7 feet to 15 feet in thickness. The approximate areas where the potentially compressible soil was encountered in explorations is shown in Figure 2.

We analyzed consolidation settlement that could happen due to new loads from fill and structure construction at the site. We considered an average of 1 foot of fill placement to achieve pad grades and building loads of averaging 450 pounds per square foot (psf) distributed equally across a structural mat foundation with dimensions up to the size of the planned lots. We used geotechnical data from the explorations and laboratory testing to develop a generalized subsurface profile and representative soil parameters for our analysis.

Based on our analysis, we estimate up to approximately 18 inches of consolidation-related settlement will occur in the southern portion of the site underlain by the compressible material from the new building loads and fill if no mitigation is performed. We anticipate that this level of estimated settlement exceeds the performance criteria of the structures; we provide surcharge recommendations in Sections 4.3 to mitigate excess building settlement.

Secondary compression settlement will occur slowly as aerial settlement over the next 30 to 50 years. It is independent of, and in addition to, the consolidation settlement that will occur due to the structural load and additional fill load. Surcharging will reduce but not eliminate this secondary settlement, so it should be addressed in design of the gravity flow utilities and surface grades.

We estimate up to 1 inch of immediate settlement will occur in the northern portion of the site, which is not underlain by the compressible material; we anticipate the immediate settlement will occur during construction as grading occurs and building loads are applied.

TABLE 3.1-1: Estimated Consolidation Settlements (no mitigation)

LOCATION	STATIC SETTLEMENT (inches)	SECONDARY COMPRESSION (inches)
Northern portion of site	< 1	n/a
Southern portion of site	< 18(1)	2 – 3

(1) Without mitigation; consolidation settlement mitigation discussed in Sections 4.3.

Considering the distance between the locations of the anticipated maximum and minimum secondary settlement, we recommend utilities are designed to accommodate a long-term differential settlement of 3 inches over 60 feet.

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

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site; therefore, is unlikely.

3.2.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some non-structural damage, and (3) resist major earthquakes without collapse, but with some structural, as well as non-structural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.2.3 Liquefaction

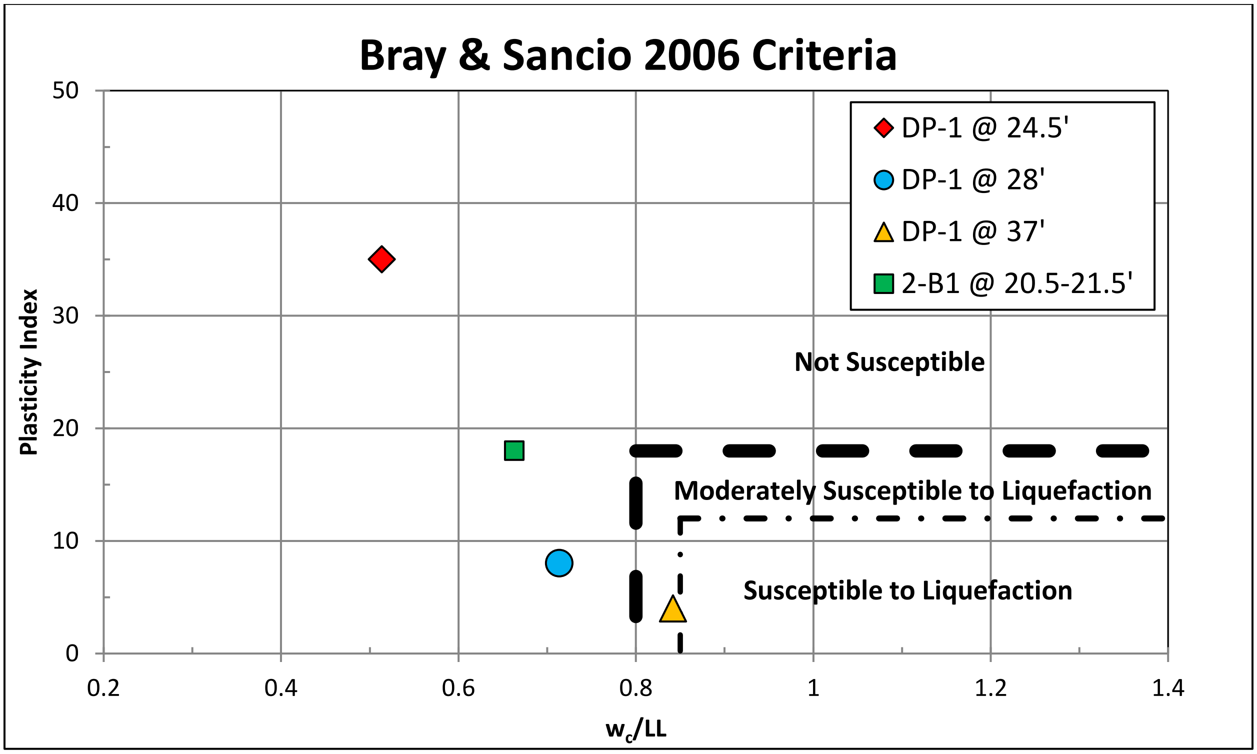
Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the saturated sandy soil is subjected to cyclic shear stresses that can cause excess pore-water pressures to develop due to volumetric repositioning of soil particles. As excess pore-water pressures approach the effective confining stress from the overlying soil, the sand will experience a reduction in effective shear strength and may undergo deformation. If the pore-water pressures exceed the effective confining stress, the sand particles are free to move within the soil-water matrix without significant resistance, at which point the soil is said to have liquefied. If the sand consolidates or vents to the surface (known as "sand boils") during and following liquefaction, ground settlement and surface deformation may occur. Furthermore, structures founded directly upon liquefied soil can result in partial or complete loss of bearing support causing significant structural damage or collapse. In addition to liquefaction of sandy materials, clayey soil can also experience "cyclic softening" or strength loss as a result of cyclic loading.

3.2.3.1 Liquefaction Susceptibility Screening of Soil Samples

We considered the criteria presented by Bray and Sancio (2006) to assess the potential for liquefaction triggering on the site soil. Bray and Sancio observed that soil with a plasticity index (PI) less than 12 and a water content (w_c) to liquid limit (LL) ratio of more than 0.85 are susceptible to liquefaction/cyclic softening. Soil with PI greater than 18 and/or w_c/LL less than 0.8 were deemed to be not susceptible to liquefaction because they are too plastic and/or their water contents are too low.

The plotted data from the current and limited geotechnical explorations is shown in Exhibit 3.2.3.1-1.

EXHIBIT 3.2.3.1-1: Assessment of the Liquefaction/Cyclic-Softening Potential of Fine-Grained Soil based on the Bray and Sancio (2006) Criteria



We considered the Bray and Sancio criteria at this site and plotted wc/LL versus PI for the laboratory data collected from the layers previously identified as potentially liquefiable. Laboratory data for samples collected at 1-DP1 (ENGEO, 2021) at a depth of 24½ feet and 28 feet plot as not susceptible to liquefaction based on these criteria. Liquefaction-induced vertical settlement from these layers amounted to approximately 1 inch. Laboratory data collected from deeper layers identified soil that is moderately susceptible to liquefaction. We noted several shallower granular layers, that when coupled with the previous analysis, we considered susceptible to liquefaction and subsequently were not tested. We combined the data from 1-DP1 in our current geotechnical exploration with previous CPTs (BSA, 2018) to evaluate liquefaction potential to 50 feet bgs.

3.2.3.2 Liquefaction-Induced Ejecta

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

Youd and Garris (1986) present a method for evaluating the potential of liquefaction-induced ejecta based on the thickness of the potentially liquefiable layer compared to the thickness of the overlying non-liquefiable soil. Based on the results of our analysis, the liquefiable portion of the site has a low potential for surface manifestation to occur during or following a strong seismic event.

3.2.3.3 Seismic-Induced Settlement

Incorporating subsurface interpretations from our borings, previous explorations by others, and the above-discussed screening methods, we estimate a total liquefaction induced vertical settlement of up to 1½ inches on the northern portion of the site and up to 1 inch on the southern portion. Based on our analysis, the liquefiable zones are generally interspersed between approximately 20 and 45 feet bgs. Therefore, as the liquefiable layers are not entirely continuous, average vertical deformations may be less than our estimates.

3.2.3.4 Lateral Spreading

Lateral spreading involves lateral ground movement caused by seismic shaking. This lateral ground movement is often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soil. The potential for lateral spreading is predominantly controlled by the presence of a free face such, as sloped condition at this site into the adjacent water, continuity of liquifiable layers, and the depth of the layers relative to the height of the free face. Given the discontinuity of the potentially liquefiable layers noted in our analysis and their depth, we find the potential for lateral spreading to be low.

3.2.4 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the San Francisco Bay Area region, but based on the site location, it is our opinion that the offset, if any, would be minor.

3.3 EXISTING FILL

We encountered existing artificial fill between 4 and 6½ feet thick in our explorations; the fill primarily consisted of stiff to very stiff lean clay with varying sand and gravel compositions and dense to very dense sand and poorly graded gravel with varying fines content. Non-engineered fill can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend mitigating potential deleterious settlements through a combination of removing and recompacting the upper 1½ feet of the existing fill across the entire site and preloading the southern portion of the site with surcharge. We present fill removal and surcharge recommendations in Sections 4.1 and 4.3, respectively.

3.4 EXPANSIVE SOIL

We encountered potentially expansive clay in the near-surface soil of our soil explorations. Expansive soil can change in volume with changes in moisture. It can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. For this project, we recommend reducing the potential for building damage due to volume changes associated with expansive soil by using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil for the residential structures and using spread footings deepened to below the zone of seasonal moisture fluctuation for ancillary structure foundations. We provide foundation recommendations in Section 5 of this report.

Successful performance of structures and improvements on expansive soil requires special attention during construction. 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.

We also provide specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. Expansive soil mitigation recommendations are presented in Sections 4 and 5 of this report.

3.5 SHALLOW GROUNDWATER

As previously discussed, we considered a design groundwater level of 5 feet bgs across the site; though during construction, groundwater may be deeper depending on the time of year and weather conditions. During underground construction, temporary dewatering procedures should be anticipated to be necessary to lower the groundwater so that excavation and working areas are kept reasonably dry and stable during construction. Dewatering should be performed in isolated areas and in limited amounts so that any drawdown of groundwater does not extend below nearby improvements so that off-site settlement is not induced. Perched groundwater conditions may exist due to plastic clay layers resulting in localized ponding of groundwater.

3.6 SLOPE STABILITY

We performed a stability analysis of the river frontage slope to confirm the slope is appropriately stable under construction, long-term, and seismic loading. The following sections describe this analysis.

3.6.1 Geometry and Soil Parameters

The Preliminary Demolition, Surcharge & Site Leveling Plans prepared by Carlson, Barbee, & Gibson Inc. (CBG), dated January 11, 2024, provides existing topographic information. Additionally, CBG provided us with a conceptual river frontage plan depicting proposed slopes along the Petaluma River to include retaining walls of up to 3½ feet and slopes of 2.5:1 (horizontal:vertical) maximum gradient. We used these conceptual plans as the basis of our slope stability analysis. We modeled the subsurface conditions from our recent explorations. We conducted slope stability analysis on one cross section. The location of this cross section is depicted in Figure 2.

We developed a shear strength profile of the soil based on CPT data and various laboratory test results obtained during this geotechnical exploration. We derived strength parameters assigned to each soil layer primarily from laboratory data provided in Appendix B. Based on our data review, we developed the idealized soil profile shown in Appendix E.

3.6.2 Method of Analysis

We used the program Slide2, 2D Limit Equilibrium Analysis for Slopes, version 9.027, and a search routine with circular surfaces to estimate the minimum factor of safety and critical slip surface location. We used Spencer's method for slope stability analysis; this analytical method is an iterative solution that satisfies both force and moment equilibrium and assumes all slice side forces have the same inclination.

In evaluating the stability of slopes under seismic conditions, we used a "pseudostatic" method of analysis. The pseudostatic method models the effects of transient or pulsating earthquake loading on a potential slide mass by using an equivalent sustained horizontal force that is the product of a seismic coefficient and the weight of the potential slide mass. We used a two-stage analysis where in the first stage the shear strengths along each surface are developed under static conditions. In the second stage, an additional horizontal force acting in the direction of potential failure is imposed on the sliding mass. This two-stage procedure is performed for each surface in the search and a surface with the lowest factor of safety is found. The additional horizontal force is equal to the soil mass multiplied by a horizontal seismic coefficient.

We selected the design seismic coefficient based on the procedure outlined in California Geological Survey Special Publication 117A (SP 117A). We used a value of 0.41g as MHAr based on two-thirds of the Maximum Credible Earthquake peak ground acceleration (PGA), which correlates to the Building Code Design Earthquake PGA. We used a moment magnitude of 7.25 earthquake based on the site's proximity to the Rodgers Creek - Healdsburg fault. Based on this, we used a seismic coefficient of 0.18g based on an upper-bound displacement value (u) of 15 cm (6 inches) outlined in SP 117A and site earthquake information.

3.6.3 Acceptable Factors of Safety and Results of Analysis

The Factor of Safety (FS) is defined as the sum of available shear strength resistance divided by mobilized shear strength. A FS value less than 1.0 indicates slope instability, and the greater the FS, the greater the anticipated stability of the slope. Our analyses for this evaluation are derived from information published in previous reports, recently performed explorations, laboratory testing, and details outlined in SP 117A. We consider a FS of 1.5 for the static condition and a FS of 1.0 for the seismic condition to be appropriate criteria for this analysis.

We performed slope stability analyses for both static and pseudostatic loading conditions. Additionally, we analyzed the static stability of the interim loading scenario of a 9-foot-tall temporary surcharge, described further in Section 4.3. We conducted a sensitivity analysis of our shear strength profile to consider a reasonable worst-case scenario in our selection of shear strength. As shown in Appendix E, the static and seismic factors of safety are 2.0 and greater than 1.0, respectively, for the long-term design scenario and greater than 1.4 for the temporary condition. As described in SP 117A, slopes that have a pseudostatic factor of safety greater than 1.0, using a seismic coefficient derived from this screening analysis procedure, can be considered stable. We opine that the slope is stable for both long-term and temporary loading conditions described above.

4.0 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. 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.

4.1 EXISTING FILL REMOVAL

At a minimum, we recommend that the upper 1½ feet of existing soil across the site be reworked and recompacted to achieve a final engineered fill thickness of approximately 3 feet at finished grade when accounting for the placement of additional civil fill. Based on the consistency of fill encountered during our explorations and by preloading the site via surcharge, we anticipate that the risk of deleterious post-construction settlement is low and that the deeper existing fill is generally suitable to leave in place. Any environmental restrictions in regard to existing material disturbance should be considered in preparing the final earthwork requirements.

4.2 ACCEPTABLE FILL

Based on the material encountered in our exploration, we anticipate that on-site soil is suitable as fill material, provided it is processed to remove concentrations of organic material (soil which contains more than 3 percent organic content by weight), debris, and particles greater than 6 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index less than 20 and at least 20 percent passing the No. 200 sieve. We should be informed when imported materials are planned for the site and be allowed to review, sample, and test (as needed) proposed imported soil fill materials at least 5 days prior to delivery to the site. Additionally, environmental sampling and testing of potential import soil sources should also be submitted to us for review.

4.3 SURCHARGE RECOMMENDATIONS

A surcharge program should be implemented to mitigate long-term consolidation settlement of the compressible Holocene estuarine deposits (Qhbm) in the southern portion of the site.

Based on the proposed grading and development, we recommend that surcharge fill grades extend at least 9 feet above planned final design grades over the proposed areas to be surcharged. Surcharge fill should remain in the areas to be mitigated until we assess that the settlement under the surcharge load has essentially been completed based on measurements of settlement. We anticipate that a 9-foot-tall surcharge can be removed after an approximately 12-month duration.

The actual settlement (total amount and rate of settlement) should be monitored with settlement plates after surcharge fill is placed and the time required for settlement will depend on the observed rates.

We recommend settlement-monitoring plates be installed prior to surcharge placement to monitor consolidation. The number and location of the settlement monitoring plates should be determined by us and coordinated with the contractor. To allow for redundancy, no fewer than two settlement plates should be installed in any surcharge phase. The settlement-monitoring plates should be surveyed to measure elevations at least weekly for the first 2 months and then monthly until we are able to assess that the desired degree of surcharge-driven pre-consolidation has been achieved; our design is based on assuming at least 80 percent of the consolidation under surcharge loading is achieved. All readings of settlement should be tied to benchmarks established well beyond the zone of surcharge influence.

With the above-described surcharge plan, we estimate post-construction primary consolidation settlement will be less then 1 inch and long-term secondary consolidation settlement, occurring over approximately 50 years, will be less than 3 inches.

4.4 FILL COMPACTION

4.4.1 Fill Placement in Structural Areas

Following removal of any loose native soil or artificial fill, the exposed non-yielding surface of areas to receive fill or to be left at grade, should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to all fill, including backfill, except for landscape areas.

TABLE 4.4.1-1: Compaction Control Requirements

FILL LOCATION	REQUIRED RELATIVE COMPACTION* (%)	MINIMUM MOISTURE CONTENT (percentage points above optimum)
General Fill, Utility Trench, and Pavement/Flatwork subgrade	87 – 92	4
Pavement Aggregate Base	95	0

* Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material.

4.4.2 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with Cal/OSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials. Utility trench backfill should conform to the recommendations in Section 4.4 for on-site utilities and City of Petaluma requirements for off-site utilities. Jetting of backfill is not an acceptable means of compaction.

4.5 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring, during or following periods of rain, within areas below the groundwater table, or beyond the extent of the dewatering program. 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;
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation. These options may also be used to provide a stable building pad.

4.6 SITE DRAINAGE

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

1. Roof downspouts should discharge into closed conduits and direct away from foundations to appropriate drainage devices.
2. Water should not be allowed to pond near foundations, pavements, or exterior flatwork.

4.7 STORMWATER BIORETENTION AREAS

We did not perform infiltration testing as part of this geotechnical exploration. Based on site soil and the relatively shallow groundwater table encountered, we anticipate low infiltration rates. For planning purposes, we recommend assuming little stormwater infiltration will occur through the existing site soil and subdrains should be included in the design.

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

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

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

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

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

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

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

5.0 FOUNDATION RECOMMENDATIONS

In order to reduce the effects of the potentially expansive soil, potential for liquefaction-induced settlement, and long-term secondary compression settlement on building foundations, the foundations should be sufficiently stiff to move as rigid units with minimum differential movements to withstand the estimated liquefaction-induced settlements and static settlements previously discussed. This can be accomplished with a relatively rigid mat foundation, such as post-tensioned structural mats. Assuming the proposed surcharge plan is implemented, we estimate a post-construction settlement of less than 1 inch with a differential settlement of approximately half of the total settlement over a lateral distance of 30 feet.

While the liquefaction settlement should be added to the static settlement for the evaluation of seismic performance, the designer may wish to consider a larger amount of allowable architectural distress of the building under the settlement from liquefaction than from static loading.

5.1 POST-TENSIONED MAT FOUNDATIONS

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

PT mats may be designed for an average allowable bearing pressure of up to 1,200 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. The allowable bearing pressures can be increased by one-third for wind or seismic loads.

Based on the proposed final pad elevations, the site will be receiving import soil. As such, we should be retained to sample the subgrade of the final pads to perform laboratory testing and analysis to develop PT mat design criteria.

PT mats should be constructed over a moisture reduction system as recommended below. In addition, moisture conditioning of the building foundation subgrade should be to a moisture content at least 4 percentage points above optimum immediately prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement. We also recommend that we be retained to observe the pre-pour moisture conditions to check that our report recommendations have been followed.

5.1.1 Floor Moisture Vapor Reduction

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

1. A vapor retarder membrane should be placed directly beneath the slab. The vapor retarder should be sealed at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."
2. Concrete should have a concrete water-cement ratio of no more than 0.50.
3. Inspection and testing should be performed during concrete placement to check that the proper concrete and water-cement ratio are used.
4. PT mats should be moist cured 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 layer or sand is specified, the edges of the mat should be thickened by at least the thickness of the granular layer to cutoff potential water intrusion between the membrane and the mat; if used a thickened edge should be at least 12 inches wide.

5.1.2 Pad Moisture Conditioning

Proper moisture conditioning of building pads immediately prior to foundation concrete placement is important to reduce potential post-construction swell of expansive soil. We recommend moisture conditioning building foundation subgrade to a moisture content at least 3 percentage points above optimum to a depth of at least 12 inches immediately prior to post-tensioned foundation construction. Moisture conditioning deeper than 12 inches may be necessary depending on the time of year, drought conditions, adjacent slopes, open utility trenches, etc. The actual depth should be determined in the field by the firm checking the pad moisture. During the drier parts of the year, it may require several days of soaking of the pads to achieve this moisture content. The subgrade should not be allowed to dry below this specified moisture content prior to concrete placement. We also recommend that we be retained to observe the pre-pour moisture conditions to check that our design recommendations have been followed.

5.2 SPREAD FOOTINGS

We anticipate that spread footings may be implemented for ancillary structures such as trash enclosures, retaining walls, and other minor structures.

5.2.1 Footing Dimensions and Allowable Bearing Capacity

Footings, when used for ancillary structures, should have the minimum footing dimensions as follows in Table 5.2.1-1 below.

TABLE 5.2.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (inches)	MINIMUM WIDTH (inches)
Continuous	24	12
Isolated	24	12

*below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade

Foundations meeting the dimensions above can be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (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.

5.2.2 Foundation Lateral Resistance

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

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

The above values are ultimate values. Appropriate factors of safety should be used based on analysis method and load type.

Passive lateral pressure should not be used for footings on or above slopes.

5.3 2022 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered in our explorations and proposed development, we characterize the site as Site Class E in accordance with the 2022 CBC. ASCE 7-16 requires a site-specific ground-motion hazard analysis for Site Class E sites with a mapped S_S value greater than or equal to 1.0 or S_1 value greater than or equal to 0.2; however, Section 11.4.8 of ASCE 7-16 and Supplement No. 3 provide an exception to this requirement. A site-specific ground-motion hazard analysis is not required where the equivalent lateral force procedure is used for design and the value of C_s is determined by equation 12.8-2 of ASCE 7-16 for all values of period, T. Refer to Supplement No. 3 of ASCE 7-16 for the requirements pertaining to the exception for non-building structures.

We provide the CBC seismic parameters based on the United States Geological Survey's (USGS) Seismic Design Maps in Table 5.3-1. When using this table, considerations should be given to exceptions in Section 11.4.8 of ASCE 7-16, as described above.

TABLE 5.3-1: 2022 CBC Seismic Design Parameters, Latitude: 38.2343 Longitude: -122.6327

PARAMETER	VALUE
Site Class	E
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	1.2
Site Coefficient, F_v	2.0*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.8
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.2*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.2
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.8*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.68
Long period transition-period, T_L (sec)	12

*The parameters above should only be used for calculation of T_s, determination of Seismic Design Category, and, when taking the exceptions under Items 1 and 2 of ASCE 7-16 Section 11.4.8. (Supplement Number 3 https://ascelibrary.org/doi/epdf/10.1061/9780784414248.sup3).

6.0 RETAINING WALLS

6.1 LATERAL EARTH PRESSURES

Retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design unrestrained retaining walls with level native soil backfill to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus one-third of any surcharge loads.

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

A drainage system, as recommended below, should be constructed behind the wall to reduce hydrostatic forces.

6.2 RETAINING WALL DRAINAGE

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

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

For both types of rock drains:

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

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

6.3 BACKFILL

Backfill behind the retaining walls should be placed and compacted in accordance with Section 4.4. Light compaction equipment should be used within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

7.0 PAVEMENT DESIGN

7.1 FLEXIBLE PAVEMENTS

Because surface soil varies across the site, it is our opinion that an R-value of 5 is applicable for design. Using estimated traffic indexes for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (6th Edition, including the asphalt factor of safety), presented in the table below.

TABLE 7.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (inches)	CLASS 2 AGGREGATE BASE (inches)
5.5	3	12
6	3.5	13
6.5	4	14
7	4	16

The civil engineer should assign the appropriate traffic indexes 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 accompanying reinforcement should be performed based on estimated traffic loads and frequencies.

The rigid pavement section should consist of Portland cement concrete paving (PCCP) over Class 2 Aggregate Base over prepared subgrade. The PCCP should achieve a minimum 28-day concrete compressive strength of 3,500 psi. Control joints, spaced in accordance with Caltrans guidelines, should also be considered. Based on Topic 620 of the Caltrans Highway Design Manual (6^{th} Edition) and assuming an R-value of 5, we recommend that rigid pavements with a TI less than 9 have a minimum section of 9 inches over 12 inches of Class 2 Aggregate Base.

7.3 PERMEABLE PAVEMENTS

We recommend that vehicular pavers be designed and constructed in accordance with guidelines provided by ASCE 68-18 and the Interlocking Concrete Pavement Institute (ICPI). Based on the guidelines provided by ASCE 68-18 and the ICPI, and considering a subgrade R-value of 5, 80-mm pavers (Aspect Ratio no greater than 3) over 2 inches of No. 8 bedding course, we recommend the following minimum base and subbase sections. Alternatively, the following sections of Class 2 Permeable Material may be placed directly on subgrade. We provided appropriate sections for each option in Tables 7.3-1 and 7.3-2, respectively.

TABLE 7.3-1: Recommended Permeable Pavement Sections: Open-Graded Rock

TRAFFIC INDEX	SECTION		
	THICKNESS FOR ASTM NO. 8 BEDDING COURSE (inches)	THICKNESS FOR ASTM NO. 57 BASE (inches)	THICKNESS FOR ASTM NO. 2 SUBBASE (inches)
5.5	2	4*	6*
6	2	4*	7*
6.5	2	4*	11*
7	2	4*	17*

*Base and Subbase requires encapsulation in geotextile.

TABLE 7.3-2: Recommended Permeable Pavement Sections: Class 2 Permeable Aggregate Base

TRAFFIC INDEX	SECTION	
	THICKNESS FOR ASTM NO. 2 BEDDING COURSE (inches)	CLASS 2 PERMEABLE AGGREGATE BASE (inches)
5.5	2	10
6	2	11
6.5	2	15
7	2	21

We developed these recommendations based on anticipated traffic loading criteria; the final reservoir thickness (aggregate thickness below the No. 2 bedding course) should be developed based on the required storm retention volume identified by the civil engineer.

7.3.1 Construction Recommendations

We provide recommendations for the paver area subgrade preparation in this section.

Construction and materials should follow the recommendations included in this geotechnical report and the ICPI specifications. The pavers should be placed in a 45-degree or 90-degree herringbone-laying pattern. Quality control and assurance is important as part of this specialty construction element and the submitted materials to be used. We recommend that we be engaged to perform quality assurance and observe and approve construction of the pavers, underdrainage, and associated materials for conformance with design, standards, and performance as design intends.

The use of permeable pavers is likely to require ongoing maintenance to address seasonal development of clogs and silting that may develop and reduce permeability characteristics; such maintenance programs should be performed in accordance with manufacturer's requirements.

If the open-graded rock materials are used (Table 7.3.1), then we recommend fully covering the open-graded rock with geotextile (Mirafi 500X or approved equivalent) to reduce potential piping of native soil into the open-graded rock and improve stability of the paver section. This requires the road subgrade, shoulders, and rock filled trenches to be covered with geotextile, including appropriate lapping and wrapping, if necessary. If additional stability of the paver section is needed, bi-axial geogrid (Tensar BX1200 or approved equivalent) can be placed within the rock section.

For paver areas constructed over utility trenches, the potential for water migration into the utility trench backfill should be reduced. We recommend a low-permeability cap, such as clay soil, sand-cement slurry, or lean concrete, be placed within trenches where the trenches pass into pavement areas. Alternatively, a waterproof barrier, such as Visqueen, can be placed at the bottom of the paver section, only within the areas of the utility trenches. The protective barrier should be placed on prepared subgrade, prior to placement of the geotextile or open-graded rock.

If Class 2 permeable material is used, placement of encapsulating geotextile is not needed but the capping of the trenches is still necessary. If using the open-graded rock, the drainage layers specified should be placed in lifts no greater than 8 inches and vibrated using designated methods and equipment under our observation. The open-graded rock or Class 2 permeable material should be placed in lifts no greater than 8 inches on a prepared subgrade that satisfies the recommendations in this report.

7.3.2 Paver Subdrain and Edge Restraints

Considering the site soil is expansive and likely to have a low permeability for infiltration, we recommend that a subdrain system be installed to reduce the amount of distress and maintenance to the pavers as well as manage stormwater. The surface of the prepared subgrade should be sloped to drain toward the subdrain system and the top of the pipe should be at or below the design rock section. The subdrain system should comprise a 4-inch-diameter (SDR 35) perforated pipe (perforations facing down) with glued joints and end caps surrounded by drain rock material. Prior to placing the subdrain pipe and the drain rock backfill, a layer of 6-ounce filter fabric or approved equivalent should be placed flush on the prepared subgrade. The flow path in the subgrade surface and pipe should be sloped at a minimum of ½ percent to drain towards an outlet approved by the civil engineer.

Concrete edge restraints that extend into subgrade below the paver base and subbase should be constructed to provide lateral constraint for the pavers. Additionally, a similar curb should be constructed at the interface between the pervious pavers and adjoining hot mix asphalt (HMA).

7.4 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Exterior flatwork should have a minimum section of 4 inches of concrete over 4 inches of aggregate base. The aggregate base should be compacted to at least 90 percent relative compaction (ASTM D1557). Where pavement areas lie downslope of any landscape areas with a slope of 4 percent or greater that are to be sprinklered or irrigated, flatwork edges should extend to a depth of at least 2 inches below the baserock layer. Control and construction joints should be constructed in accordance with current Portland Cement Association Guidelines.

7.5 SUBGRADE AND AGGREGATE BASE COMPACTION

Finished subgrade and aggregate base should be compacted in accordance with Section 4.4. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

This document must not be subject to unauthorized reuse, that is, reusing without our written authorization. Such authorization is essential because it requires us 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 our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we 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 assigned the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information.

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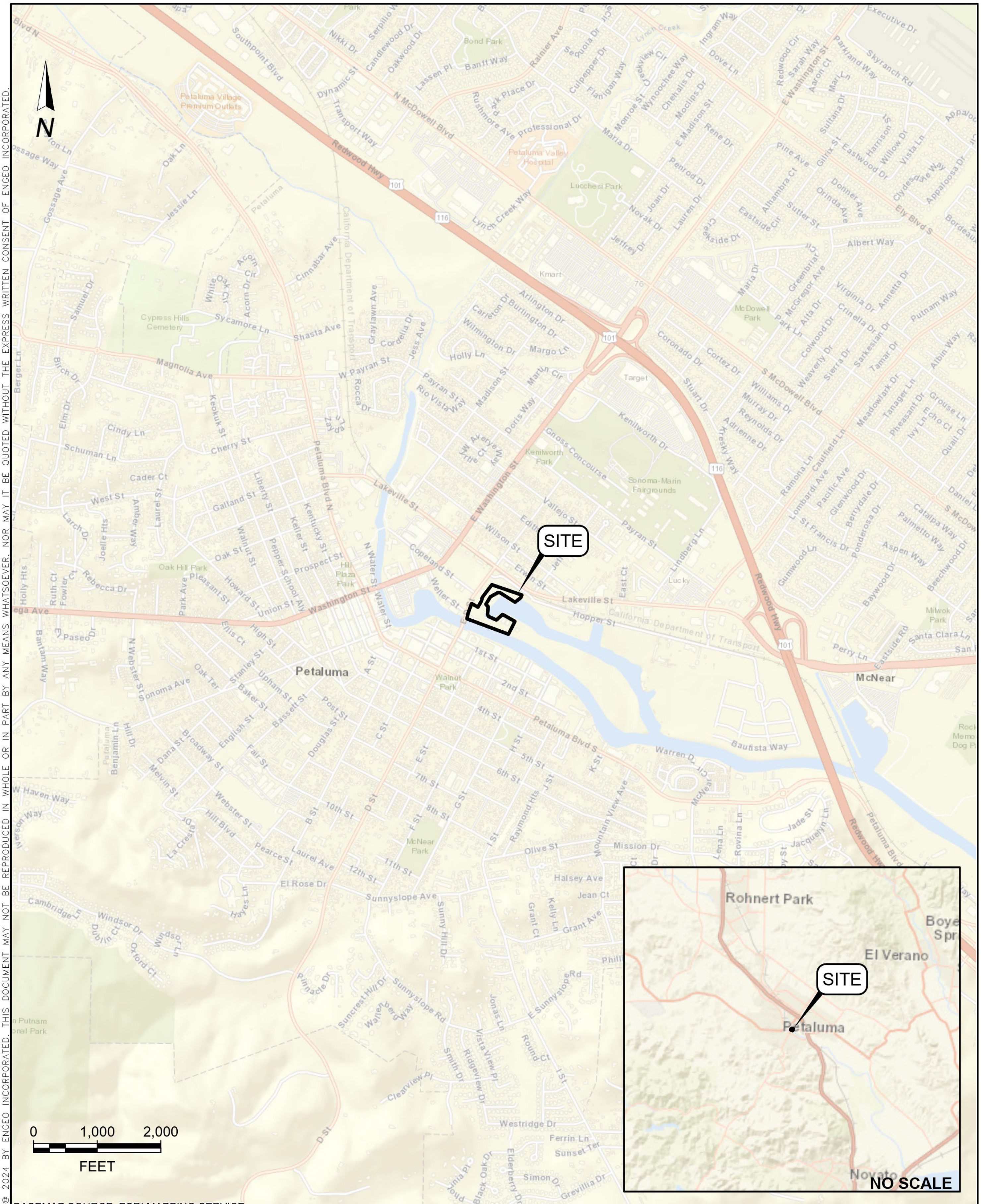
FIGURES



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VICINITY MAP

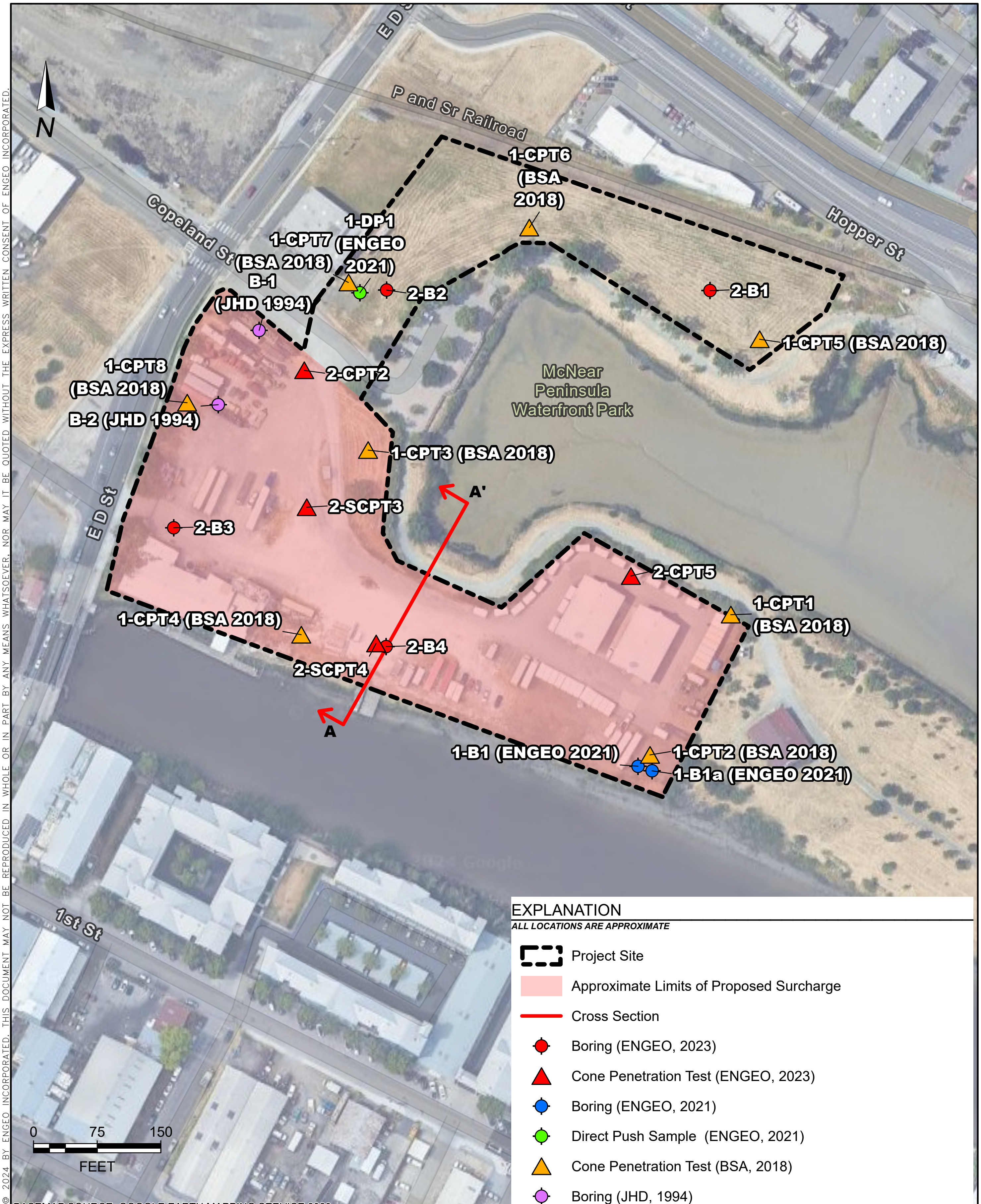
OYSTER COVE

PETALUMA, CALIFORNIA

PROJECT NO. : 15571.003.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: NLK / CHECKED BY: TTB	

PATH: G:\DRAFTING\PROJECTS_14000 TO 15999\15571\15571003000\15571003000_OYSTERCOVER.APRX
LAYOUT: 01. VICINITY 8.5 X 11 PORTRAIT USER: NLAMOTTEKERR

ORIGINAL FIGURE PRINTED IN COLOR



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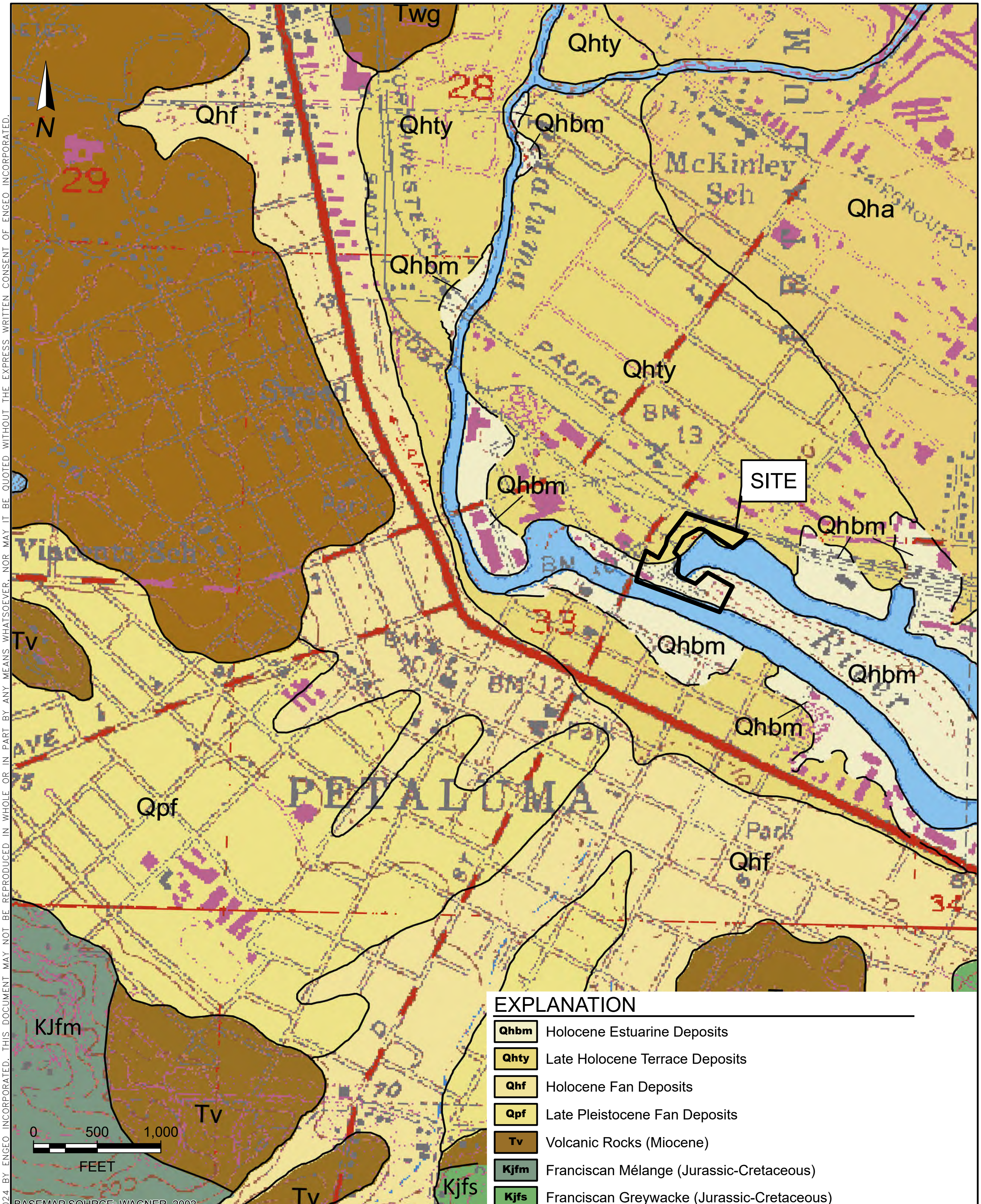
SITE PLAN
OYSTER COVE
PETALUMA, CALIFORNIA

PROJECT NO.: 15571.003.000		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: NLK	CHECKED BY: TTB	

PATH: G:\DRAFTING\PROJECTS_14000 TO 15999\15571\15571003000\15571003000_OYSTERCOVER.APRX
LAYOUT: 02. SITE PLAN 8.5 X 11 PORTRAIT USER: MHAWKINS

ORIGINAL FIGURE PRINTED IN COLOR



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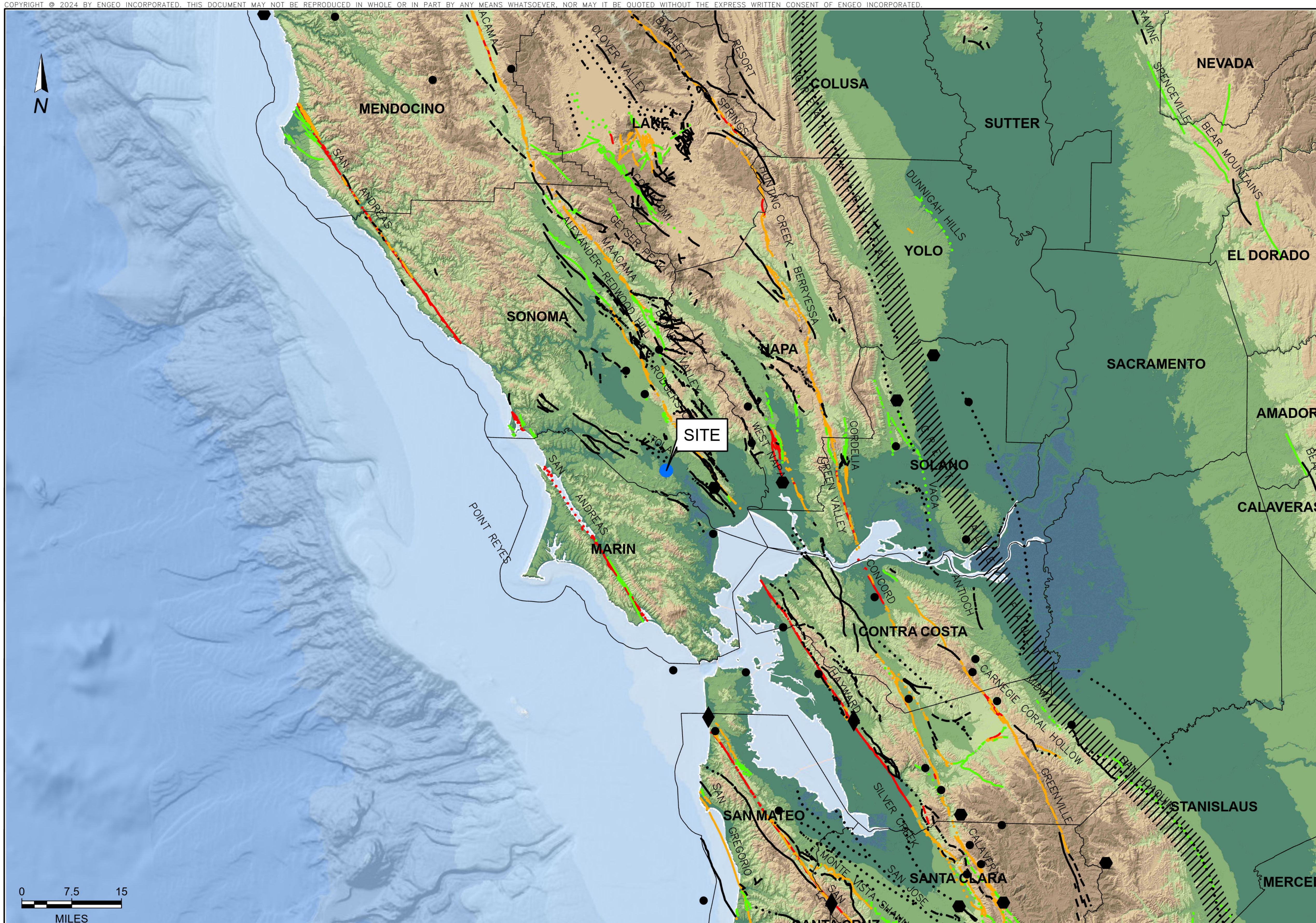
REGIONAL GEOLOGIC MAP
OYSTER COVE
PETALUMA, CALIFORNIA

PROJECT NO.:	15571.003.000
SCALE:	AS SHOWN
DRAWN BY: NLK	CHECKED BY: TTB
FIGURE NO.	3

PATH: G:\DRAFTING\PROJECTS_14000 TO 15999\15571\15571003000\15571003000_OYSTERCOVER.APRX
LAYOUT: 03. GEOLOGIC 8.5 X 11 PORTRAIT USER: MHAWKINS

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- Project Site

HISTORIC EARTHQUAKE EPICENTERS

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

QUATERNARY FAULTS 2020

Based on time of most recent surface deformation

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

BASE MAP SOURCE:
CSUMB, ESRI, GARMIN, NATURALVUE, ESRI, GEBCO, GARMIN, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
C.G.S. HISTORIC EARTHQUAKE DATABASE

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REGIONAL FAULTING AND SEISMICITY MAP
OYSTER COVE
PETALUMA, CALIFORNIA

PROJECT NO.: 15571.003.000		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: NLK	CHECKED BY: TTB	

PATH: G:\DRAFTING\PROJECTS_14000 TO 15999\15571\15571003000\15571003000_OYSTERCOVER.APRX
LAYOUT: 04. FAULT&SEISMICITY 11 X 17 LANDSCAPE USER: NLAMOTTEKERR

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

KEY TO BORING LOGS
EXPLORATION 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: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

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

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
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

GROUNDWATER 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 2-B1

LATITUDE: 38.2357 LONGITUDE: -122.633

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 15 ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Mud Rotary
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
			TOPSOIL 6"												
			LEAN CLAY WITH SAND (CL), dark grayish brown, medium stiff, moist, medium plasticity, fine- to coarse-grained sand [FILL]												
						10					27.2	91.9	694		UC
			LEAN CLAY (CL), dark brown mottled with yellowish red, medium stiff to hard, moist, iron oxide and manganese staining, contains <15% coarse-grained sand and fine rounded gravel [NATIVE]												
5	10														
						12					31.5	79.6	720		UC
			Sand and fine gravel content increases towards bottom of sampler								22.5	87.6			
			LEAN CLAY WITH SAND (CL), yellowish red to yellowish brown, stiff, moist, medium plasticity, fine- to coarse-grained sand, trace fine rounded gravel												
10	5														
						28					17			>4.5*	PP
15	0														
						17					25.2	100.7		3.25*	PP
20	-5														

LOG OF BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 38.2357 LONGITUDE: -122.633

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 15 ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Mud Rotary
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
			LEAN CLAY WITH SAND (CL), yellowish red to yellowish brown, stiff, moist, medium plasticity, fine- to coarse-grained sand, trace fine rounded gravel			11	35	17	18		23.2			1.75*	PP
						13									
25	-10														
						15									
			Boring terminated at approximately 26½ feet below ground surface. Groundwater not observed due to drilling method. Boring backfilled with cement grout.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 15571003000_OYSTER COVE.GPJ ENGEO INC.GDT 1/29/24

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LOG OF BORING 2-B2

LATITUDE: 38.2348 LONGITUDE: -122.6352

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 27 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 12½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, 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
			TOPSOIL 6"												
			LEAN CLAY WITH SAND (CL), very dark grayish brown, moist, fine- to medium-grained sand, trace fine subangular to rounded gravel [FILL]												
			LEAN CLAY WITH SAND (CL), yellowish brown mottled with olive gray, stiff, moist, medium plasticity, fine- to medium-grained sand, iron oxide staining [NATIVE]												
	10														
						13								1.75*	PP
5															
			Grades to very stiff			34					16.9	110.3	2582		UC
	5														
						26								4.0*	PP
			CLAYEY SAND (SC), yellowish brown, medium dense, wet, fine- to medium-grained sand, trace fine rounded gravel												
10															
						11									
					▽										
	0														
			POORLY GRADED SAND WITH CLAY AND GRAVEL (SP-SC), dark yellowish brown to yellowish red, medium dense, wet, fine subangular to rounded gravel, medium- to coarse-grained sand, ~20-30% gravel, <12% fines												
15															
						24				8					
	-5														
20															

LOG OF BORING 2-B2

LATITUDE: 38.2348 LONGITUDE: -122.6352

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 27 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 12½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, 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), dark yellowish brown to yellowish red, medium dense, wet, fine subangular to rounded gravel, coarse-grained sand			18									
	-10		LEAN CLAY WITH SAND (CL), yellowish brown, stiff to very stiff, wet, medium plasticity												
25						32									
						21					12.1				

Boring terminated at approximately 27 feet below ground surface.

Groundwater encountered at approximately 10 feet below ground surface.

Boring backfilled with cement grout.

LOG OF BORING 2-B3

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LATITUDE: 38.2345 LONGITUDE: -122.6343

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 23 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 11½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Water Level	Blow Count/Foot	Liquid Limit	Plastic Limit	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
		ASPHALT CONCRETE (AC) 1"											
		AGGREGATE BASE (AB) 3"											
	10	CLAYEY SAND WITH GRAVEL (SC), reddish brown, moist, fine- to medium-grained sand, fine angular to subrounded gravel [FILL]		14									
		CLAYEY SAND (SC), very dark grayish brown, loose, wet, fine- to medium-grained sand, contains fine angular to rounded gravel, pockets of yellowish red clay [FILL]											
5	5	SANDY FAT CLAY (CH), dark grayish brown, soft, moist, high plasticity, fine- to medium-grained sand, trace coarse-grained sand [NATIVE]	∇	4					38	82.2	266		UC
10	0	FAT CLAY (CH), very dark greenish gray, soft to medium stiff, moist, high plasticity, organic odor, contains rootlets/organics							126.1 145.6	36.3 31.3	886		UU
15	-5			5					51.6			0.75*	PP
20													

LOG - GEOTECHNICAL_SU+QU W/ ELEV 15571003000_OYSTER COVE.GPJ ENGEO INC.GDT 1/29/24

LOG OF BORING 2-B3

ENGEO Expect Excellence

LATITUDE: 38.2345 LONGITUDE: -122.6343

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 23 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 11½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, 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
			FAT CLAY (CH), very dark greenish gray, soft to medium stiff, moist, high plasticity, organic odor, contains rootlets/organics			53					30.6			2.75*	PP
	-10		POORLY GRADED SAND WITH CLAY (SP-SC), dark yellowish brown, very dense, wet, medium- to coarse-grained sand, trace fine rounded to subrounded gravel			51									
			Boring terminated at approximately 23 feet below ground surface. Groundwater encountered 6 feet below ground surface. Boring backfilled with cement grout.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 15571003000_OYSTER COVE.GPJ ENGEO INC.GDT 1/29/24

LOG OF BORING 2-B4

ENGEO — Expect Excellence

LATITUDE: 38.2356 LONGITUDE: -122.6343

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 29½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 12½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

DESCRIPTION	Water Level	Blow Count/Foot	Liquid Limit	Plastic Limit	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
SILTY SAND WITH GRAVEL (SM), white to pale olive, fine- to coarse-grained sand, fine angular gravel, contains crushed shells and shell fragments [FILL]		50/6"									
CLAYEY SAND WITH GRAVEL (SC), reddish brown to olive gray, moist, fine- to coarse-grained sand, fine and coarse rounded to angular gravel, pockets of clay	▽	11									
FAT CLAY (CH), dark greenish gray, soft to medium stiff, moist, high plasticity, contains organics/rootlets, shell fragments, trace fine-grained sand, manganese staining [NATIVE]											
1/4" layer of organics/rootlets at 8.5'		5							600*	0.625*	PP+TV
		100 psi									
							78.5	55	205		UU
									550*	0.75*	PP+TV
		100 psi					86.1	51.6	400		UU
Greenish gray, soft											

LOG OF BORING 2-B4

ENGEO Expect Excellence

LATITUDE: 38.2356 LONGITUDE: -122.6343

Geotechnical Exploration
Oyster Cove
Petaluma, CA
15571.003.000

DATE DRILLED: 1/5/2024
HOLE DEPTH: Approx. 29½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 12½ ft.

LOGGED / REVIEWED BY: K. Katzenberger / JAF
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, 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
	-10		CLAYEY SAND (SC), greenish gray, medium dense, moist, medium- to coarse-grained sand, trace fine angular gravel			33					21.1	109.8			
						20				32					
25															
	-15		Grades to dense and yellowish brown, iron oxide staining, gravel lense at 29'			45									
			LEAN CLAY WITH GRAVEL (CL), yellowish brown, hard, moist, fine angular gravel, trace medium-grained sand												

Boring terminated at approximately 29½ feet below ground surface.

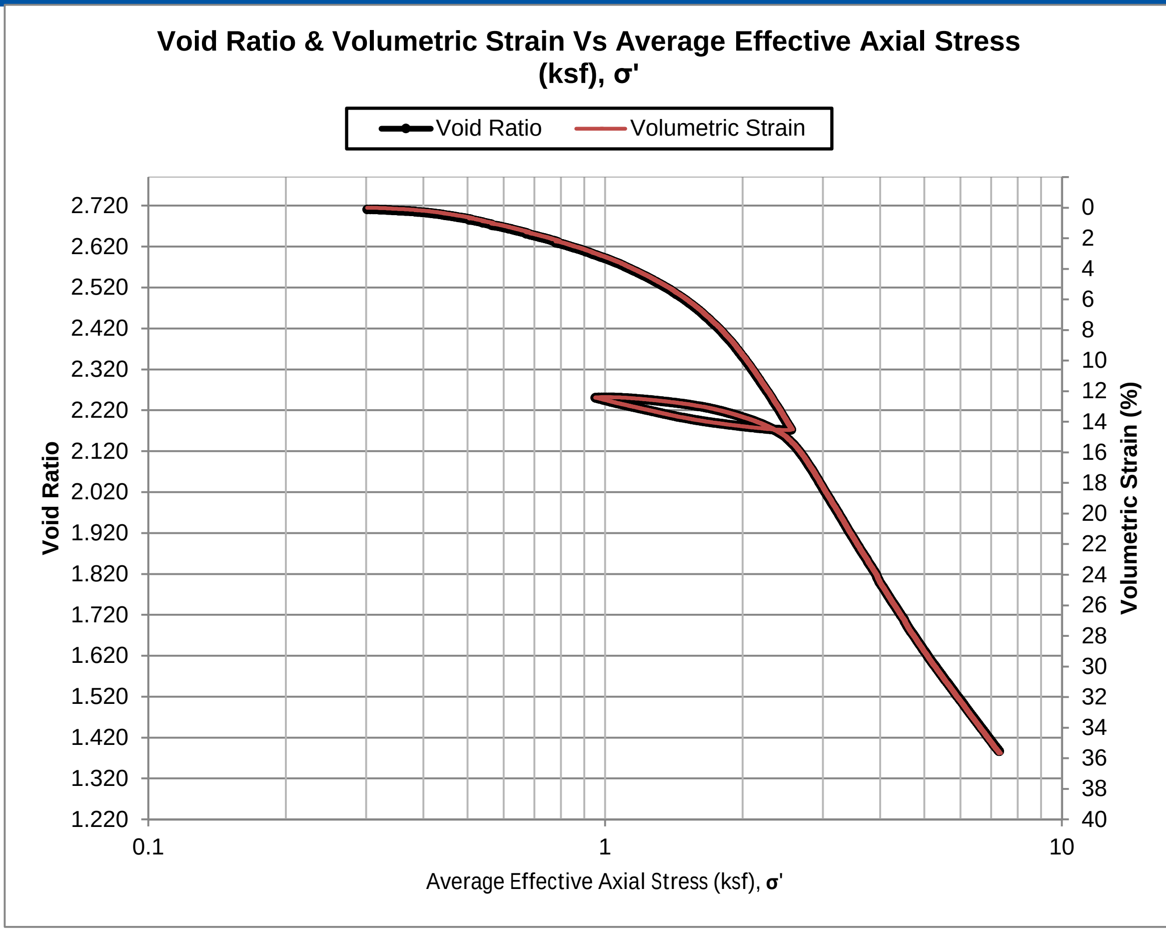
Groundwater encountered at approximately 6 feet below ground surface.

Boring backfilled with cement grout.

APPENDIX B

LABORATORY TEST RESULTS

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	2-B3 at 10-12.5	**DEPTH:**	12-12.5 ft.
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

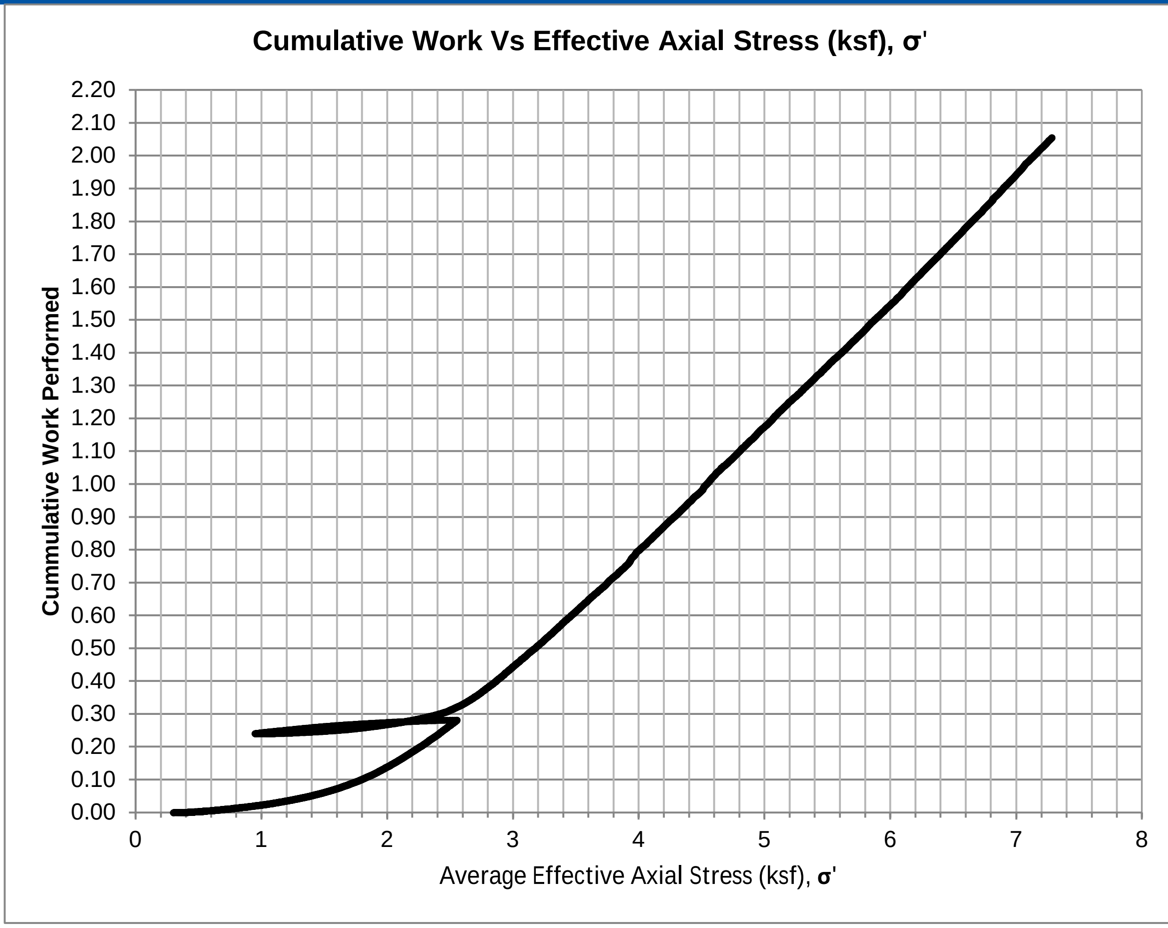
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	145.62	84.49	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	31.26	48.62	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	2.711	1.386	**SPECIFIC GRAVITY**	1.862
STRAIN RATE (in/min):	0.000056			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000-P:001
PROJECT LOCATION: Petaluma, California
REPORT DATE: 1/19/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 2-B3 at 10-12.5 **DEPTH:** 12-12.5 ft.

SOIL DESCRIPTION: See exploration logs

REMARKS:

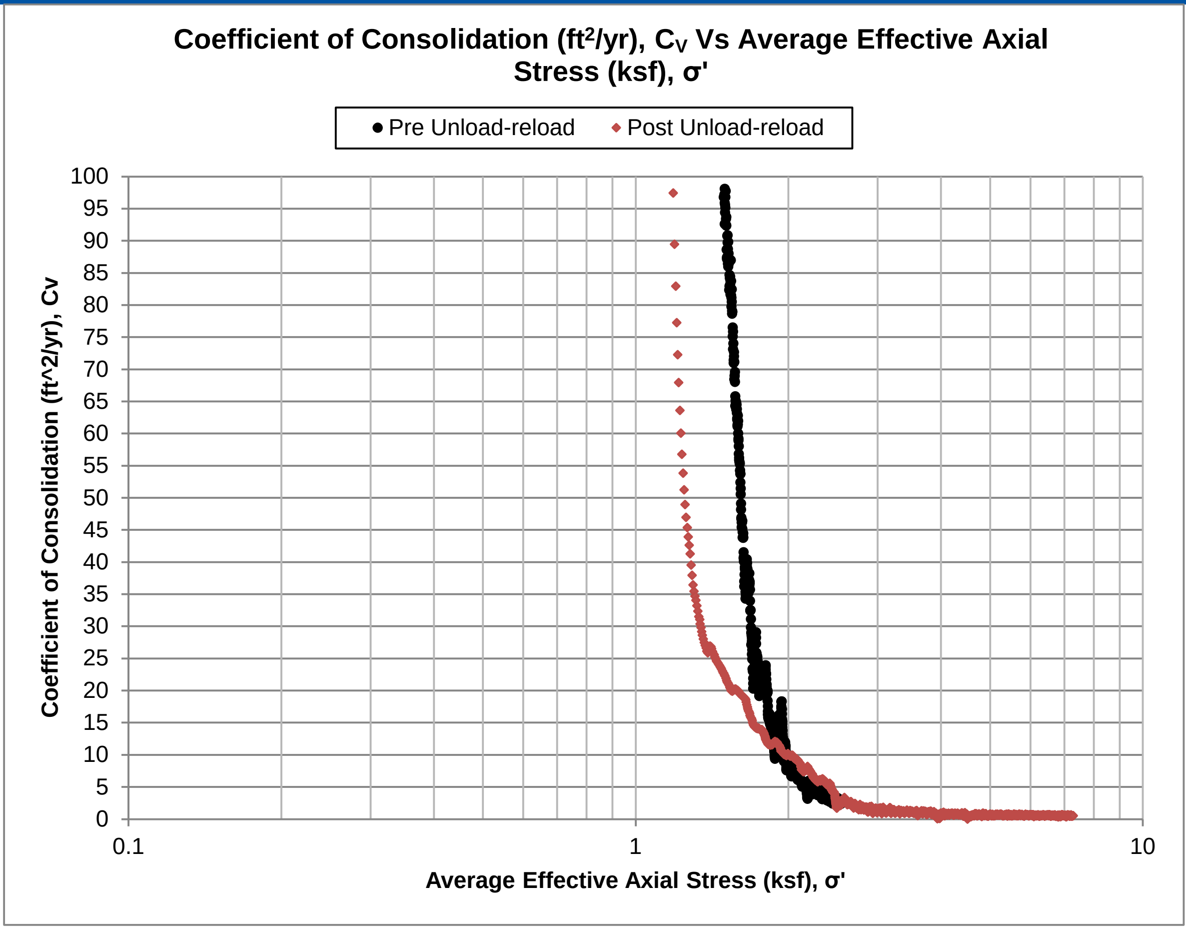
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	145.62	84.49	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	31.26	48.62	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	2.711	1.386	**SPECIFIC GRAVITY**	1.862
STRAIN RATE (in/min):	0.000056			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000-P:001
PROJECT LOCATION: Petaluma, California
REPORT DATE: 1/19/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 2-B3 at 10-12.5 **DEPTH:** 12-12.5 ft.

SOIL DESCRIPTION: See exploration logs

REMARKS:

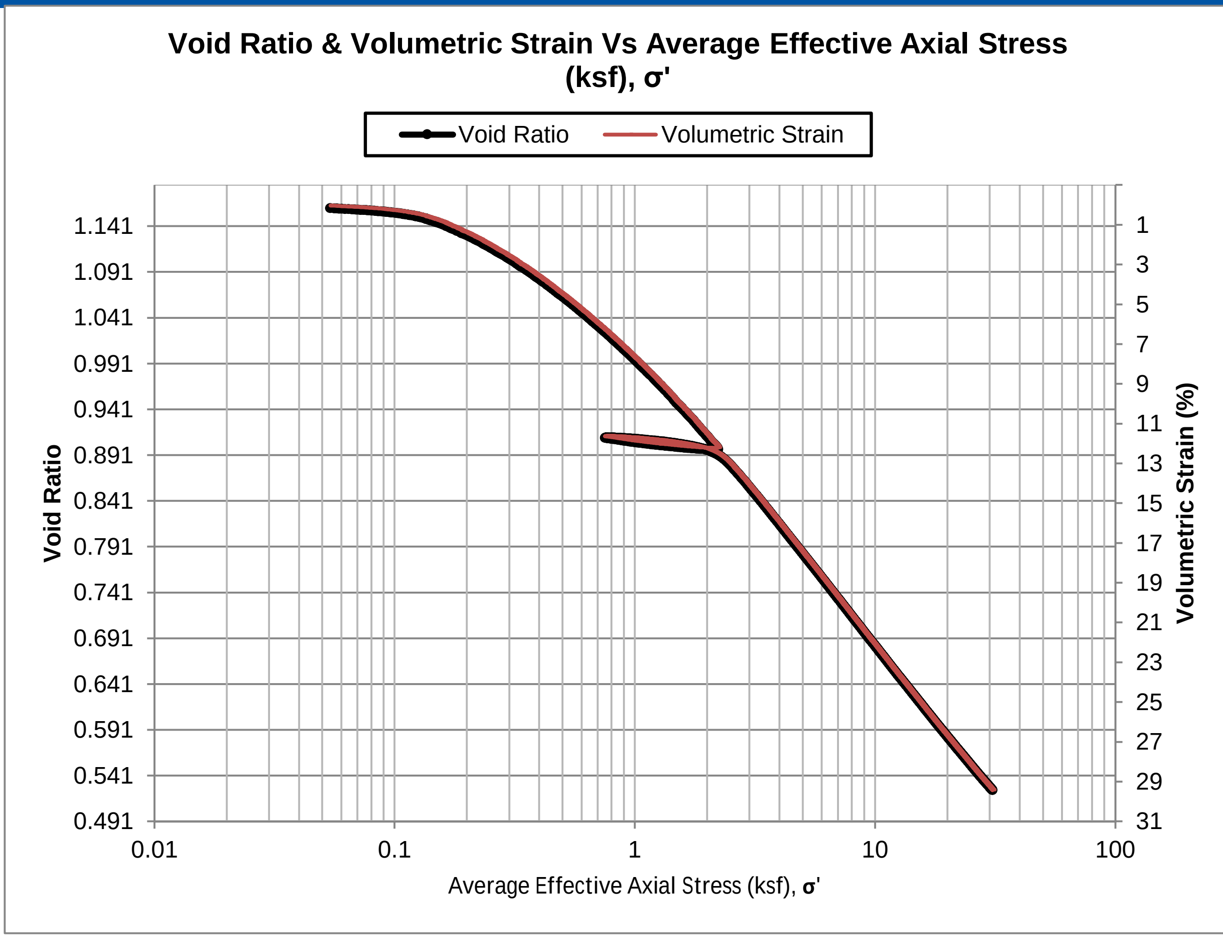
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	145.62	84.49	LIQUID LIMIT:	
DRY DENSITY (pcf):	31.26	48.62	PLASTIC LIMIT:	
SATURATION (%):	100.00	100.00	ASTM D854 - Measured	
VOID RATIO:	2.711	1.386	SPECIFIC GRAVITY	1.862
STRAIN RATE (in/min):	0.000056			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000-P:001
PROJECT LOCATION: Petaluma, California
REPORT DATE: 1/19/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 2-B4@9.5-12' **DEPTH:** 11-11.5'

SOIL DESCRIPTION: See exploration logs

REMARKS:

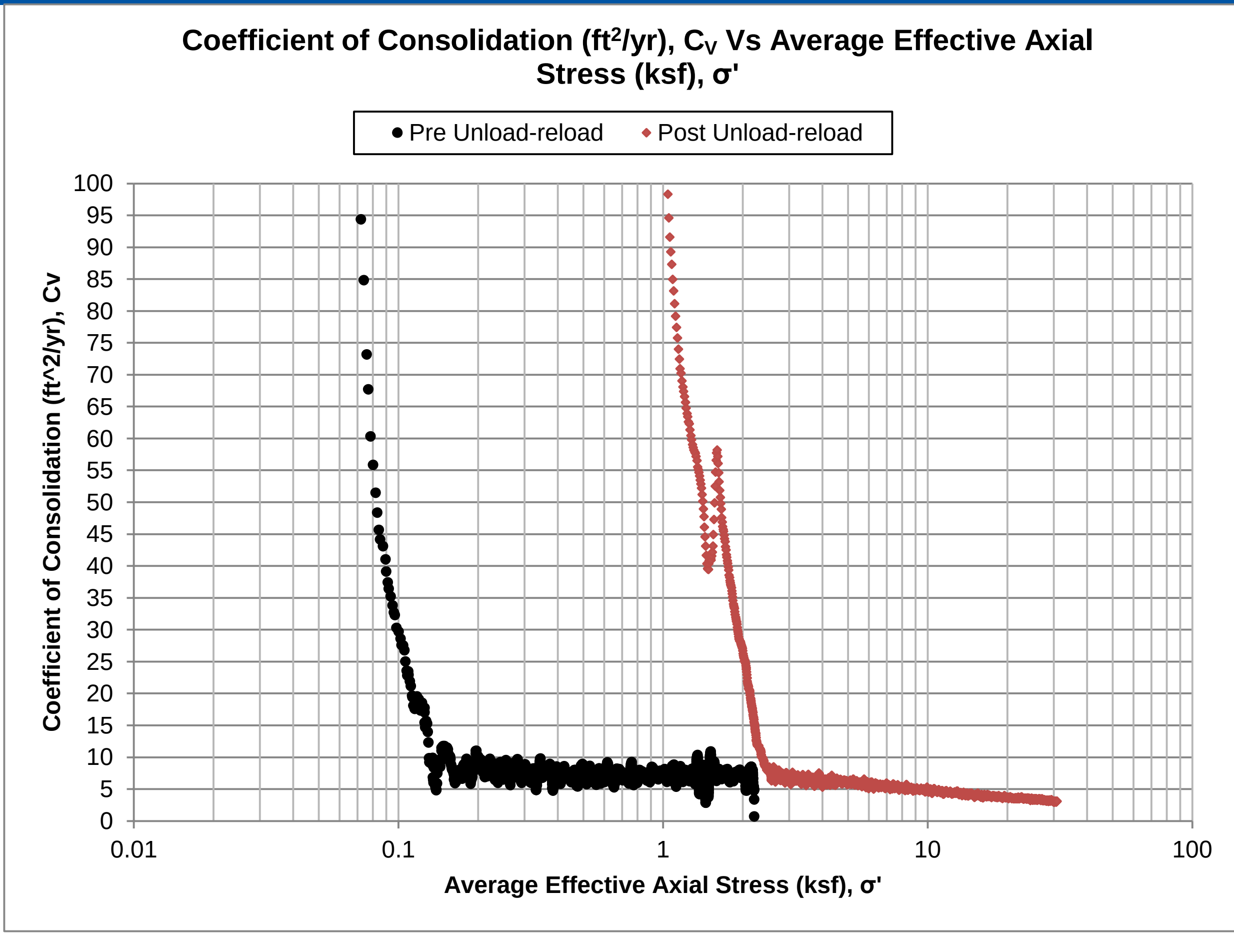
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	44.26	26.92	LIQUID LIMIT:	
DRY DENSITY (pcf):	75.64	107.22	PLASTIC LIMIT:	
SATURATION (%):	100.00	100.00	ASTM D854 - Measured	
VOID RATIO:	1.161	0.525	SPECIFIC GRAVITY	2.623
STRAIN RATE (in/min):	0.000060			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/20/2024
TESTED BY: K. Nguyen
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	2-B4@9.5-12'	**DEPTH:**	11-11.5'
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

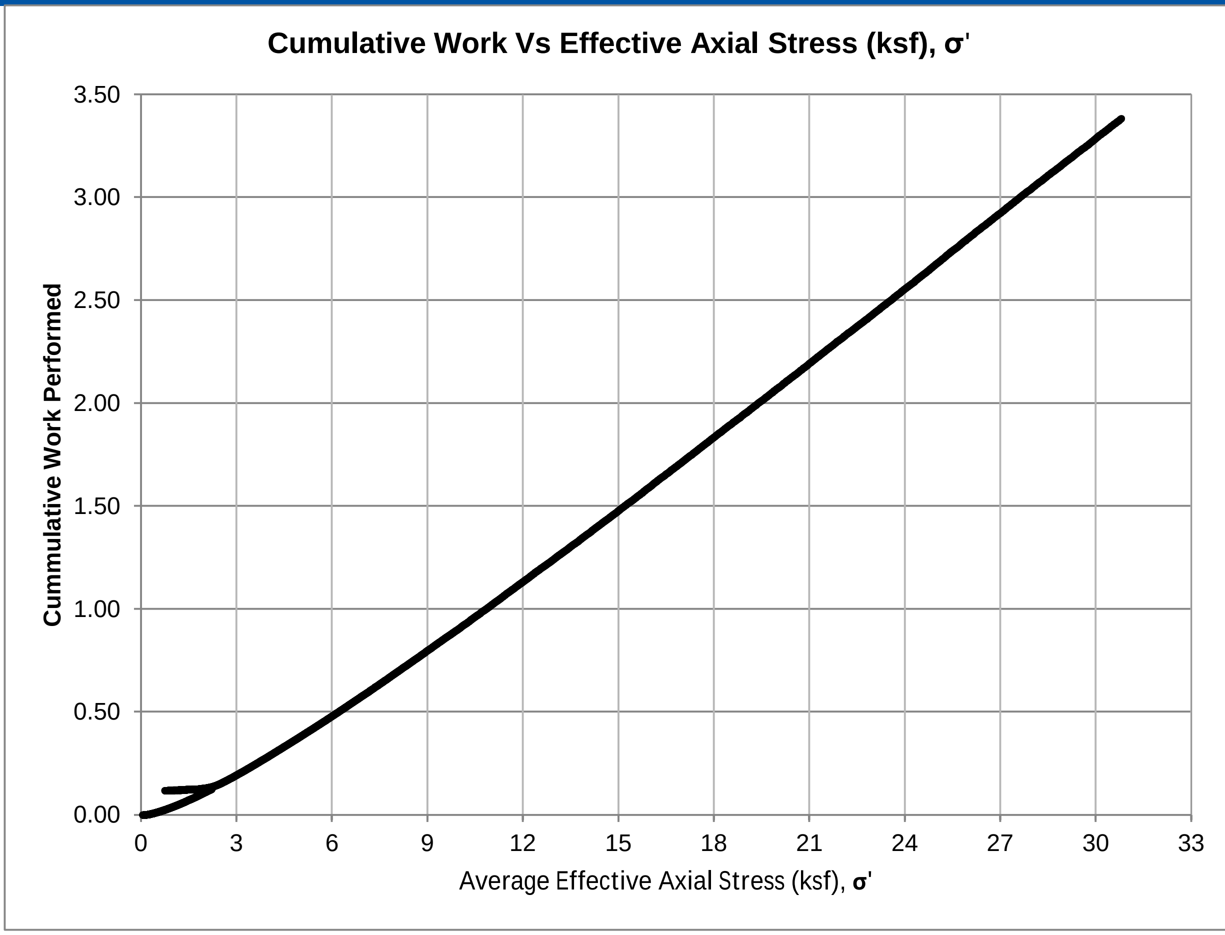
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	44.26	26.92	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	75.64	107.22	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	1.161	0.525	**SPECIFIC GRAVITY**	2.623
STRAIN RATE (in/min):	0.000060			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/20/2024
TESTED BY: K. Nguyen
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	2-B4@9.5-12'	**DEPTH:**	11-11.5'
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

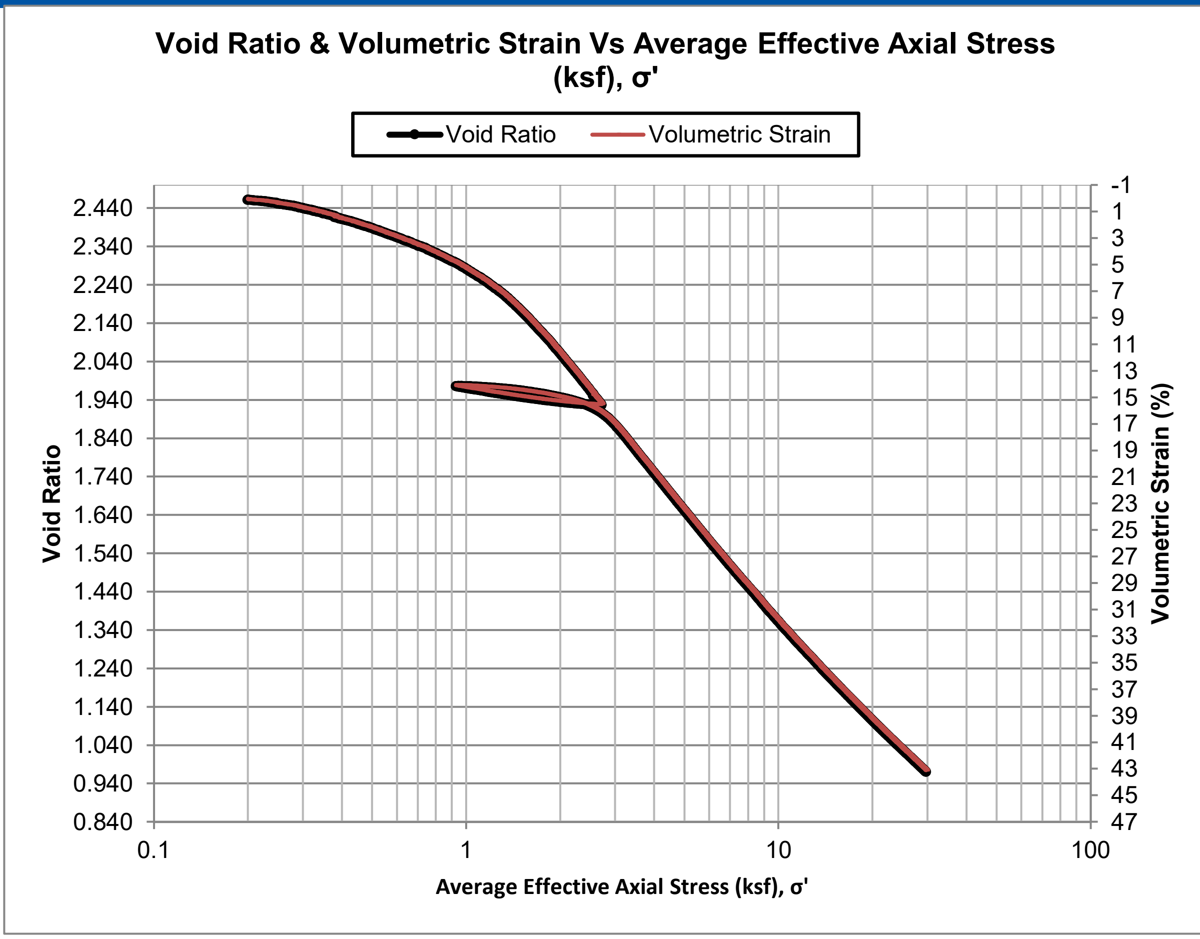
TEST DATA

	INITIAL	FINAL	**ASTM D4318 - Wet Method**	
MOISTURE CONTENT (%):	44.26	26.92	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	75.64	107.22	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	1.161	0.525	**SPECIFIC GRAVITY**	2.623
STRAIN RATE (in/min):	0.000060			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/20/2024
TESTED BY: K. Nguyen
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 2-B4@16-18.5' **DEPTH:** 17.5-18'

SOIL DESCRIPTION: See exploration logs

REMARKS:

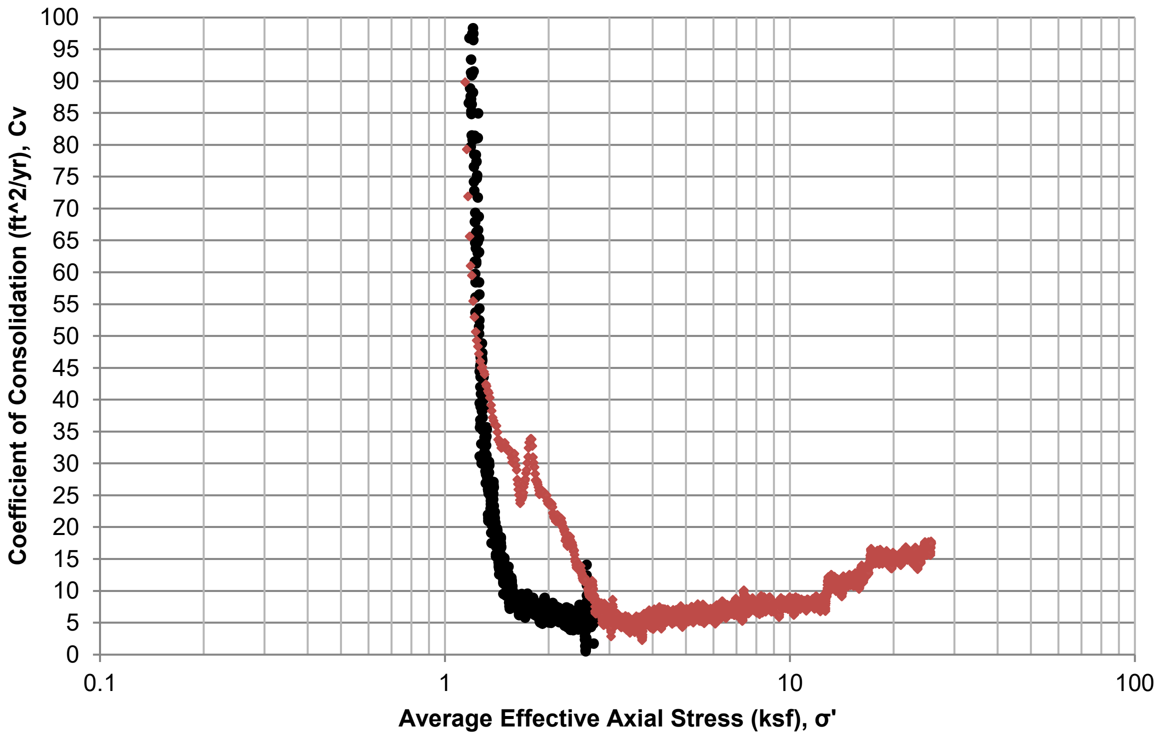
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	89.09	46.53	LIQUID LIMIT:	
DRY DENSITY (pcf):	48.75	85.79	PLASTIC LIMIT:	
SATURATION (%):	98.00	100.50	ASTM D854 - Measured	
VOID RATIO:	2.463	0.968	SPECIFIC GRAVITY	2.709
STRAIN RATE (in/min):	0.000080			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/19/2024
TESTED BY: O. Espinoza
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	2-B4@16-18.5'	**DEPTH:**	17.5-18'
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

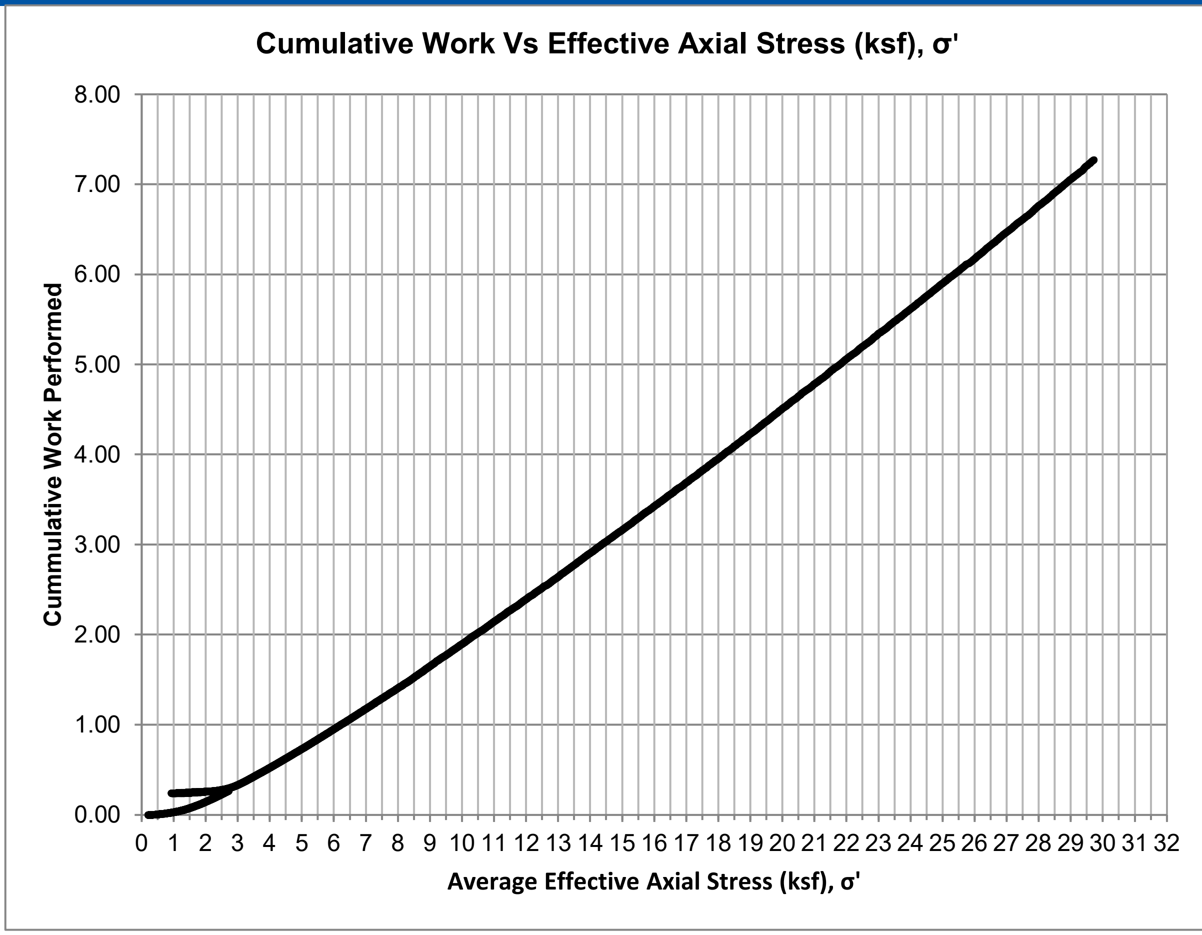
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	89.09	46.53	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	48.75	85.79	**PLASTIC LIMIT:**	
SATURATION (%):	98.00	100.50	**ASTM D854 - Measured**	
VOID RATIO:	2.463	0.968	**SPECIFIC GRAVITY**	2.709
STRAIN RATE (in/min):	0.000080			

ENGEO — Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/19/2024
TESTED BY: O. Espinoza
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 2-B4@16-18.5' **DEPTH:** 17.5-18'

SOIL DESCRIPTION: See exploration logs

REMARKS:

TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	89.09	46.53	LIQUID LIMIT:	
DRY DENSITY (pcf):	48.75	85.79	PLASTIC LIMIT:	
SATURATION (%):	98.00	100.50	ASTM D854 - Measured	
VOID RATIO:	2.463	0.968	SPECIFIC GRAVITY	2.709
STRAIN RATE (in/min):	0.000080			

ENGEO Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/19/2024
TESTED BY: O. Espinoza
REVIEWED BY: D. Seibold

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	2-B1@ 11-11.5	2-B1@ 20.5-21.5	2-B2@ 25.5-27	2-B3@ 15.5-16	2-B3@ 21-21.5			
DEPTH (ft.)	11-11.5	20.5-21.5	25.5-27	15.5-16	21-21.5			
METHOD A OR B	B	B	B	B	B			
MOISTURE CONTENT (%)	17.0	23.2	12.1	51.6	30.6			

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/12/2024
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

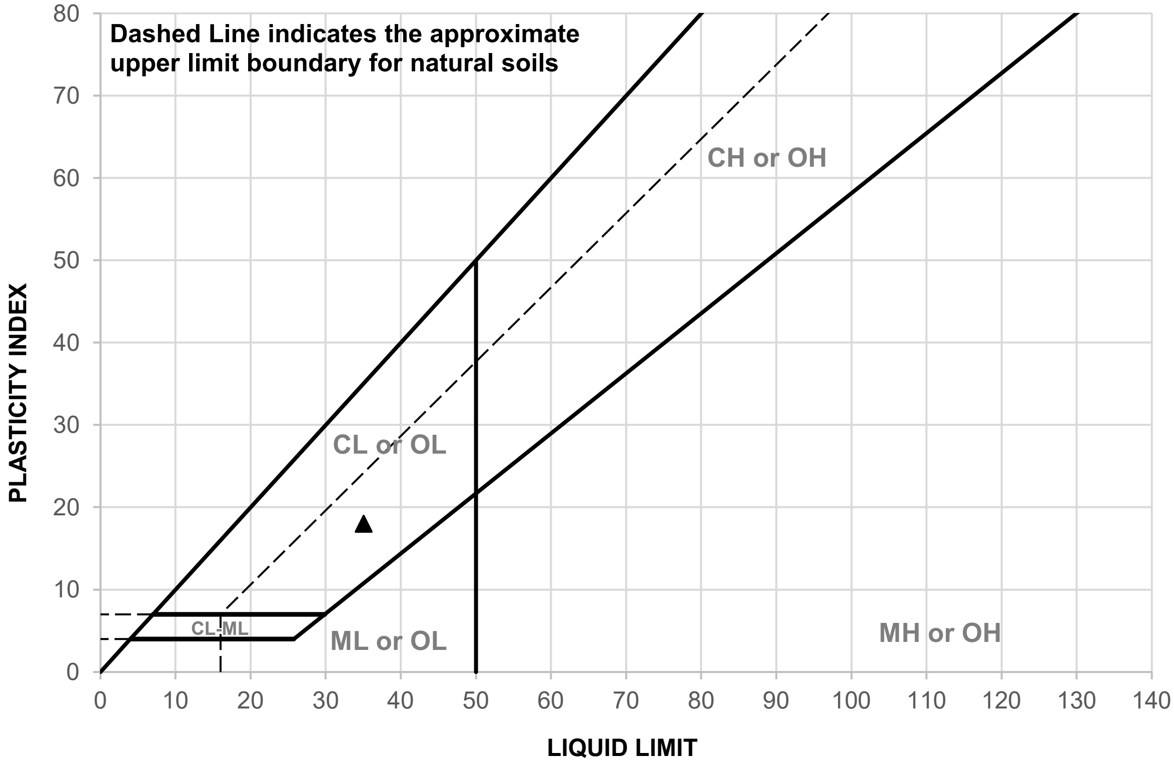
MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	2-B1@ 6-6.5	2-B1@ 16-16.5	2-B4@ 22.5-23					
DEPTH (ft.)	6-6.5	16-16.5	22.5-23					
METHOD A OR B	B	B	B					
MOISTURE CONTENT (%)	22.5	25.2	21.1					
DRY DENSITY (pcf)	87.6	100.7	109.8					

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/12/2024
TESTED BY: Y. Cabrales
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	2-B1@20.5-21.5	20.5-21.5	See exploration logs	35	17	18

	SAMPLE ID	TEST METHOD	REMARKS
▲	2-B1@20.5-21.5	PI: ASTM D4318, Wet Method	

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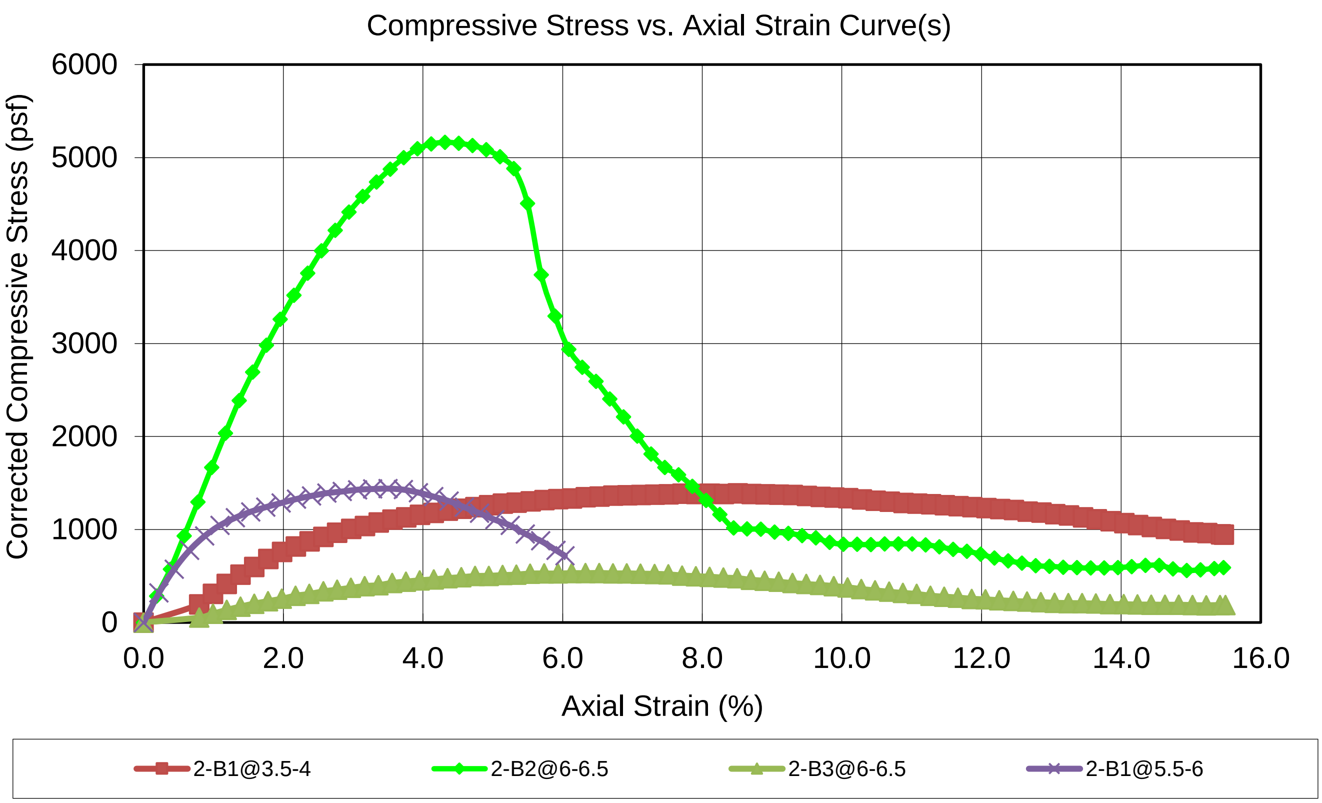
CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/15/2023
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 2-B1@3.5-4	SPECIMEN 2-B2@6-6.5	SPECIMEN 2-B3@6-6.5	SPECIMEN 2-B1@5.5-6
Test Moisture Content (%)	27.22	16.89	37.95	31.53
Dry Density (pcf)	91.9	110.3	82.2	79.6
Saturation (%)	87.4	85.2	96.9	75.7
Void Ratio	0.85	0.54	1.07	1.13
Diameter (in)	2.413	2.387	2.383	2.380
Height (in)	5.056	5.083	5.060	4.572
Height-To-Diameter Ratio	2.10	2.13	2.12	1.92
TEST DATA				
Unconfined Compressive Strength (psf)	1387.55	5164.05	532.47	1439.63
Undrained Shear Strength (psf)	693.78	2582.02	266.23	719.81
Strain Rate (in/min)	0.051	0.051	0.051	0.046
Specific Gravity (ASSUMED)	2.720	2.720	2.720	2.720
Strain at Failure(%)	8.51	4.32	6.52	3.50
Test Remarks				

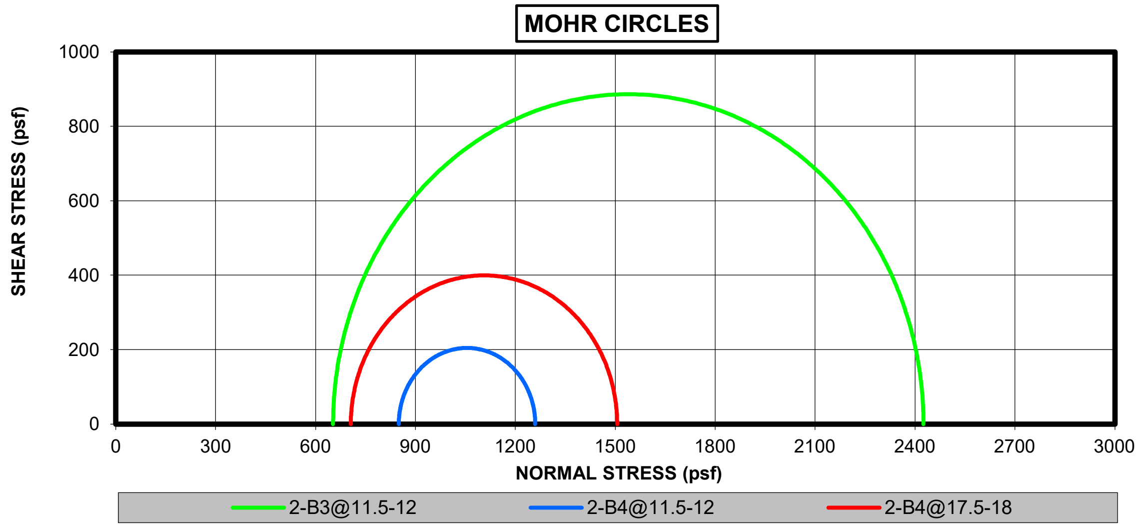
SPECIMEN	DESCRIPTION
2-B1@3.5-4	See exploration logs
2-B2@6-6.5	See exploration logs
2-B3@6-6.5	See exploration logs
2-B1@5.5-6	See exploration logs

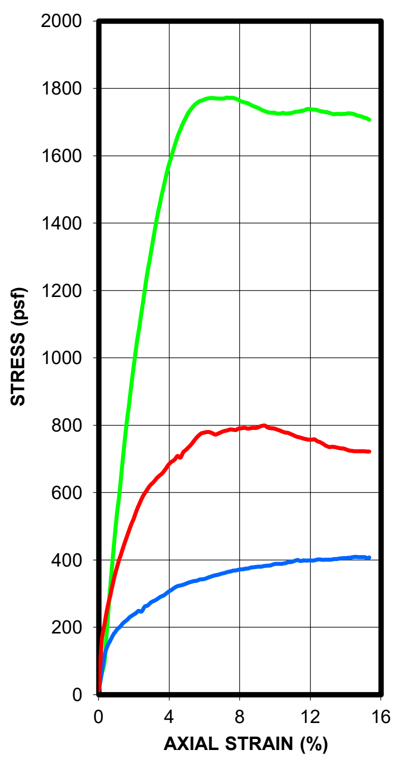
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
CLIENT: Brookfield Bay Area Holdings, LLC
LOCATION: Petaluma, CA

Test Date: 1/17/24
Tested By: Y. Cabrales
Reviewed By: G. Criste

ENGEO Expect Excellence

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850





SPECIMEN				
INITIAL PARAMETERS	2-B3@11.5-12	2-B4@11.5-12	2-B4@17.5-18	
MOISTURE (%)	126.05	78.45	86.05	
DRY DENSITY (PCF)	36.30	55.00	51.60	
SATURATION (%)	93.33	99.92	100.00	
VOID RATIO	3.673	2.237	2.460	
DIAMETER (IN.)	2.834	2.836	2.835	
HEIGHT (IN.)	5.936	5.770	6.050	
DIAMETER-TO-HEIGHT RATIO	2.095	2.035	2.134	
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.850	2.860	
FINAL PARAMETERS	2-B3@11.5-12	2-B4@11.5-12	2-B4@17.5-18	
MOISTURE (%)	126.05	78.45	86.05	
SATURATION (%)	93.33	99.92	100.00	
STRAIN RATE (%/MIN.)	0.059	0.058	0.061	
PEAK DEVIATOR STRESS (PSF)	1772.7	409.7	799.7	
AXIAL STRAIN AT FAILURE (%)	7.244	14.558	9.422	
CELL PRESSURE				
CELL PRESSURE (PSF)	652.3	566.0	705.6	
BACK PRESSURE (PSF)	n/a	n/a	n/a	
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	2425.0	1259.3	1505.3	
σ3 (PSF)	652.3	566.0	705.6	
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	886.4	204.9	399.9	
REMARKS				

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/16/2024
TESTED BY: G. Criste
REVIEWED BY: D. Seibold

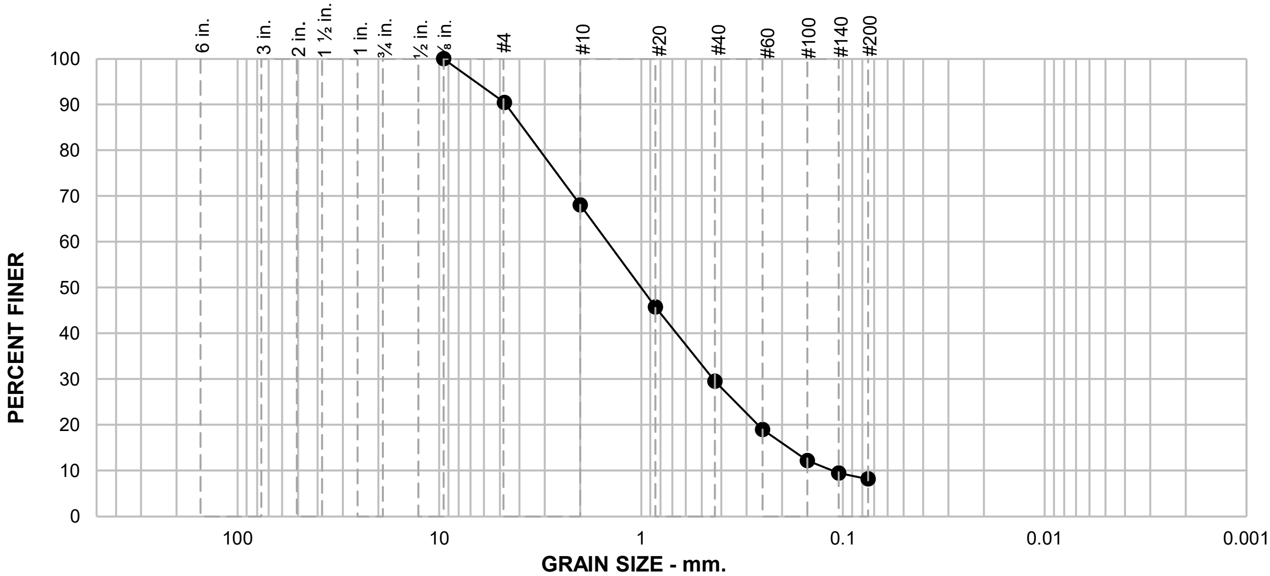
ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850

ENGEO
Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC
PROJECT NAME: Oyster Cove
PROJECT NO: 15571.003.000 PH001
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 1/16/2024
TESTED BY: G. Criste
REVIEWED BY: D. Seibold

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 2-B2@15-16.5

DEPTH (ft): 15-16.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		10	22	39	21	8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100		
#4	90		
#10	68		
#20	46		
#40	30		
#60	19		
#100	12		
#140	9		
#200	8		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 4.7500 mm D_{85} = 3.9023 mm D_{60} = 1.4652 mm

D_{50} = 0.9931 mm D_{30} = 0.4300 mm D_{15} = 0.1867 mm

D_{10} = 0.1183 mm C_u = 12.39 C_c = 1.07

CLASSIFICATION

USCS =

REMARKS

ENGEO Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC

PROJECT NAME: Oyster Cove

PROJECT NO: 15571.003.001 PH001

PROJECT LOCATION: Petaluma, CA

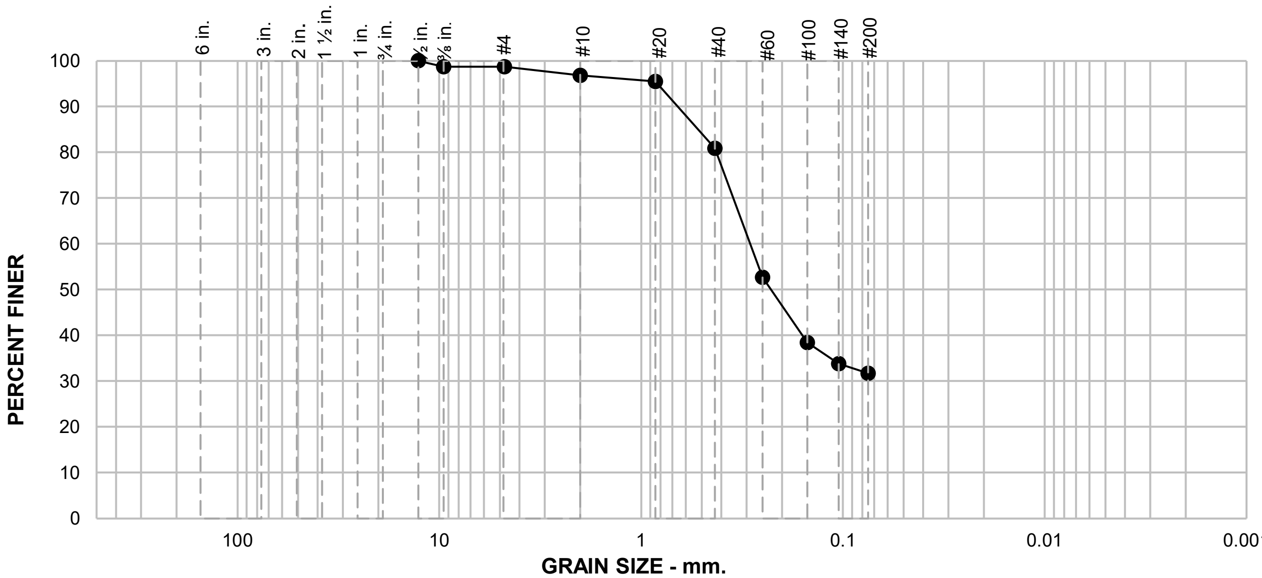
REPORT DATE: 1/15/2024

TESTED BY: Y. Cabrales

REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 2-B4@23.5-25

DEPTH (ft): 23.5-25

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		1	2	16	49	32	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
½ in.	100		
⅜ in.	99		
#4	99		
#10	97		
#20	95		
#40	81		
#60	53		
#100	38		
#140	34		
#200	32		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 0.6664 mm D_{85} = 0.5224 mm D_{60} = 0.2863 mm

D_{50} = 0.2257 mm D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

ENGEO Expect Excellence

CLIENT: Brookfield Bay Area Holdings, LLC

PROJECT NAME: Oyster Cove

PROJECT NO: 15571.003.001 PH001

PROJECT LOCATION: Petaluma, CA

REPORT DATE: 1/15/2024

TESTED BY: Y. Cabrales

REVIEWED BY: G. Criste

APPENDIX C

CONE PENETRATION TEST RESULTS

PRESENTATION OF SITE INVESTIGATION RESULTS

Oyster Cove Petaluma

Prepared for:

ENGEO, Inc.

ConeTec Job No: 24-56-27037

Project Start Date: 2024-01-04
Project End Date: 2024-01-04
Report Date: 2024-01-09

Prepared by:

ConeTec Inc.

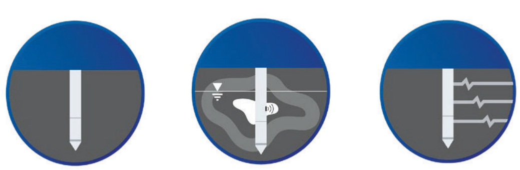
506 De Carlo Ave., Richmond, CA 80177

Tel: (510)357-3677

ConeTecCA@conetec.com

www.conetec.com

www.conetecdataservices.com



ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Seismic Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project	
Client	ENGEO, Inc.
Project	Oyster Cove Petaluma
ConeTec Project Number	24-56-27037
Rig Description	30-ton Truck CPT Rig (C-15)
Test Types	CPTu/SCPTu
Additional Comments	None

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM ZONE 10)
Additional Comments	None

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

Project Information

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.
Additional Comments	

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

CONETEC

LIMITATIONS

3rd Party Disclaimer

- The "Report" refers to this report titled Oyster Cove Petaluma
- The Report was prepared by ConeTec for ENGEO, Inc.

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

Client Disclaimer

- ConeTec was retained by ENGEO, Inc.
- The "Report" refers to this report titled Oyster Cove Petaluma
- ConeTec was retained to collect and provide the raw data ("Data") which is included in the Report.

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

CONTENTS

The following listed below are included in the report:

SITE MAP

ConeTec Job Number: 24-56-27037

Client: ENGEO, Inc.

Project: Oyster Cove Petaluma

Report Date: 2024-01-09

Sounding Location

All sounding locations are approximate

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

ConeTec

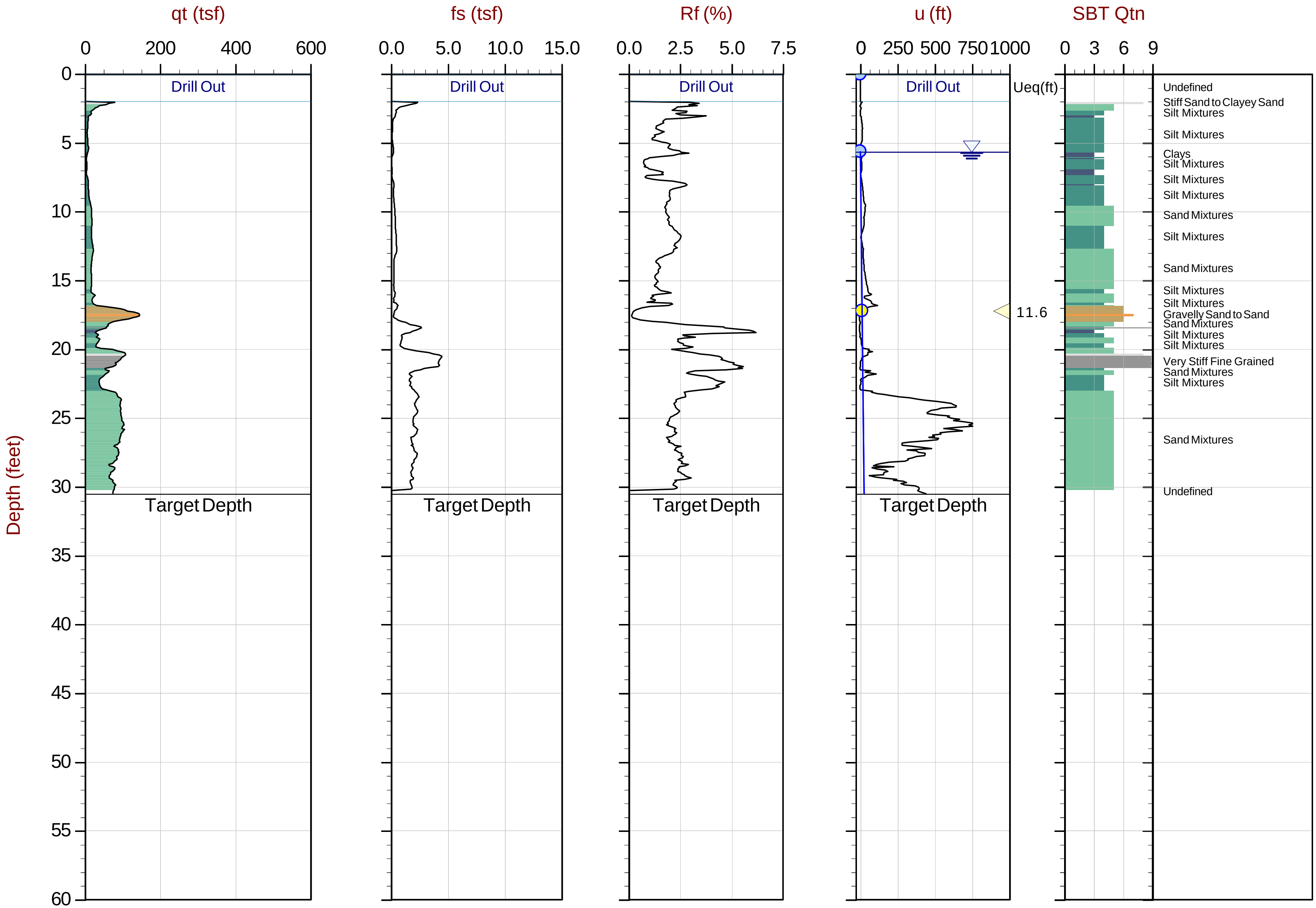
Job No: 24-56-27037
Client: ENGEO, Inc.
Project: Oyster Cove Petaluma
Start Date: 2024-01-04
End Date: 2024-01-04

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Rig	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Shear Wave Velocity Tests	Northing[2] (m)	Easting[2] (m)	Surface Elevation[3] (ft)	Refer to Notation Number
2-CPT-2	24-56-27037_CP02	2024-01-04	C-15	817:T1500F15U35	15	5.6	30.51		4231992	531961	19	
2-SCPT-3	24-56-27037_SP03	2024-01-04	C-15	817:T1500F15U35	15	5.6	55.45	16	4232018	531950	19	4
2-SCPT-4	24-56-27037_SP04	2024-01-04	C-15	817:T1500F15U35	15	13.0	35.84	11	4231896	531992	20	4
2-CPT-5	24-56-27037_CP05	2024-01-04	C-15	817:T1500F15U35	15	10.4	50.52		4231929	532090	21	4
Totals	4 Soundings						172.32	27				

1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using a consumer grade GPS receiver. EPSG number: 32610 (WGS84 / UTM Zone 10).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. Assumed phreatic surface is based on the dynamic pore pressure profile.

CONETEC ENGEO, Inc.	Job No: 24-56-27037	Sounding: 2-CPT-2
	Date: 2024-01-04 16:27	Cone: 817:T1500F15U35
	Site: Oyster Cove Petaluma	



Max Depth: 9.300 m / 30.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231992m E: 531961m

Equilibrium Pore Pressure (Ueq) — Assumed Ueq — Dissipation, Ueq achieved — Dissipation, Ueq not achieved — Hydrostatic Line

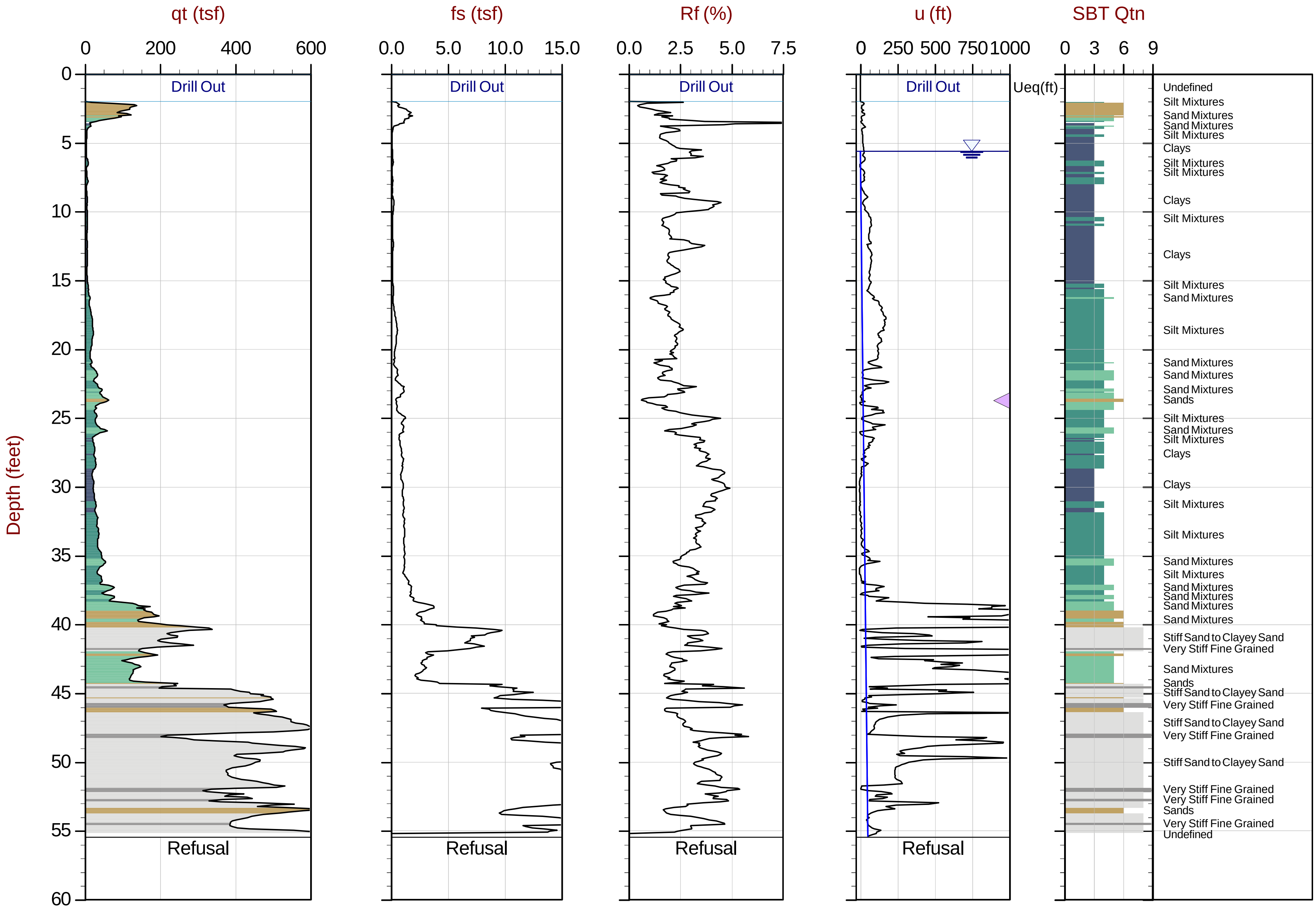
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35



Max Depth: 16.900 m / 55.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4232018m E: 531950m

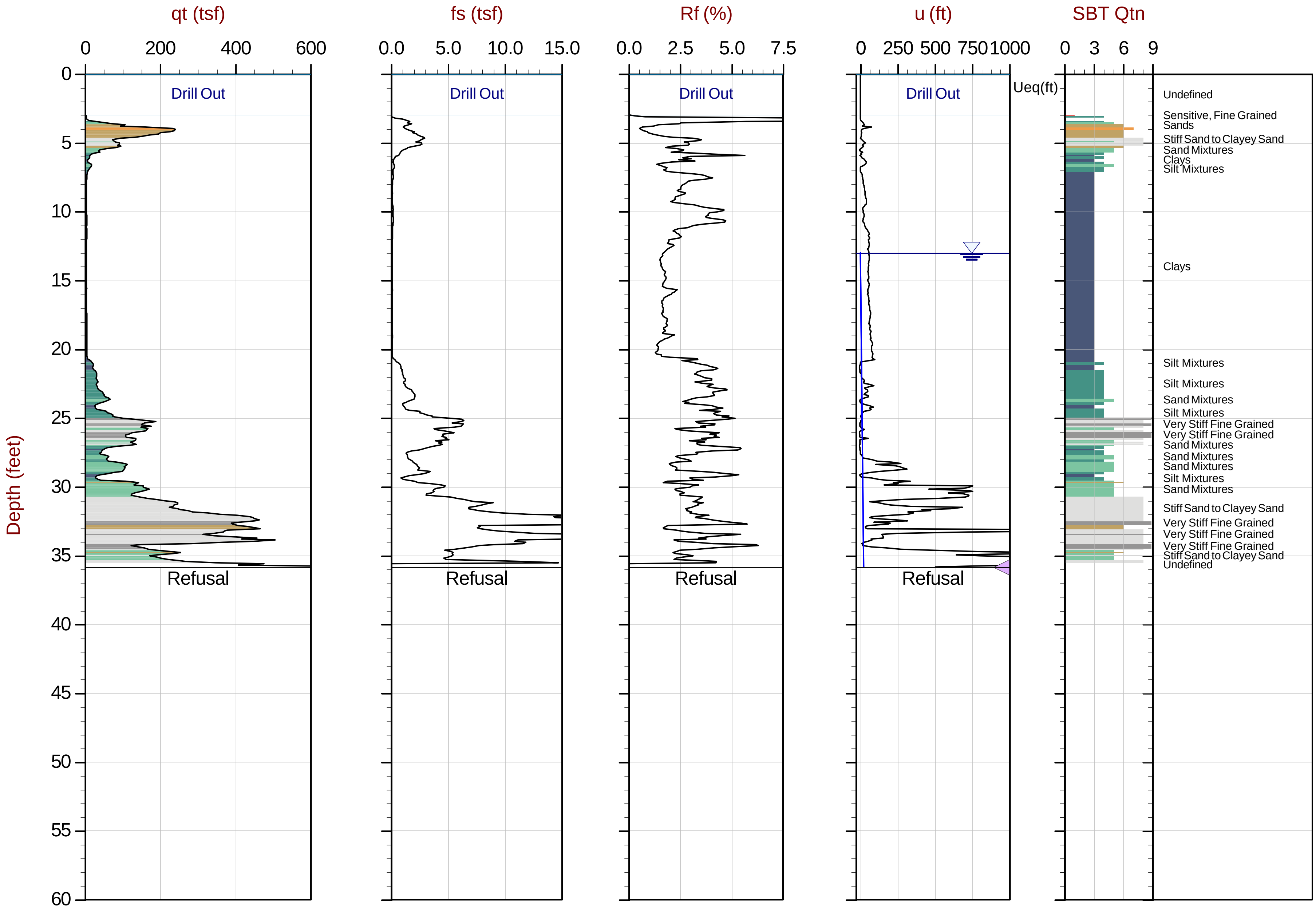
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 09:11
Site: Oyster Cove Petaluma
Sounding: 2-SCPT-4
Cone: 817:T1500F15U35



Max Depth: 10.925 m / 35.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231896m E: 531992m

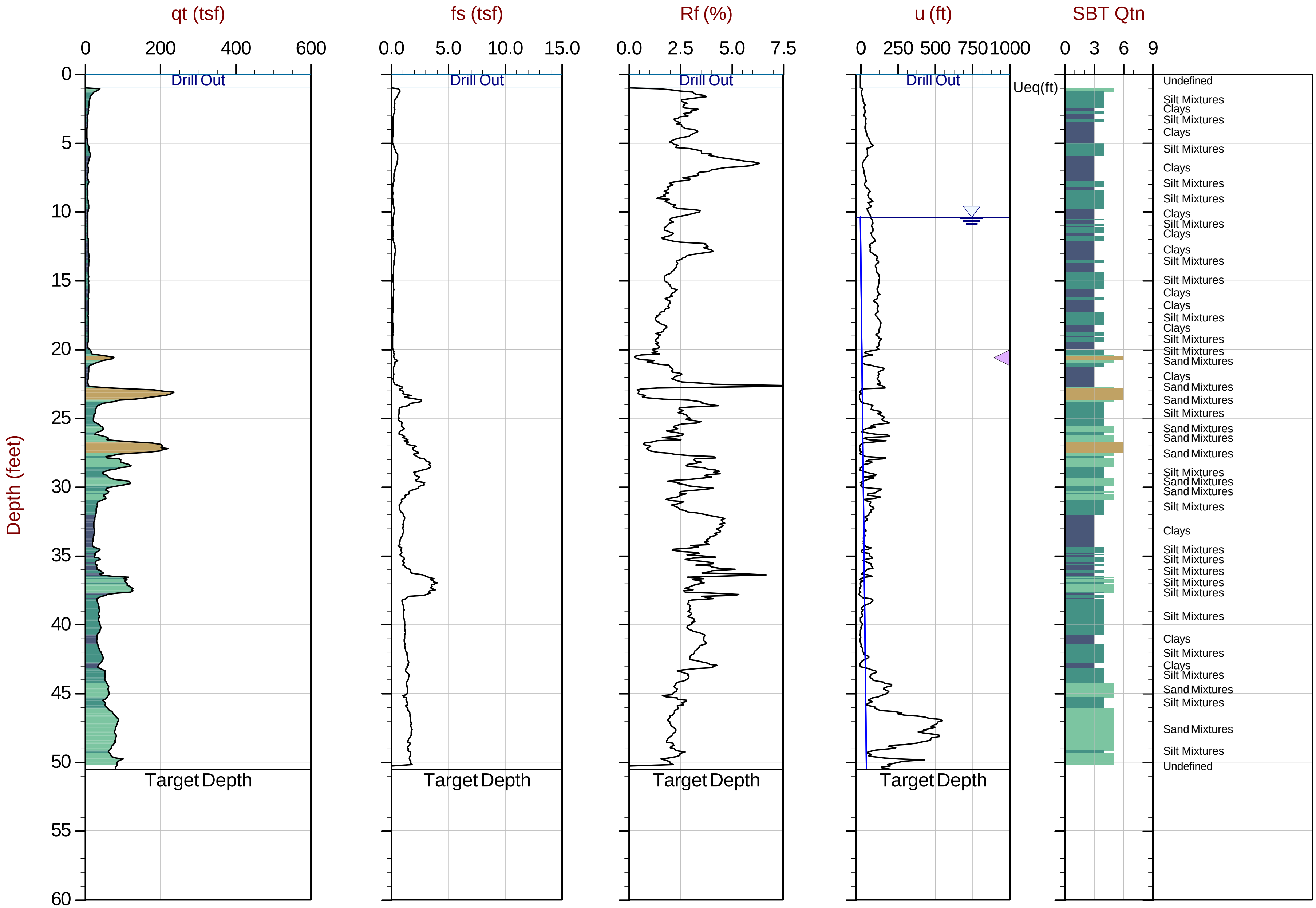
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 11:06
Site: Oyster Cove Petaluma
Sounding: 2-CPT-5
Cone: 817:T1500F15U35



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231929m E: 532090m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

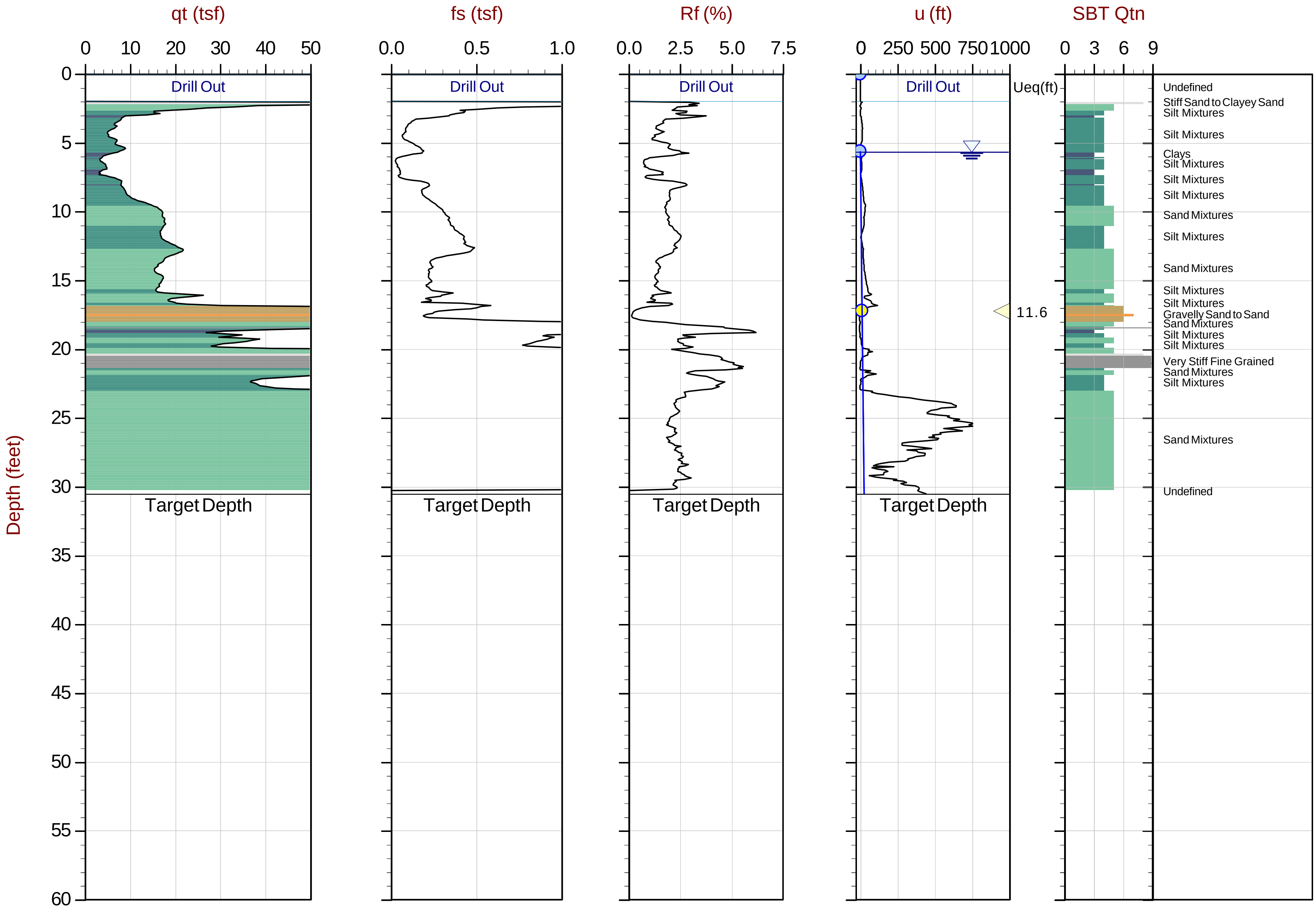
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Small Scaled Penetration Test Plots

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 16:27
Site: Oyster Cove Petaluma

Sounding: 2-CPT-2
Cone: 817:T1500F15U35



Max Depth: 9.300 m / 30.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231992m E: 531961m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

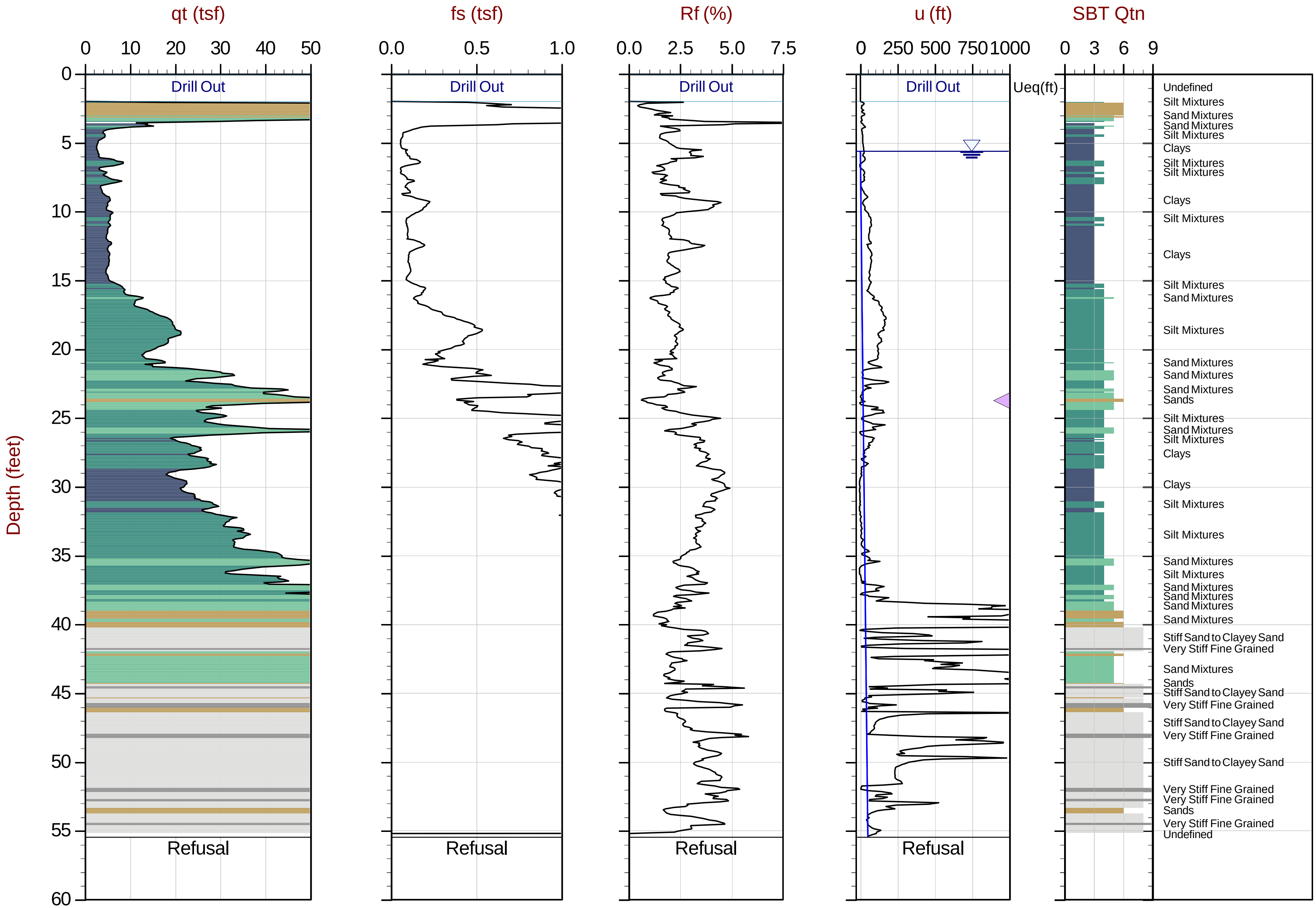
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35



Max Depth: 16.900 m / 55.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4232018m E: 531950m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

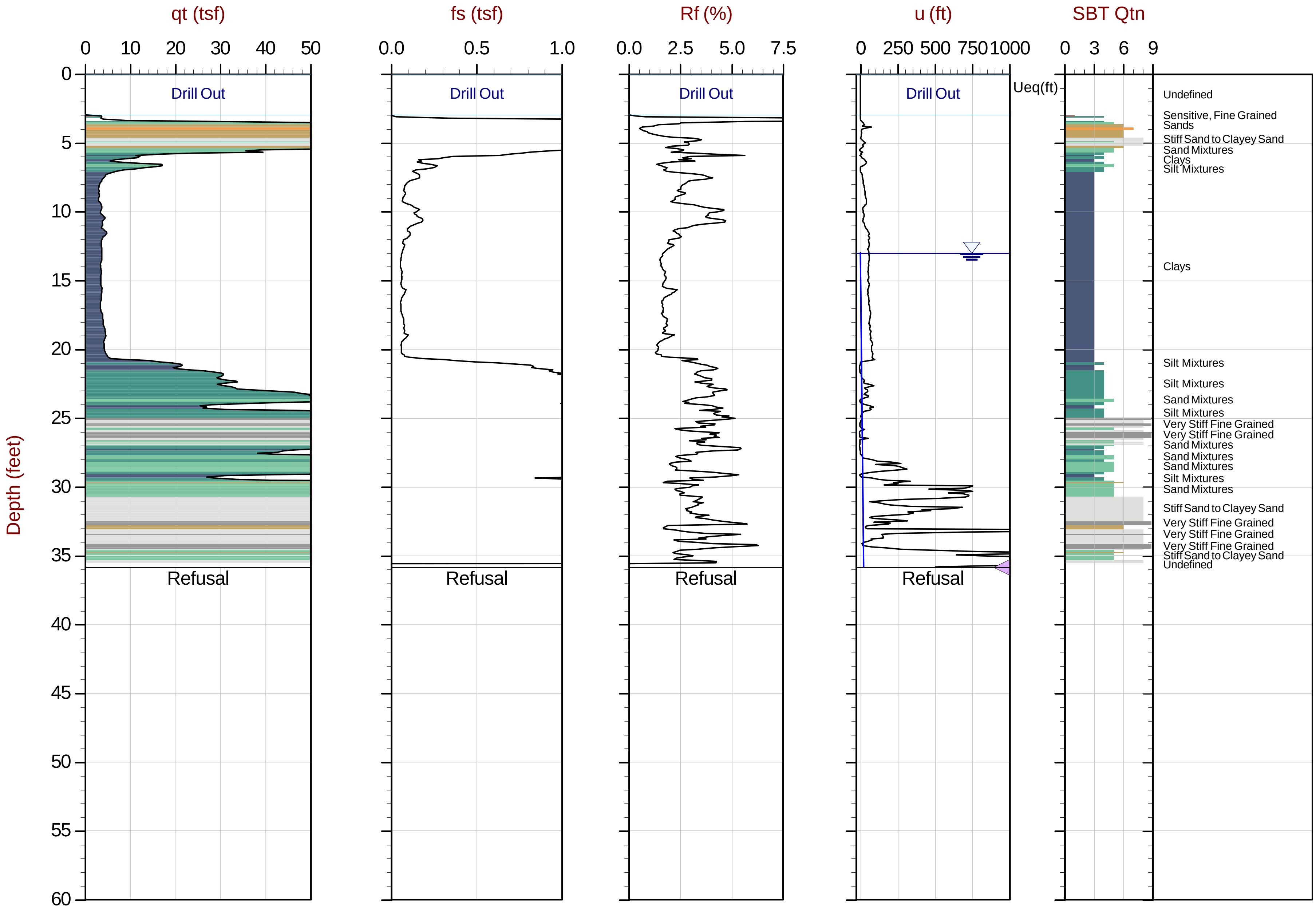
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 09:11
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-4
Cone: 817:T1500F15U35



Max Depth: 10.925 m / 35.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231896m E: 531992m

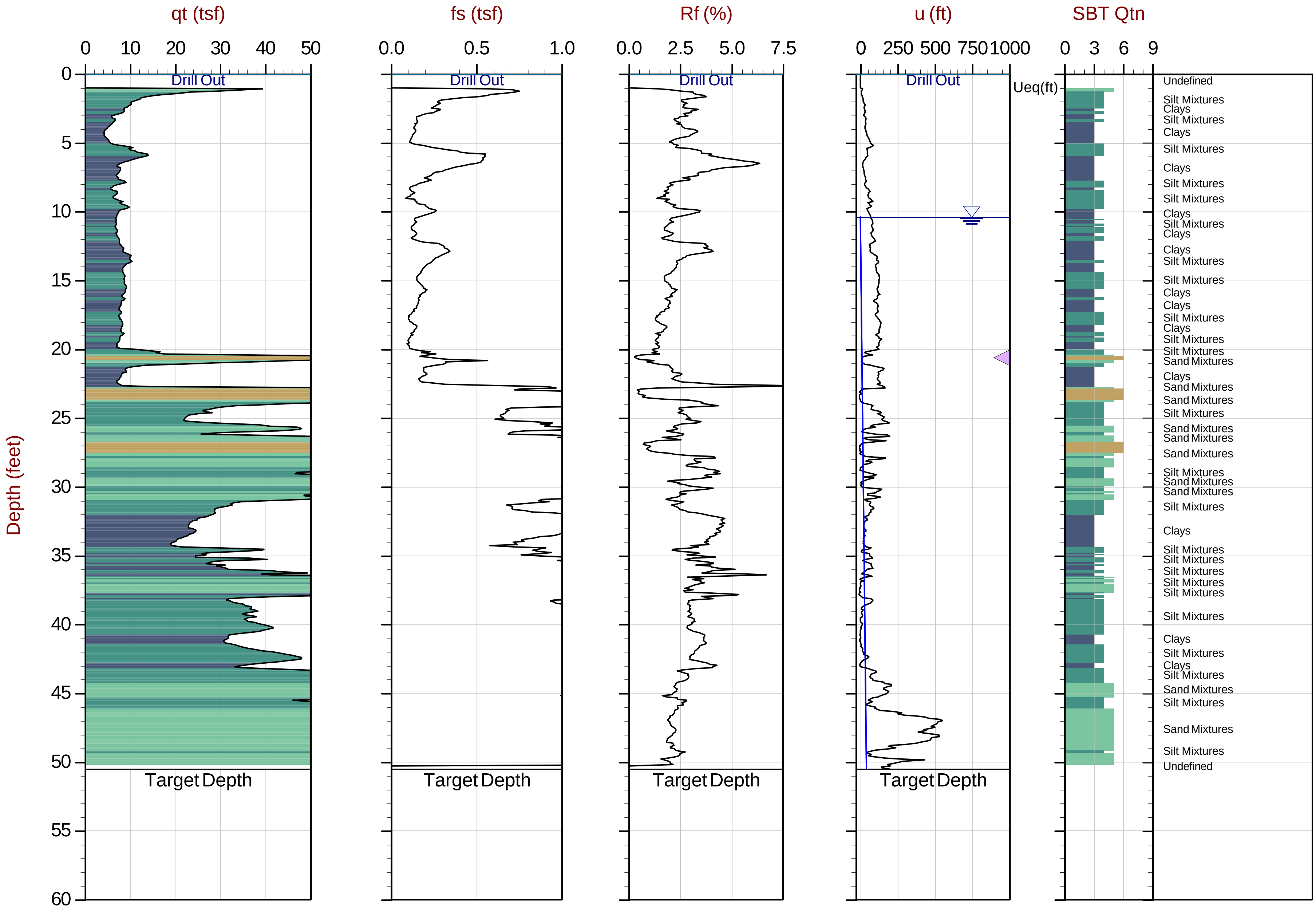
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 11:06
Site: Oyster Cove Petaluma

Sounding: 2-CPT-5
Cone: 817:T1500F15U35



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231929m E: 532090m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

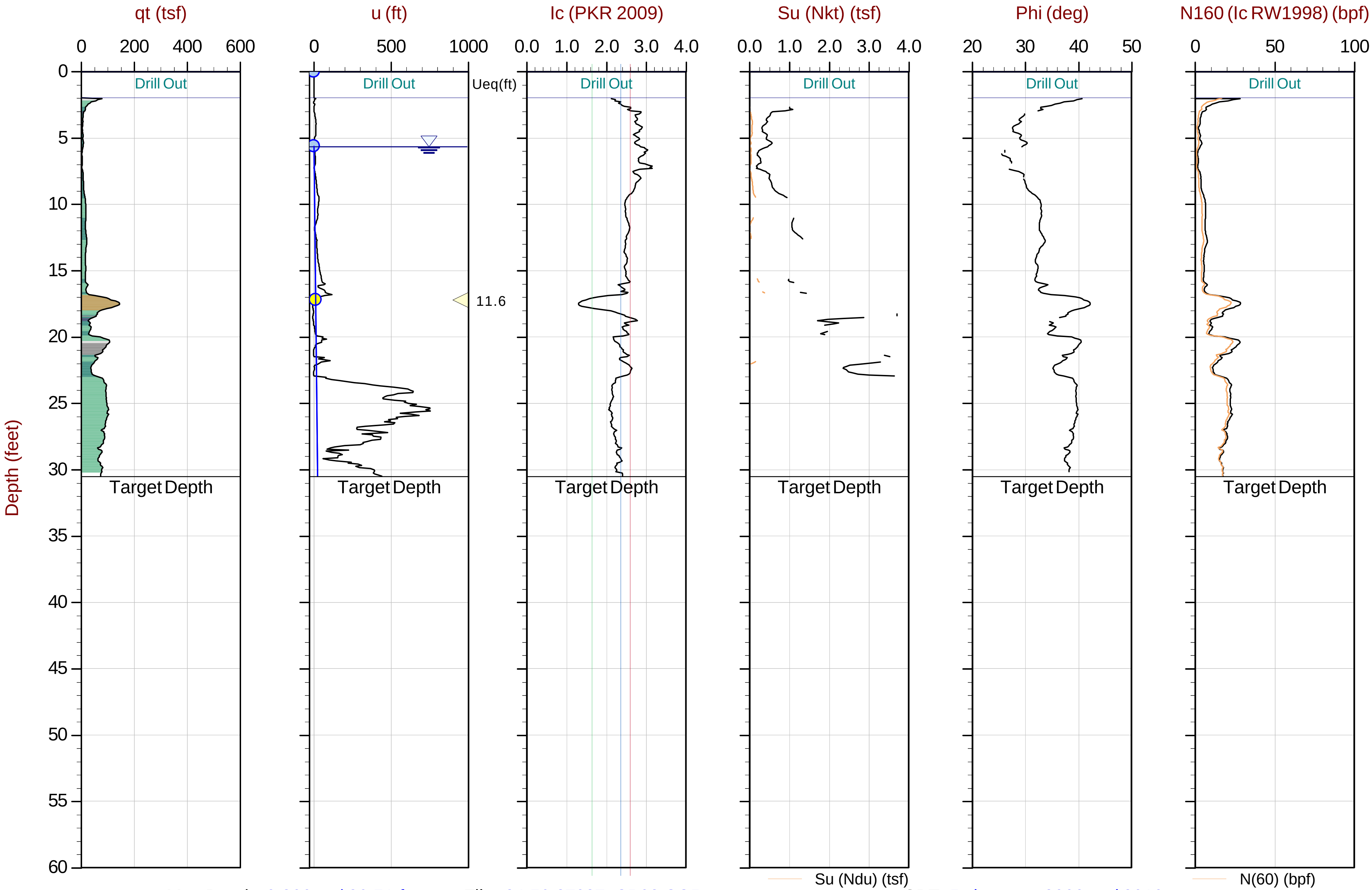
Advanced Cone Penetration Test Plots

ConeTec

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 16:27
Site: Oyster Cove Petaluma
Sounding: 2-CPT-2
Cone: 817:T1500F15U35



Max Depth: 9.300 m / 30.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231992m E: 531961m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

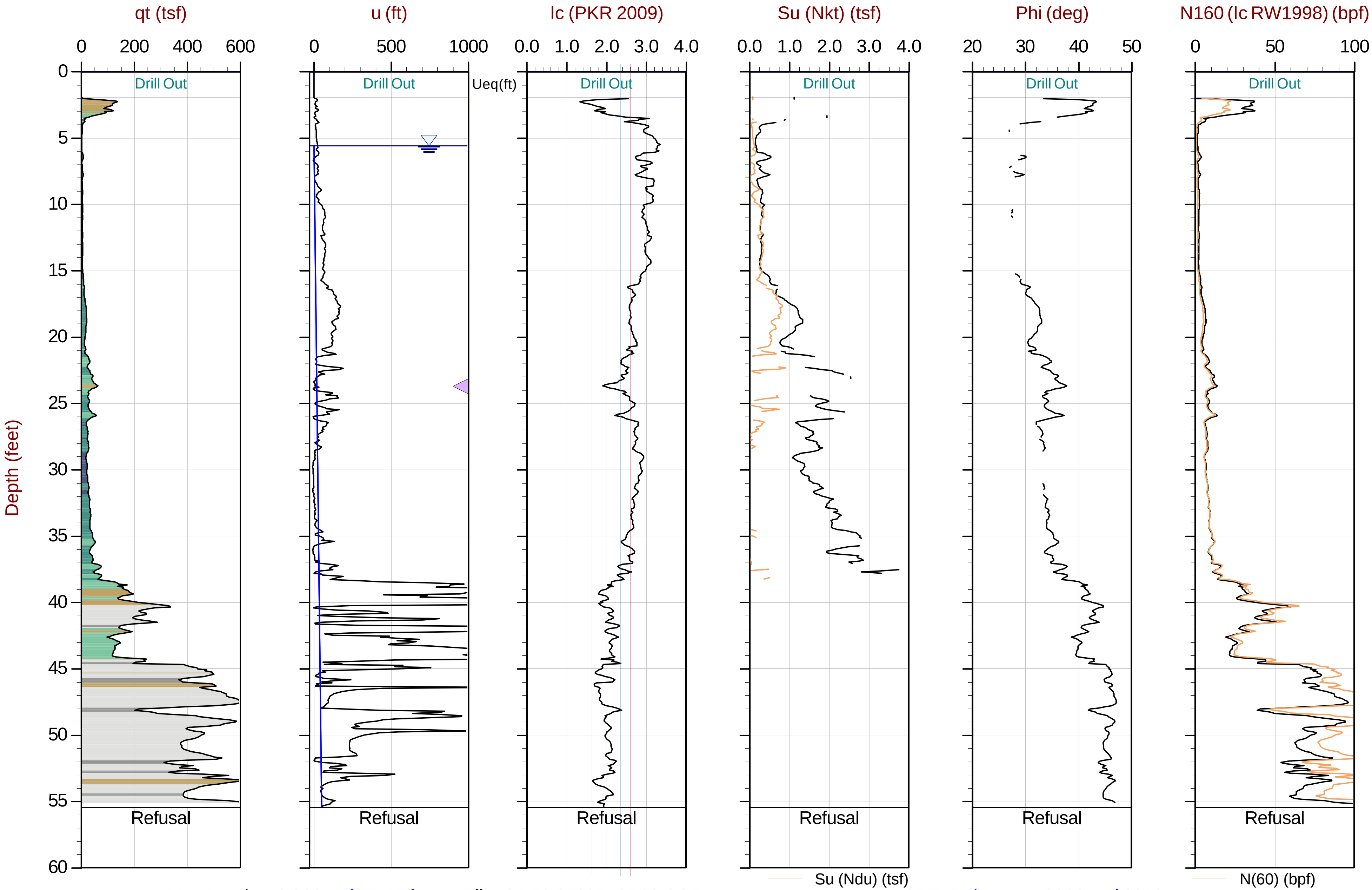
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35



Max Depth: 16.900 m / 55.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4232018m E: 531950m

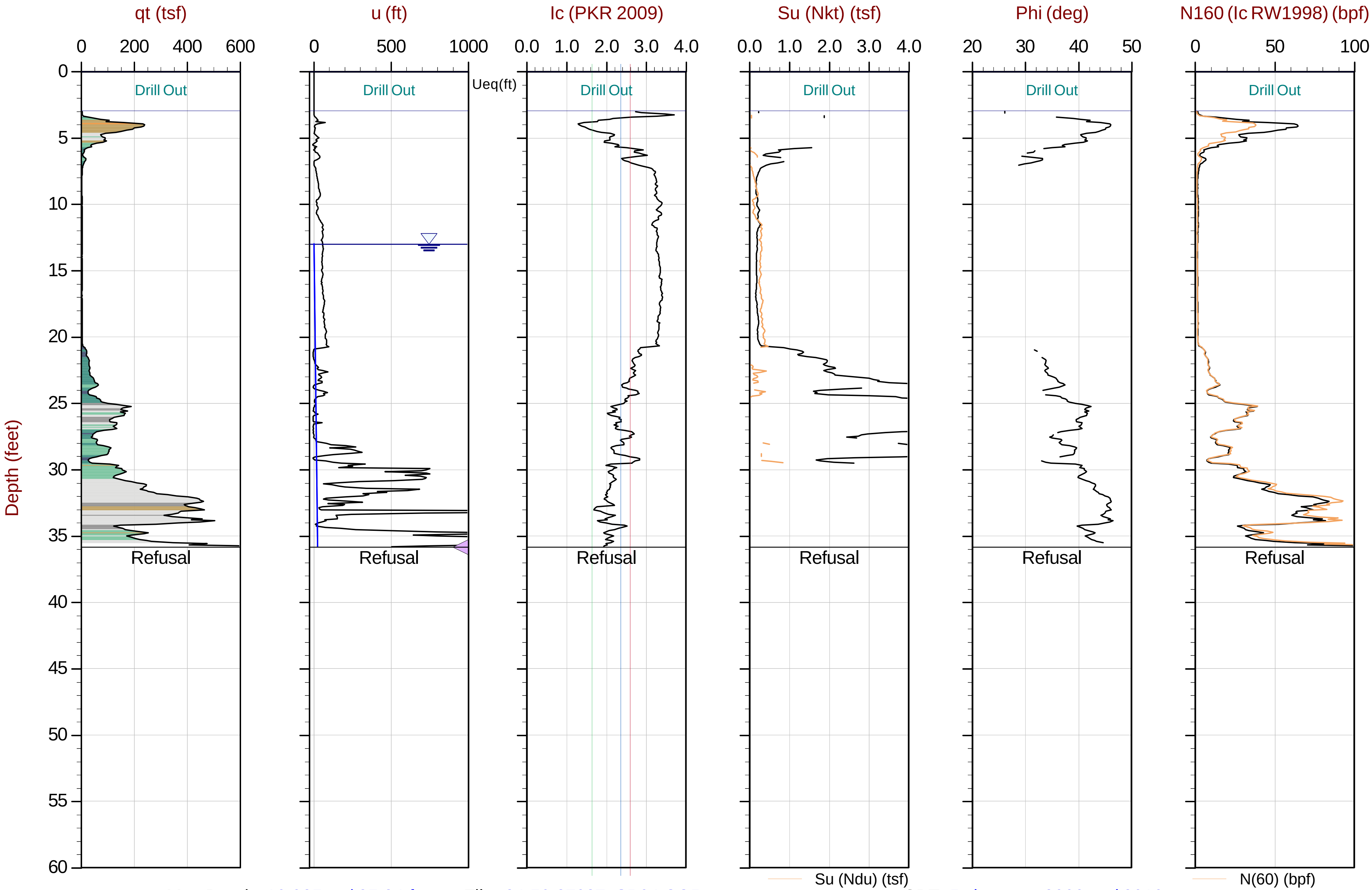
Equilibrium Pore Pressure (Ueq) — Assumed Ueq — Dissipation, Ueq achieved — Dissipation, Ueq not achieved — Hydrostatic Line

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

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 09:11
Site: Oyster Cove Petaluma
Sounding: 2-SCPT-4
Cone: 817:T1500F15U35



Max Depth: 10.925 m / 35.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231896m E: 531992m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

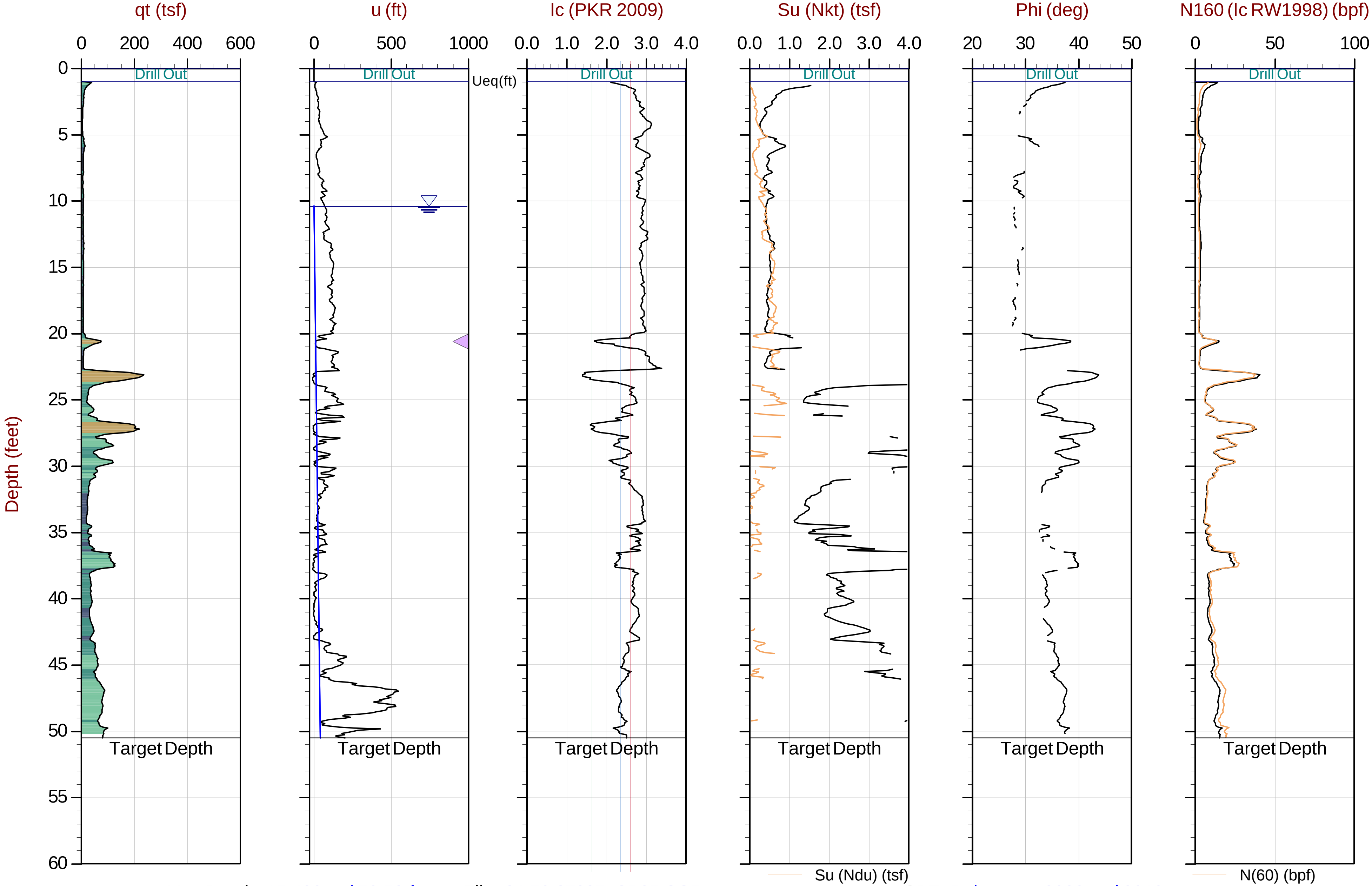
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 11:06
Site: Oyster Cove Petaluma

Sounding: 2-CPT-5
Cone: 817:T1500F15U35



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

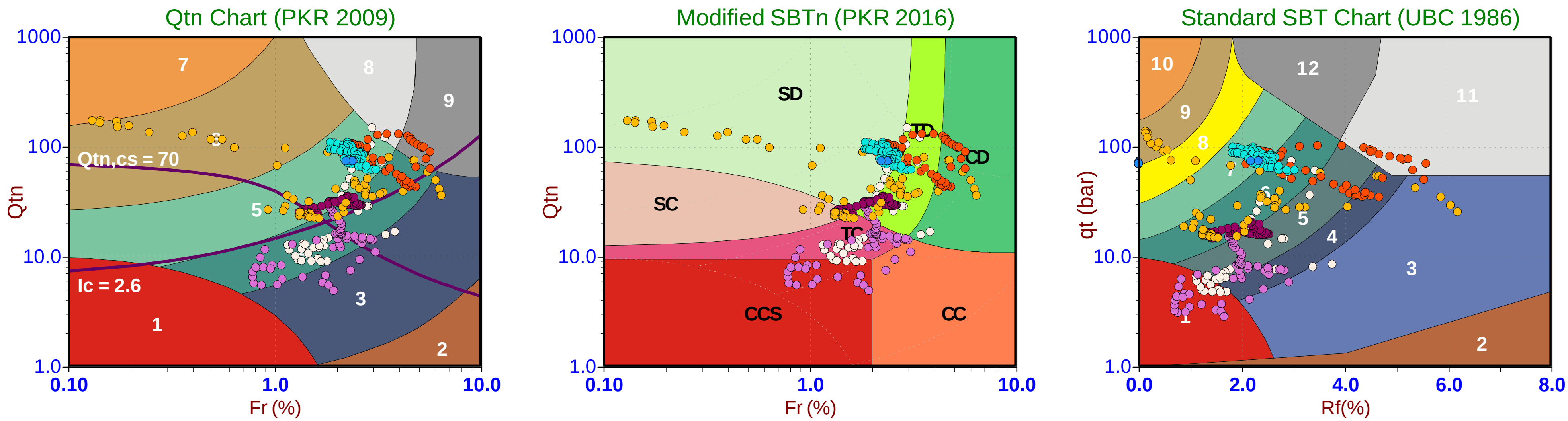
File: 24-56-27037_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231929m E: 532090m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Soil Behavior Type (SBT) Scatter Plots



Depth Ranges

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

Legend

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

Legend

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

Legend

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

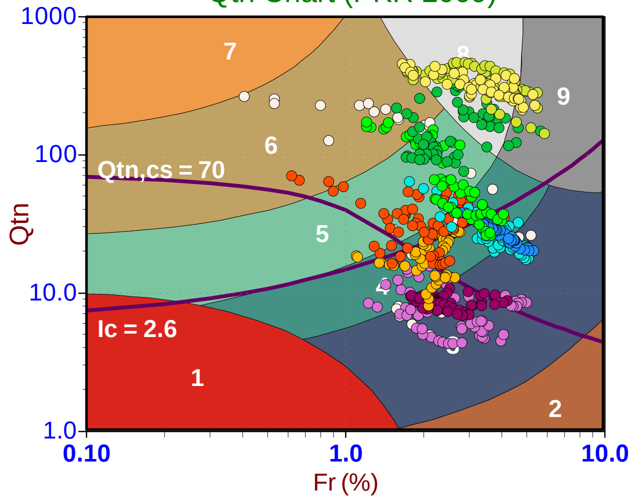
CONETEC

ENGEO, Inc.

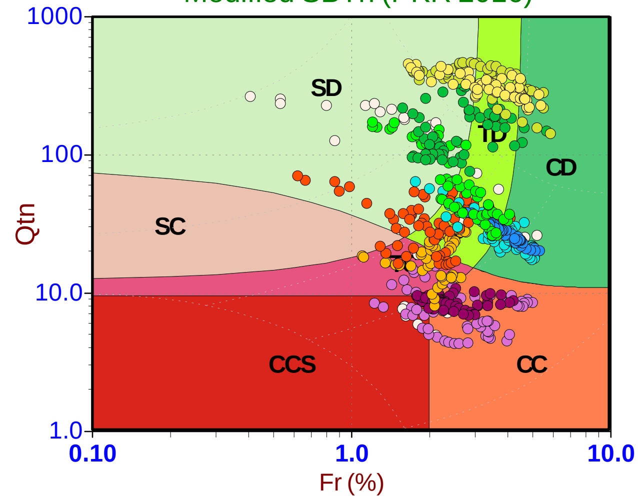
Job No: 24-56-27037
Date: 2024-01-04 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35

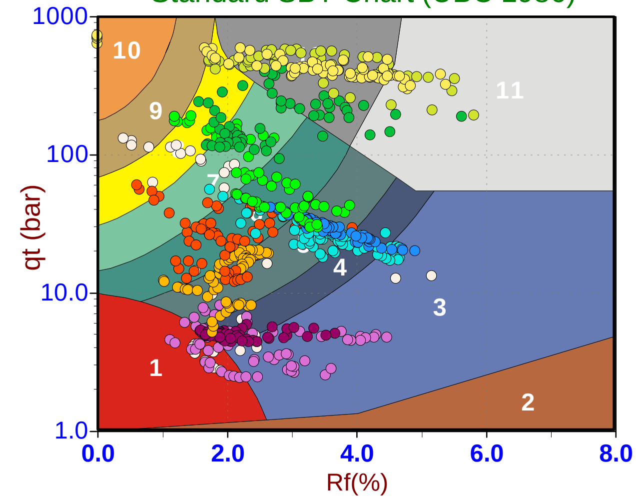
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)

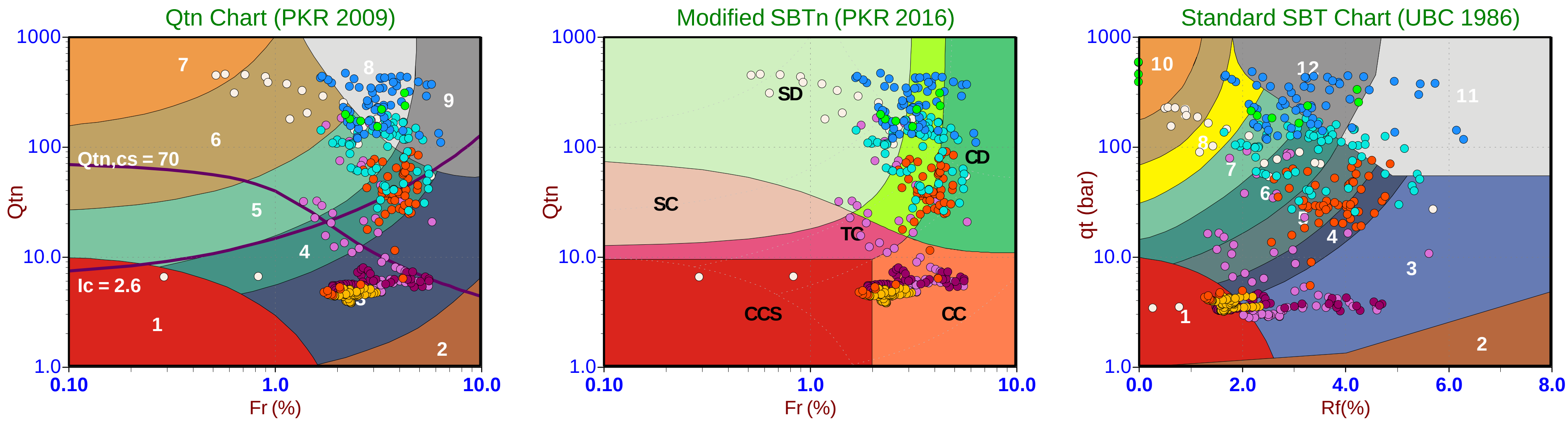


Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
- >25.0 to 30.0 ft
- >30.0 to 35.0 ft
- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

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

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

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



Depth Ranges
- >0.0 to 5.0 ft
- >5.0 to 10.0 ft
- >10.0 to 15.0 ft
- >15.0 to 20.0 ft
- >20.0 to 25.0 ft
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Legend
- Sensitive, Fine Grained
- Organic Soils
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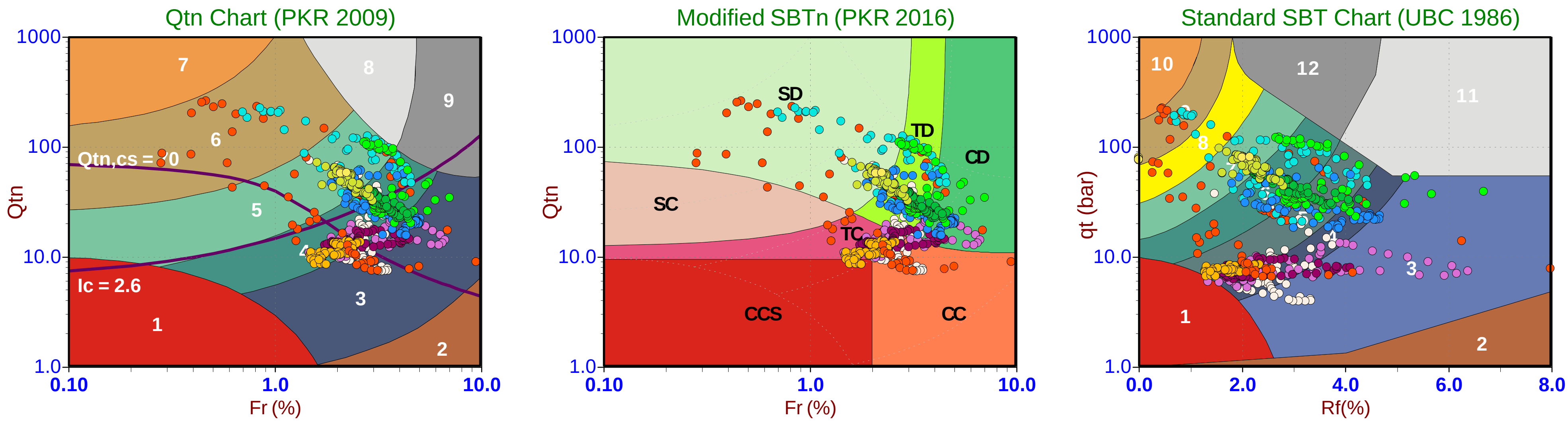
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 11:06
Site: Oyster Cove Petaluma

Sounding: 2-CPT-5
Cone: 817:T1500F15U35



Depth Ranges

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

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
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- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
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- CD (Dil. clay like)
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- 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:	24-56-27037
	Client:	ENGEO, Inc.
	Project:	Oyster Cove Petaluma
	Start Date:	2024-01-04
	End Date:	2024-01-04

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)	Refer to Notation Number
2-CPT-2	24-56-27037_CP02	15	300	17.2	11.6	5.6	
2-SCPT-3	24-56-27037_SP03	15	405	23.7			1
2-SCPT-4	24-56-27037_SP04	15	315	35.8			1
2-CPT-5	24-56-27037_CP05	15	525	20.6			1
Total:			**25.8 Mins**				

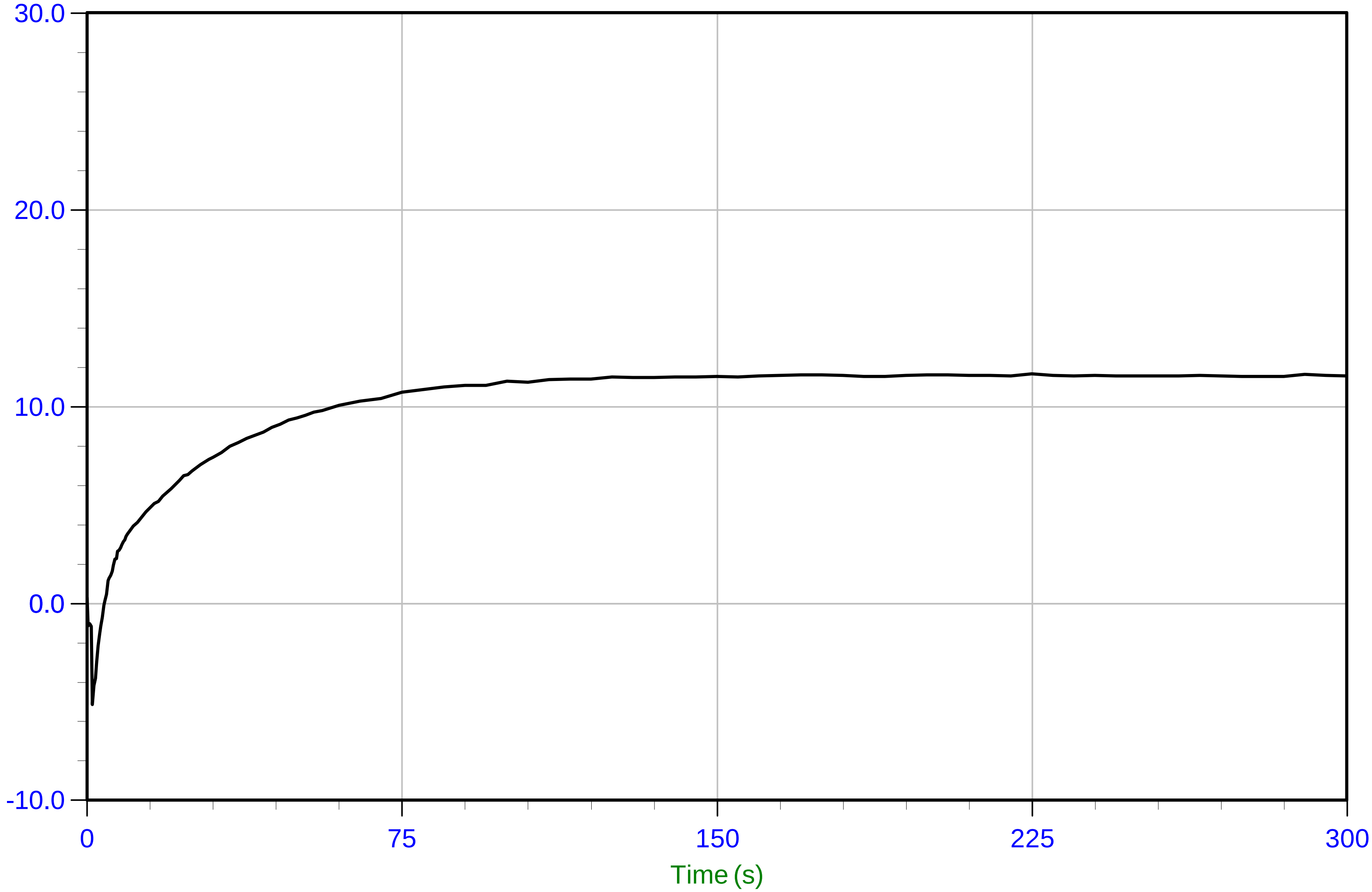
1. Equilibrium pore pressure not achieved.

CONETEC

ENGEO, Inc.

Job No: 24-56-27037
Date: 01/04/2024 16:27
Site: Oyster Cove Petaluma

Sounding: 2-CPT-2
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-27037_CP02.ppd2
Depth: 5.250 m / 17.224 ft
Duration: 300.0 s

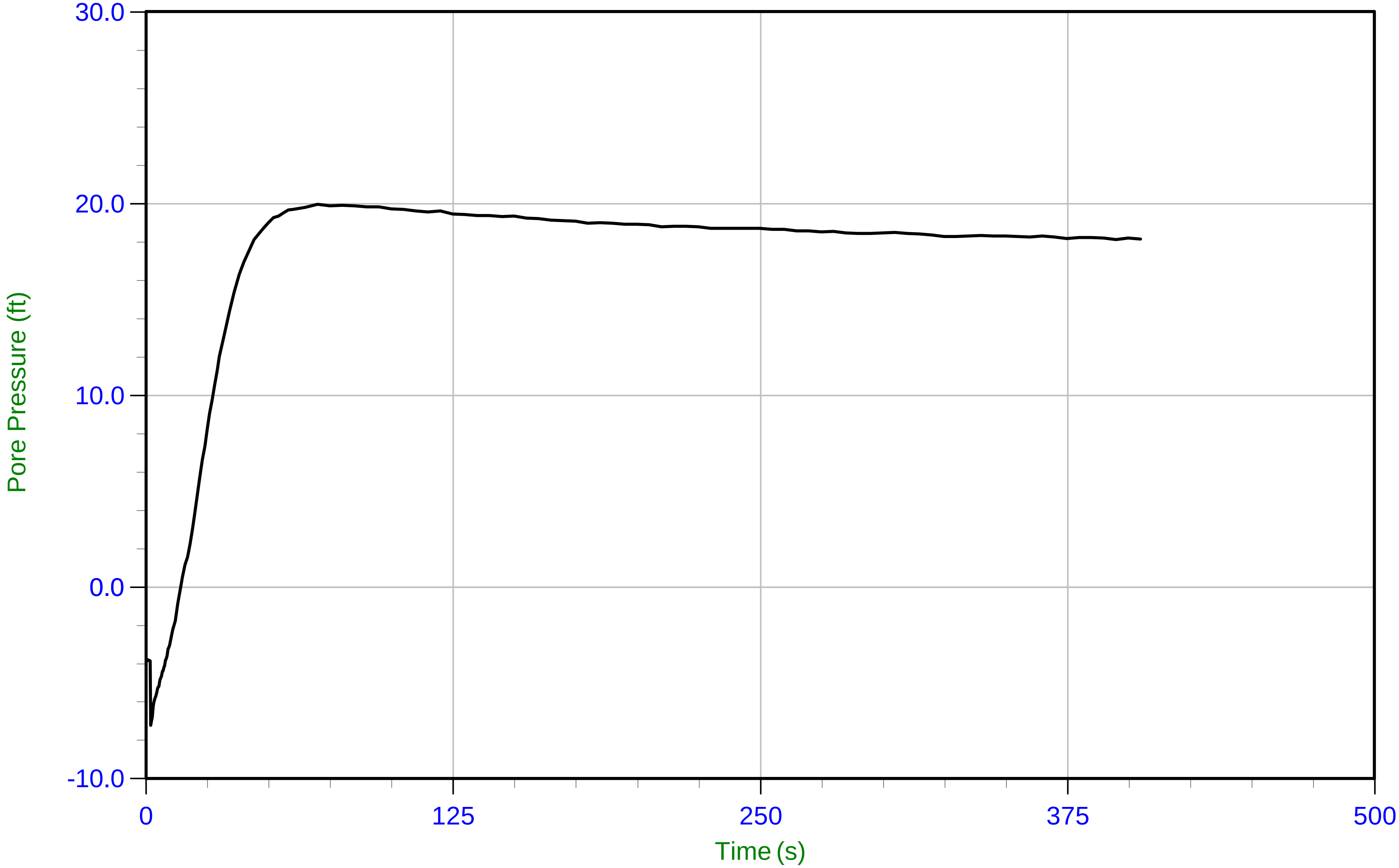
u Min: -5.1 ft
u Max: 11.7 ft
u Final: 11.6 ft

WT: 1.721 m / 5.645 ft
Ueq: 11.6 ft

ENGEO, Inc.

Job No: 24-56-27037
Date: 01/04/2024 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

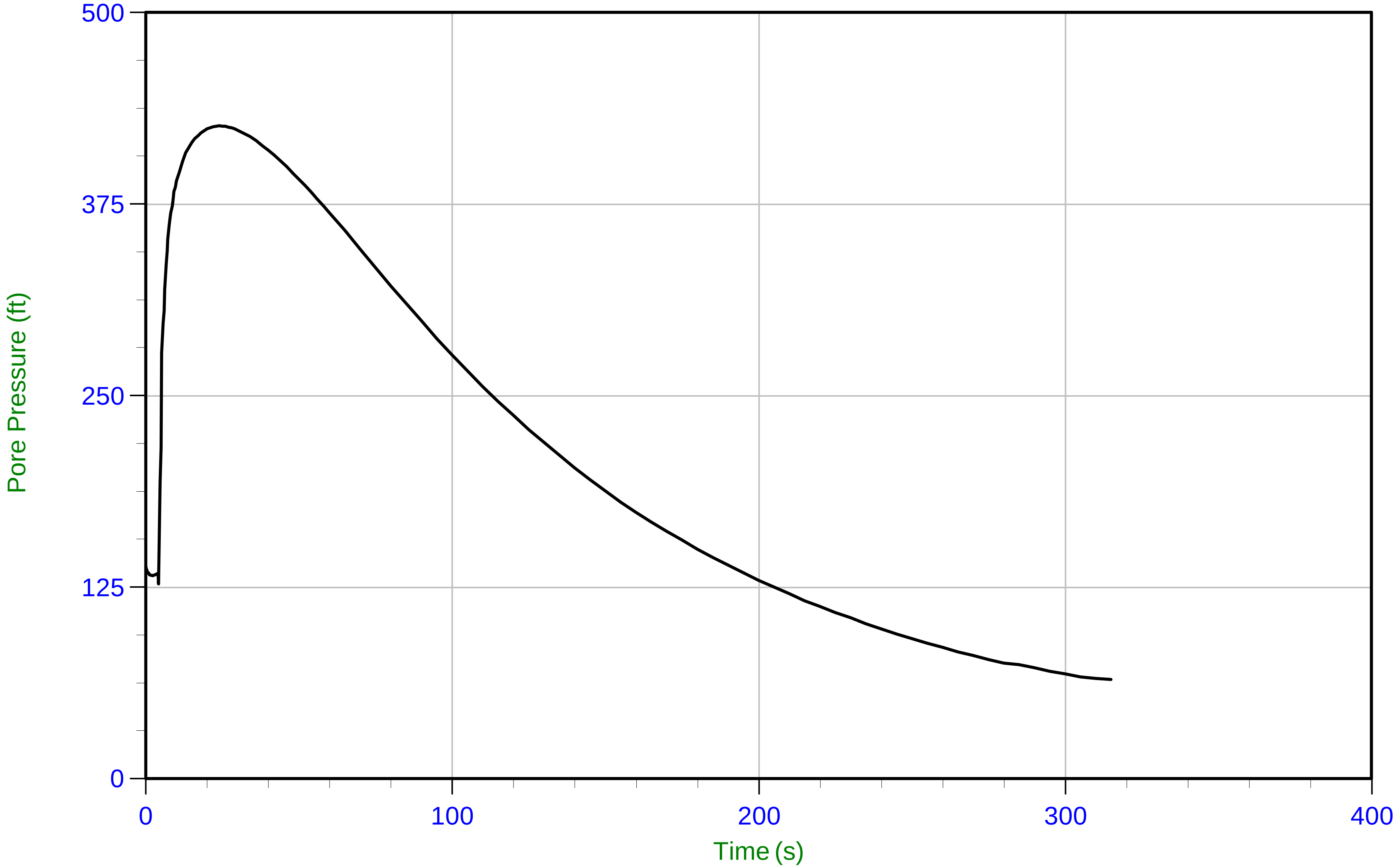
Filename: 24-56-27037_SP03.ppd2
Depth: 7.225 m / 23.704 ft
Duration: 405.0 s

u Min: -7.2 ft
u Max: 20.0 ft
u Final: 18.1 ft

ENGEO, Inc.

Job No: 24-56-27037
Date: 01/04/2024 09:11
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-4
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-27037_SP04.ppd2
Depth: 10.925 m / 35.843 ft
Duration: 315.0 s

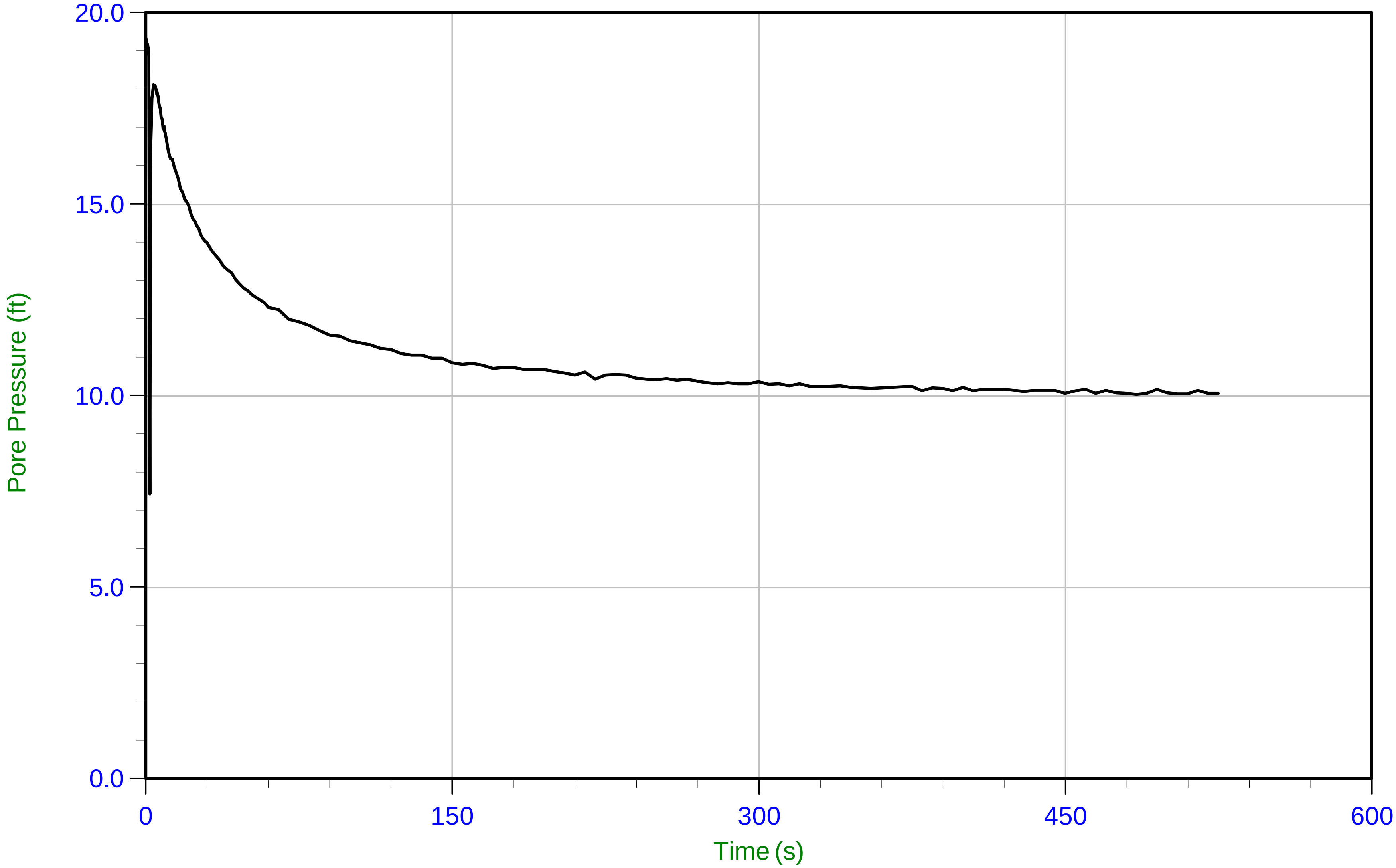
u Min: 64.8 ft
u Max: 426.0 ft
u Final: 64.8 ft

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 01/04/2024 11:06
Site: Oyster Cove Petaluma

Sounding: 2-CPT-5
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-27037_CP05.ppd2
Depth: 6.275 m / 20.587 ft
Duration: 525.0 s

u Min: 7.4 ft
u Max: 19.3 ft
u Final: 10.1 ft

Seismic Cone Penetration Test Tabular Results

Job No: 24-56-27037
Client: ENGEO, Inc.
Project: Oyster Cove Petaluma
Sounding ID: 2-SCPT-3
Date: 4-Jan-2024

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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
9.12	8.46	8.69			
12.30	11.65	11.81	3.12	10.85	288
15.58	14.93	15.06	3.24	12.42	261
18.77	18.11	18.22	3.16	7.49	422
22.15	21.49	21.58	3.36	4.98	676
25.43	24.77	24.85	3.27	4.75	688
28.81	28.15	28.22	3.37	5.10	661
32.09	31.43	31.49	3.27	4.84	676
35.37	34.71	34.77	3.28	4.13	792
38.65	37.99	38.04	3.28	3.02	1085
41.93	41.27	41.32	3.28	2.65	1238
45.21	44.55	44.60	3.28	2.37	1385
48.33	47.67	47.71	3.11	1.80	1730
54.89	54.23	54.27	6.56	2.66	2465

CONETEC

Job No: 24-56-27037
Client: ENGEO, Inc.
Project: Oyster Cove Petaluma
Sounding ID: 2-SCPT-4
Date: 4-Jan-2024

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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
9.25	8.60	8.82			
12.63	11.97	12.14	3.32	12.95	256
15.91	15.26	15.38	3.25	14.24	228
19.19	18.54	18.64	3.26	13.09	249
22.38	21.72	21.81	3.17	9.67	327
25.75	25.10	25.18	3.37	3.85	874
29.13	28.48	28.55	3.37	2.91	1158
32.41	31.76	31.82	3.27	2.69	1218
35.60	34.94	35.00	3.18	1.88	1689

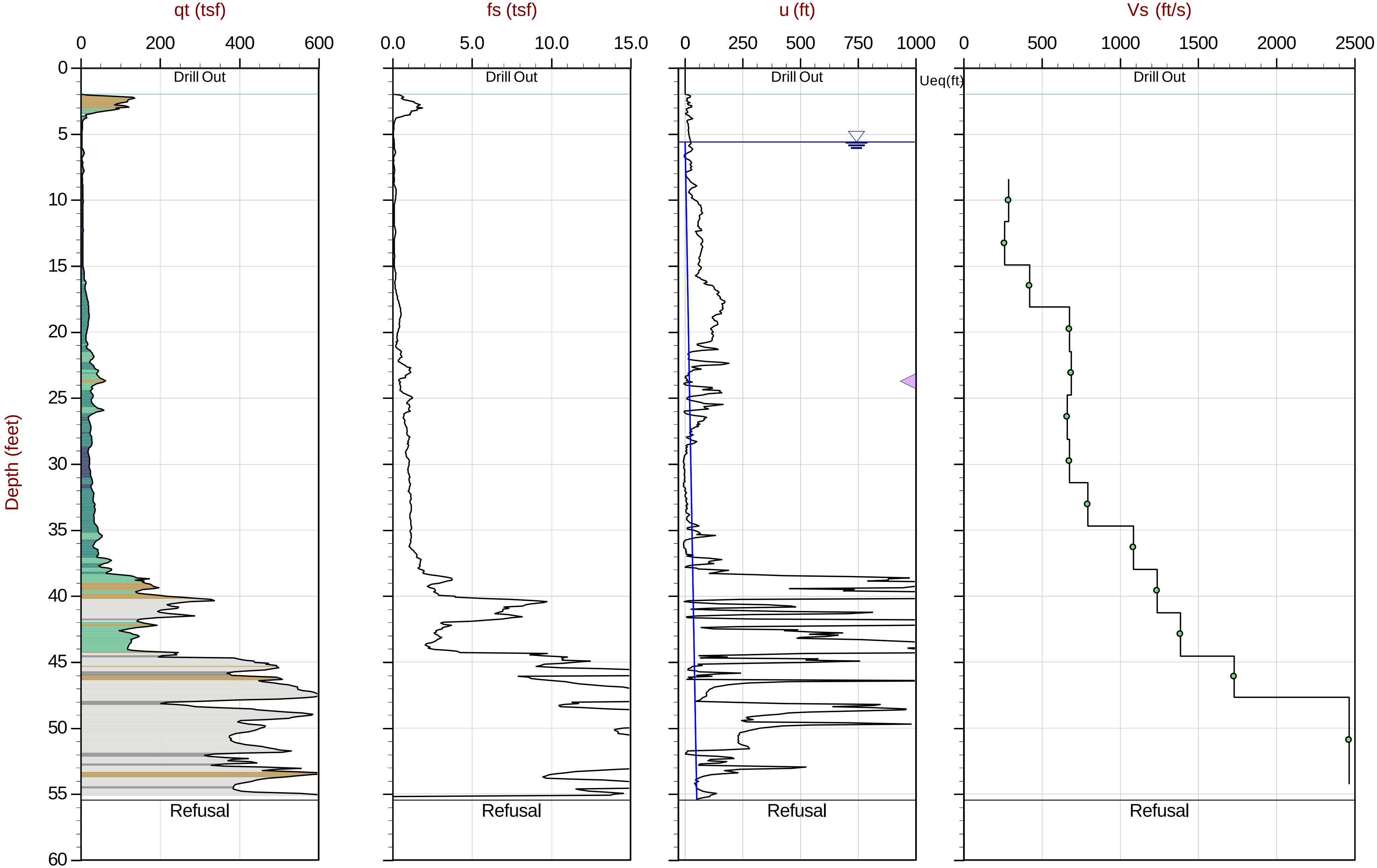
Seismic Cone Penetration Test Plots

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 12:20
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-3
Cone: 817:T1500F15U35



Max Depth: 16.900 m / 55.45 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4232018m E: 531950m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

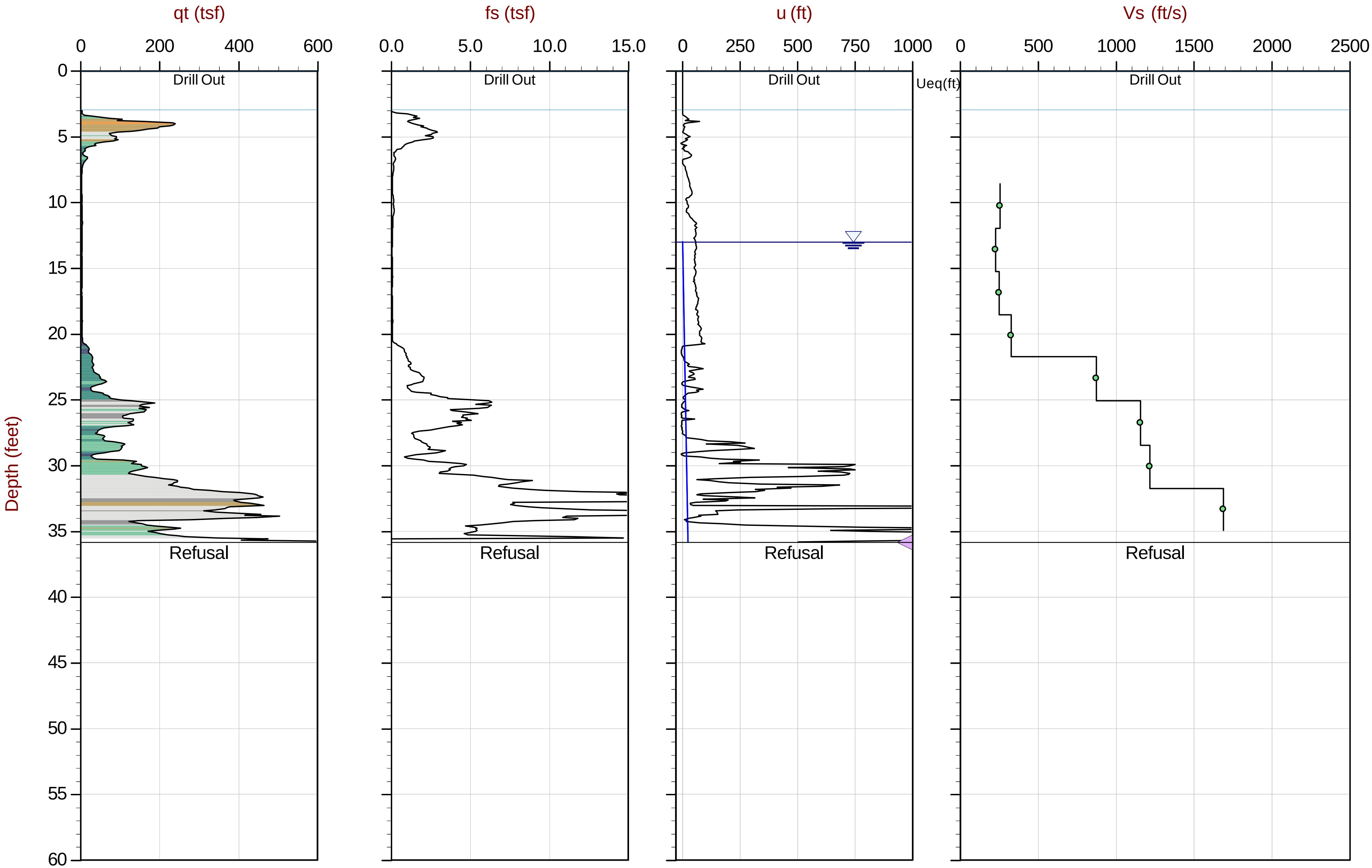
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

ENGEO, Inc.

Job No: 24-56-27037
Date: 2024-01-04 09:11
Site: Oyster Cove Petaluma

Sounding: 2-SCPT-4
Cone: 817:T1500F15U35



Max Depth: 10.925 m / 35.84 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-27037_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM ZONE 10) N: 4231896m E: 531992m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 24-56-27037
Date: 01:04:24 12:20

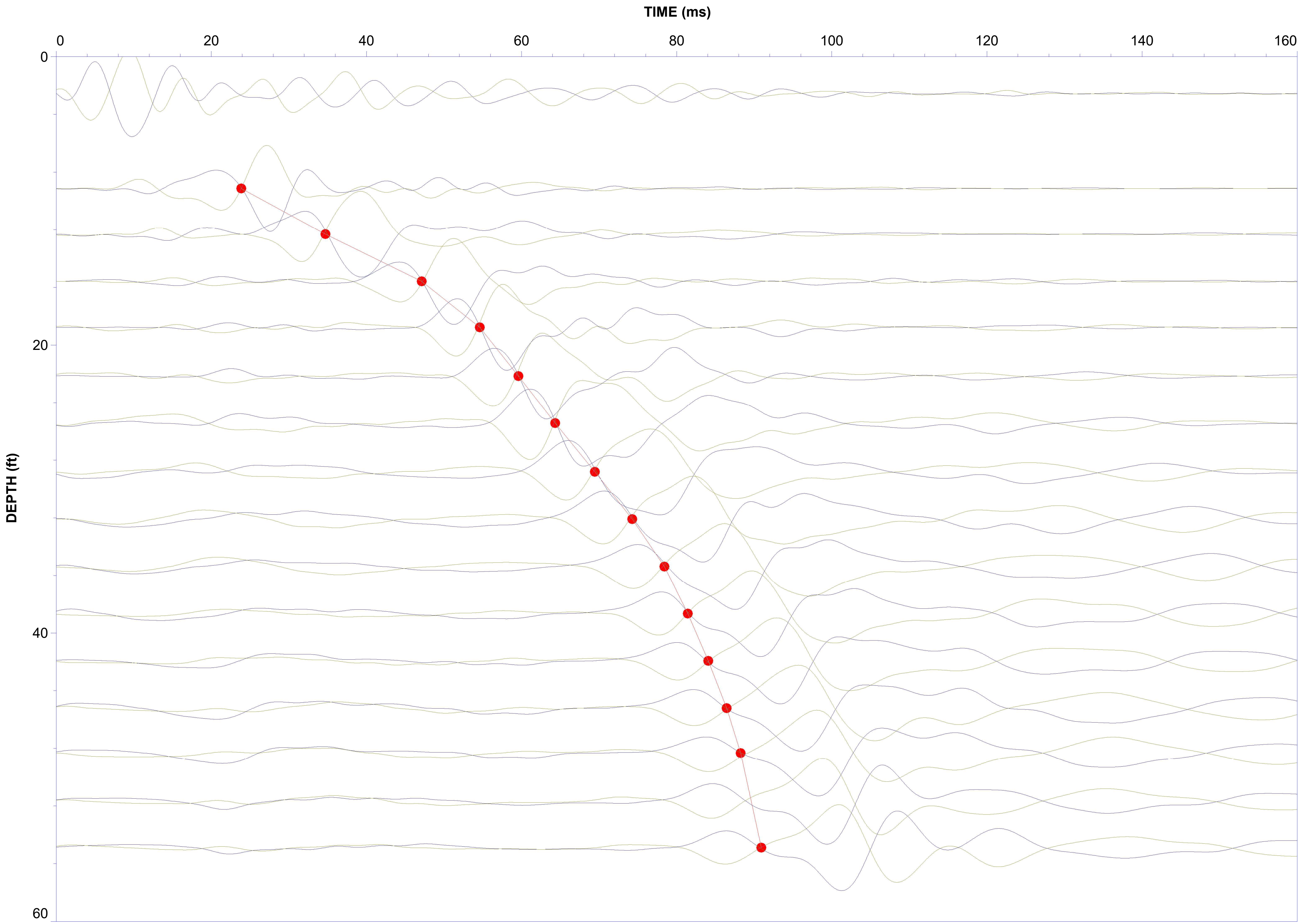
Client: ENGEO, Inc.
Cone: 817:T1500F15U35

Project: Oyster Cove Petaluma

Analysis: Shear Wave

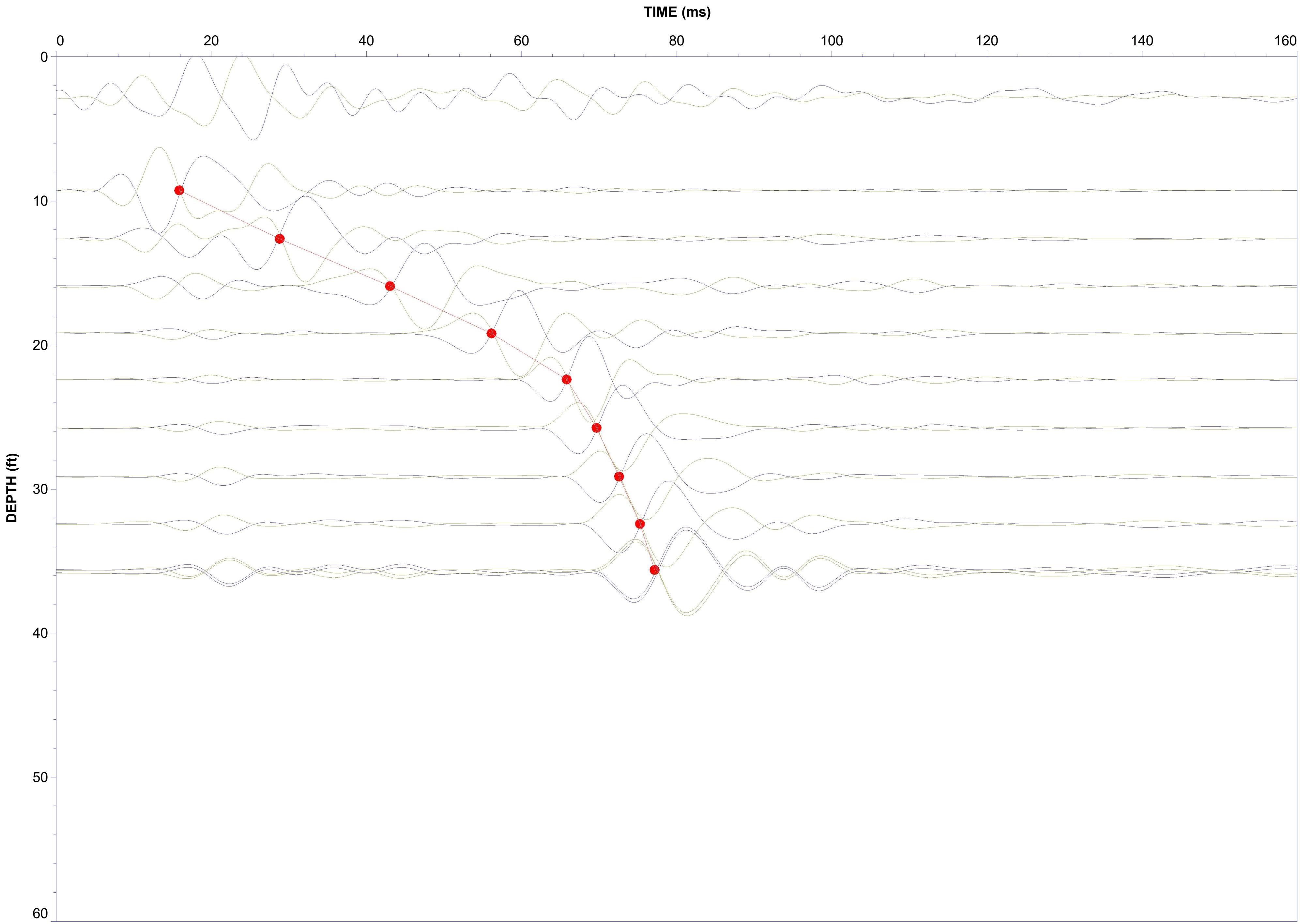
Sounding: 2-SCPT-3

Filter: BP: 0 to 200 Hz



ConeTec

Job No: 24-56-27037
Date: 01:04:24 09:11
Client: ENGEO, Inc.
Cone: 817:T1500F15U35
Project: Oyster Cove Petaluma
Analysis: Shear Wave
Sounding: 2-SCPT-4
Filter: BP: 0 to 200 Hz



Methodology Statements and Data File Formats

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

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

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

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

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

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

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

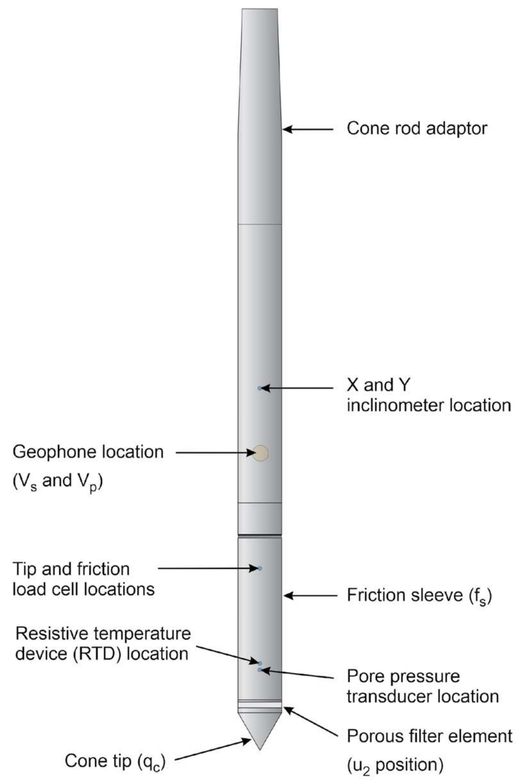


Figure CPTu. Piezocone Penetrometer (15 cm²)

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

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

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

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

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

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

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

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

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

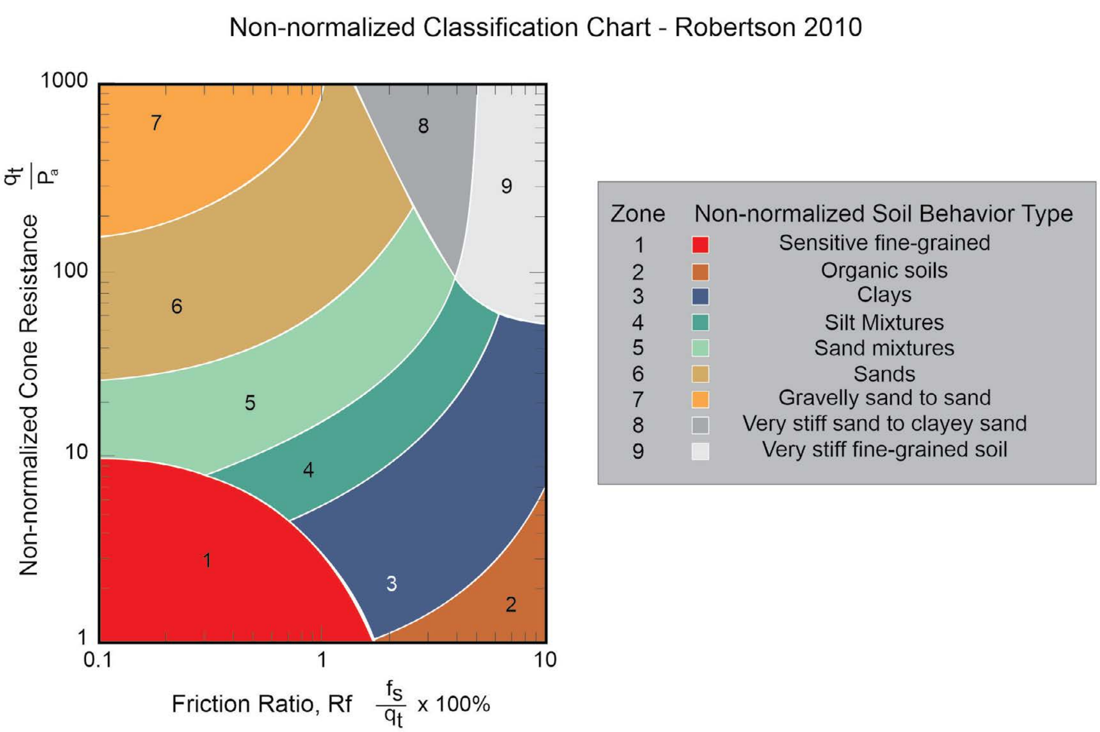


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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

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

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

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

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

REFERENCES

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

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. DOI: 10.1061/9780784412770.027.

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.

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

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., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Robertson, P.K., 2010. Soil behavior type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT’10, Huntington Beach, CA, USA

PORE PRESSURE DISSIPATION TEST

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

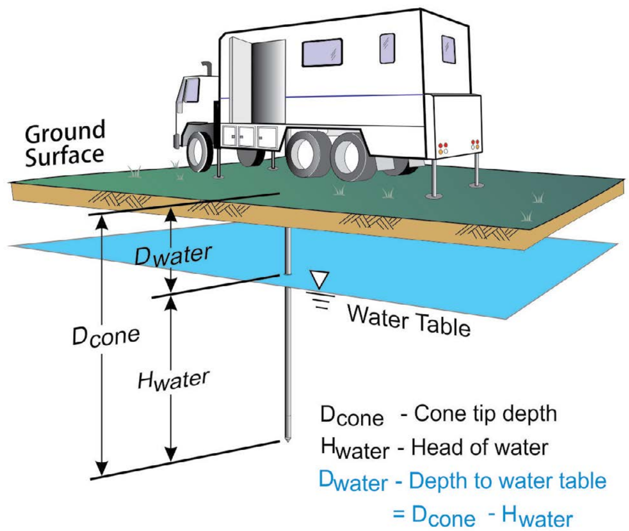


Figure PPD-1. Pore pressure dissipation test setup

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

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

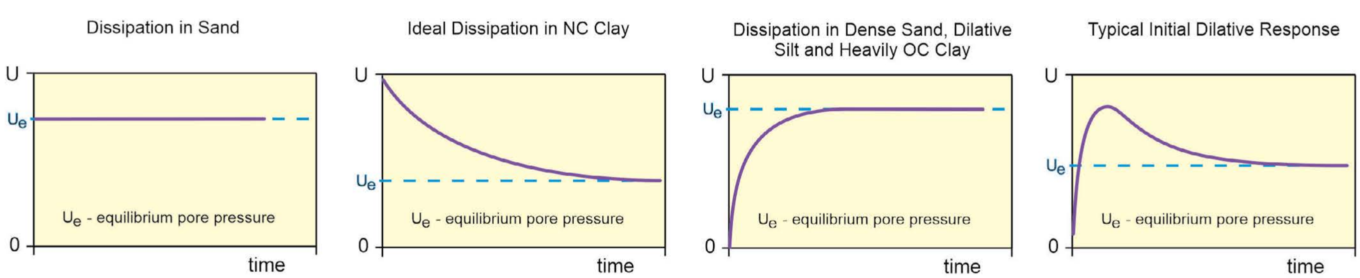


Figure PPD-2. Pore pressure dissipation curve examples

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

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

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

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

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

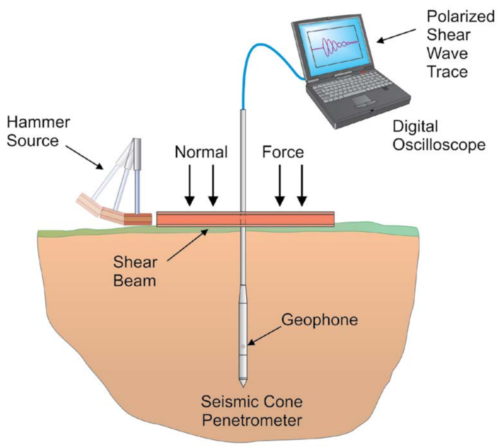


Figure SCPTu-1. Illustration of the SCPTu system

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

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

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

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

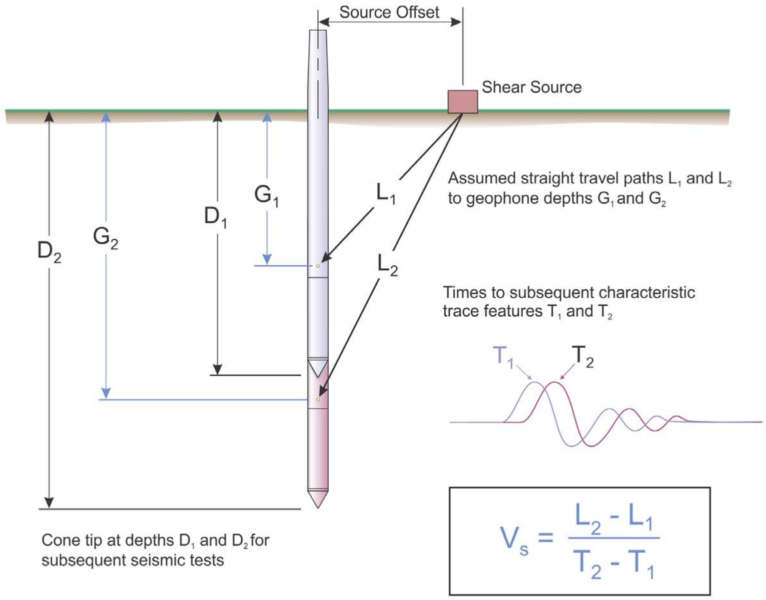


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

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

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)

d_i = the thickness of any layer between 0 and 100 ft (30 m)

v_{si} = the shear wave velocity in ft/s (m/s)

$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

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

REFERENCES

American Society of Civil Engineers (ASCE), 2010, "Minimum Design Loads for Buildings and Other Structures", Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

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

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

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

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

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

Header Lines

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

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

Data Records

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

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

End of Data Marker

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

Units Information

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

CPT Data Files (XLS Extension)

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

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

CPT Dissipation Files (XLS Extension)

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

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

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

Data Records

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

Cone Type Designations

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

refers to the Cone ID number
**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023

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

CONETEC

Limitations

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

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

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

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

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

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

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

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

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

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

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

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

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

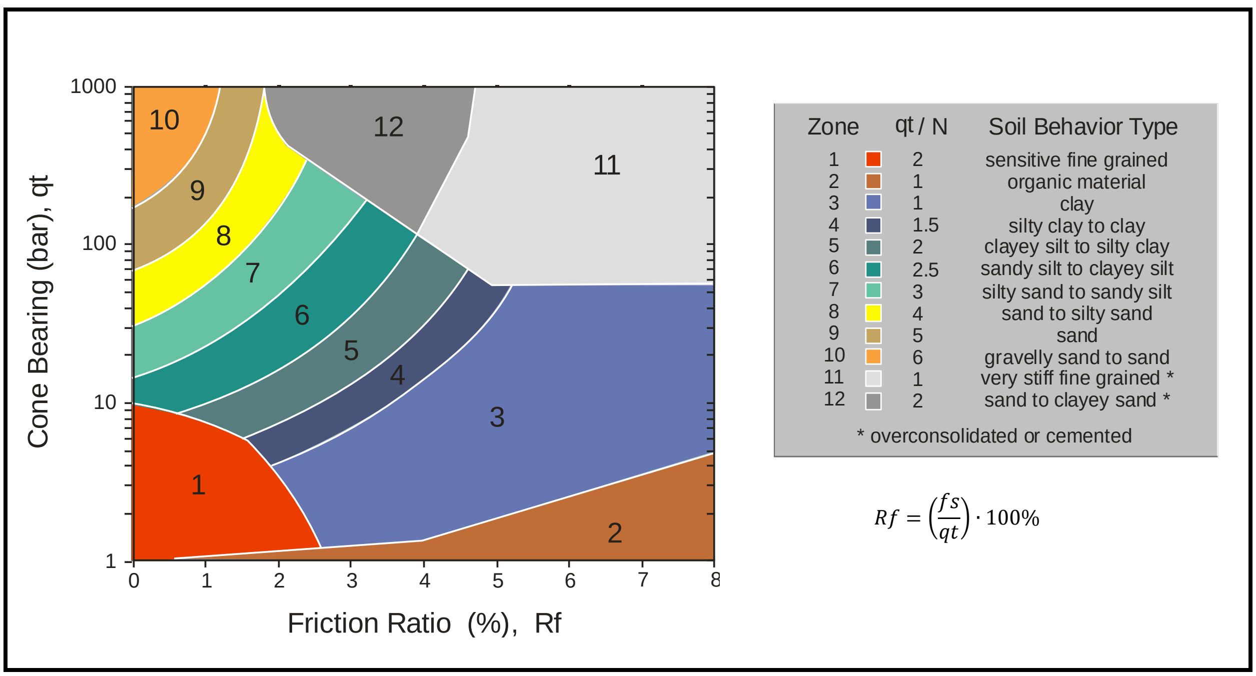


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

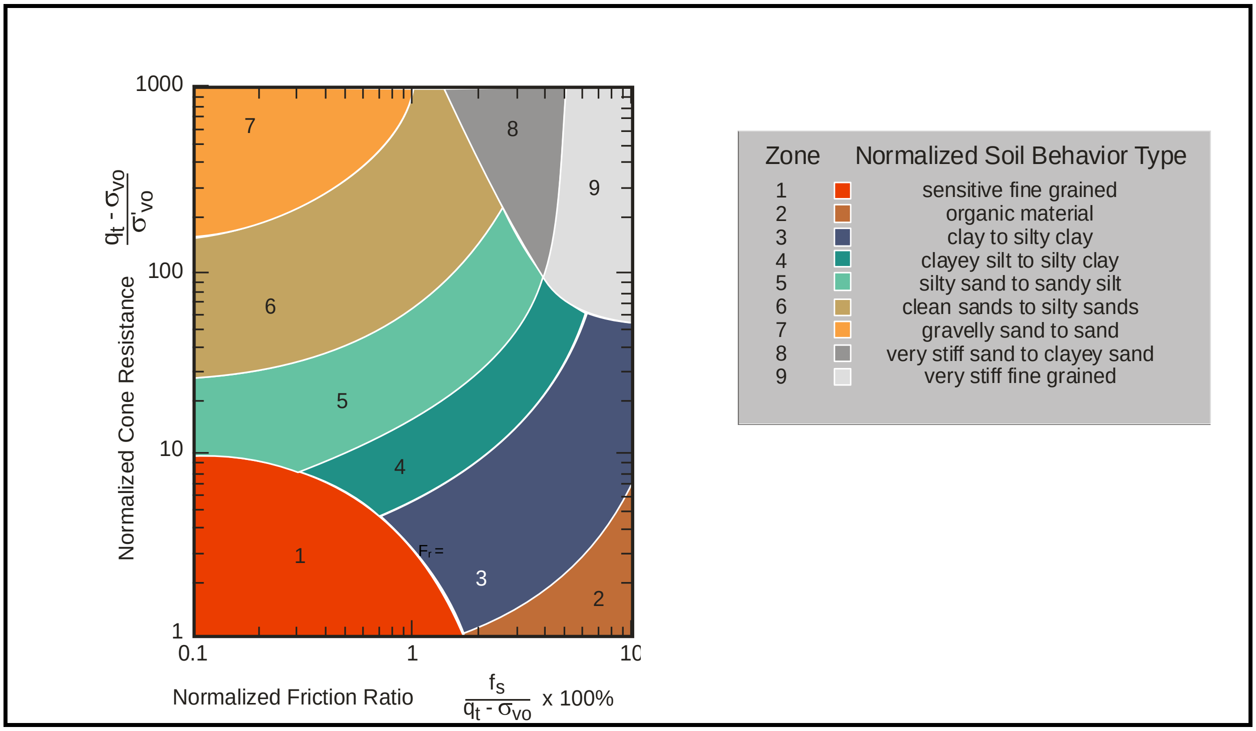


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

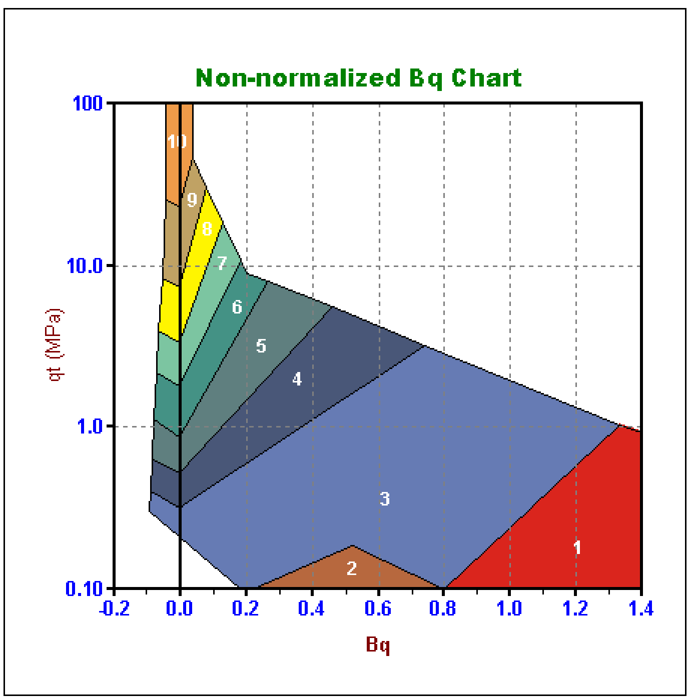


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

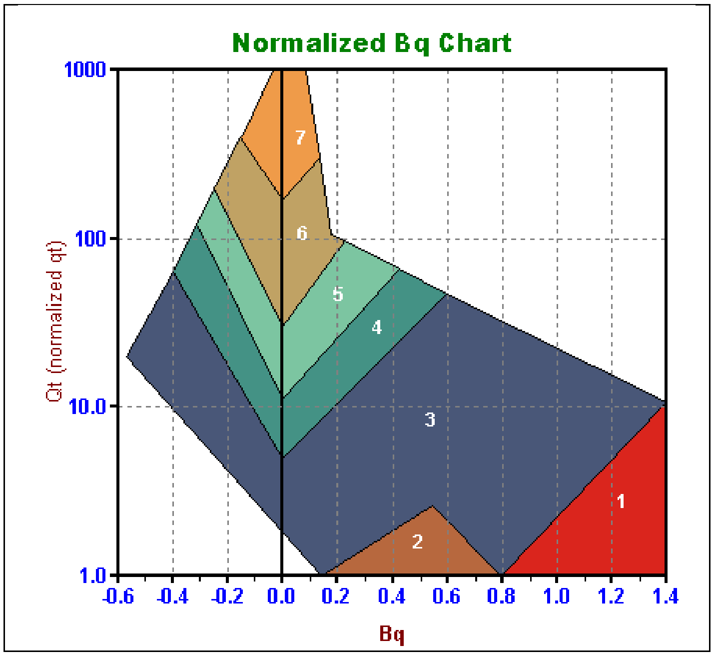


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

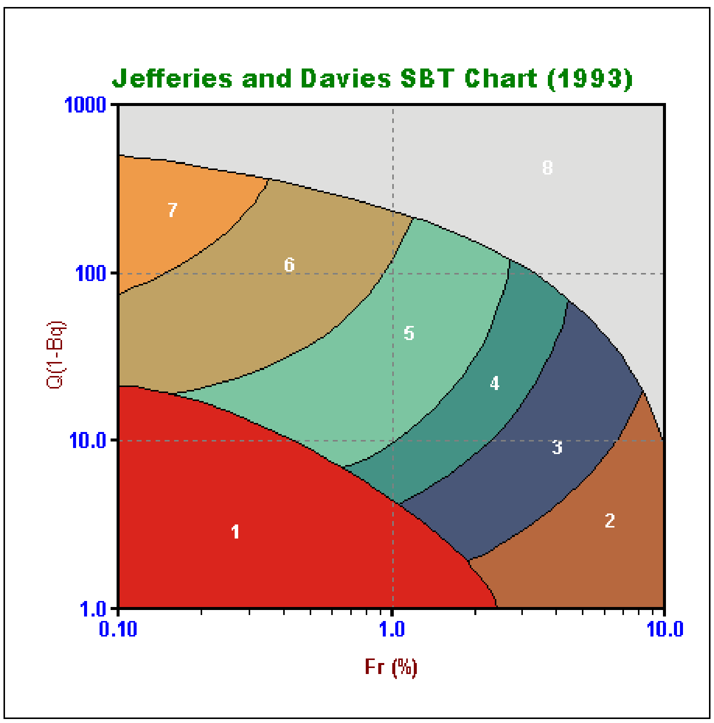


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

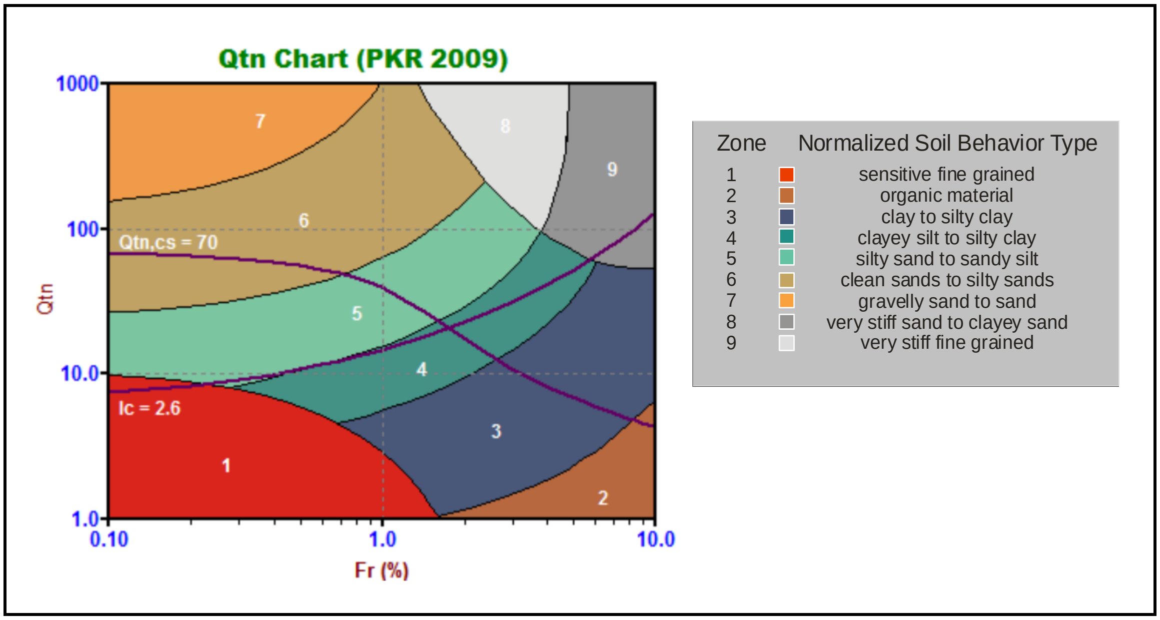


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

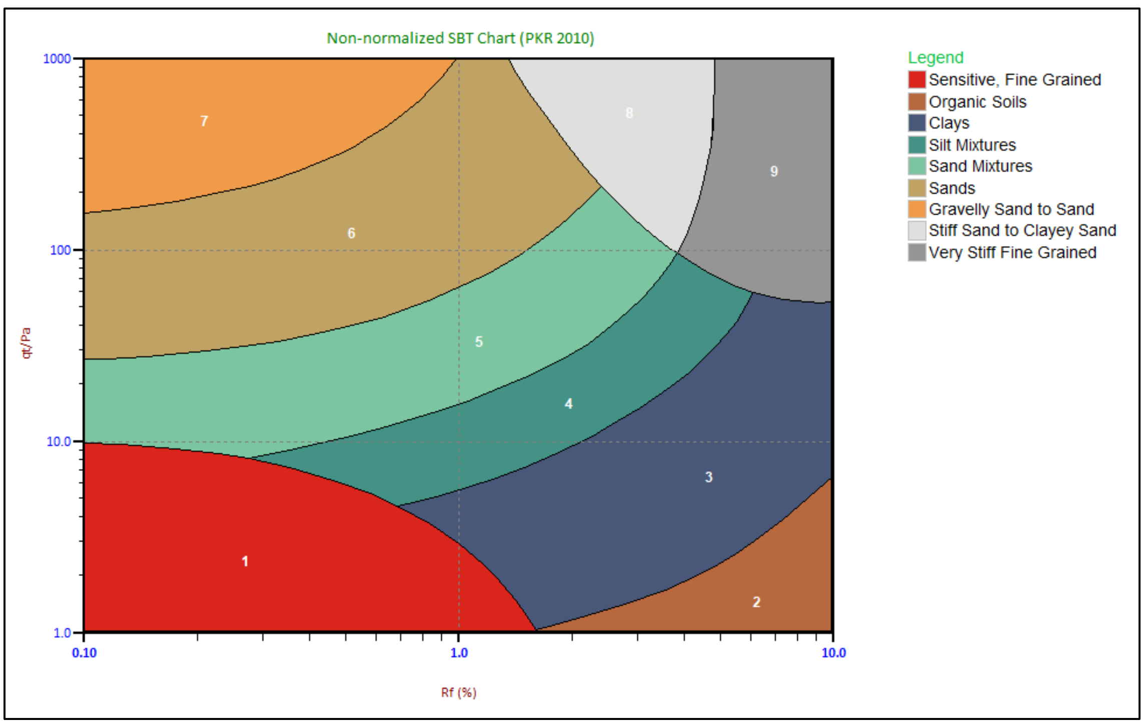


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

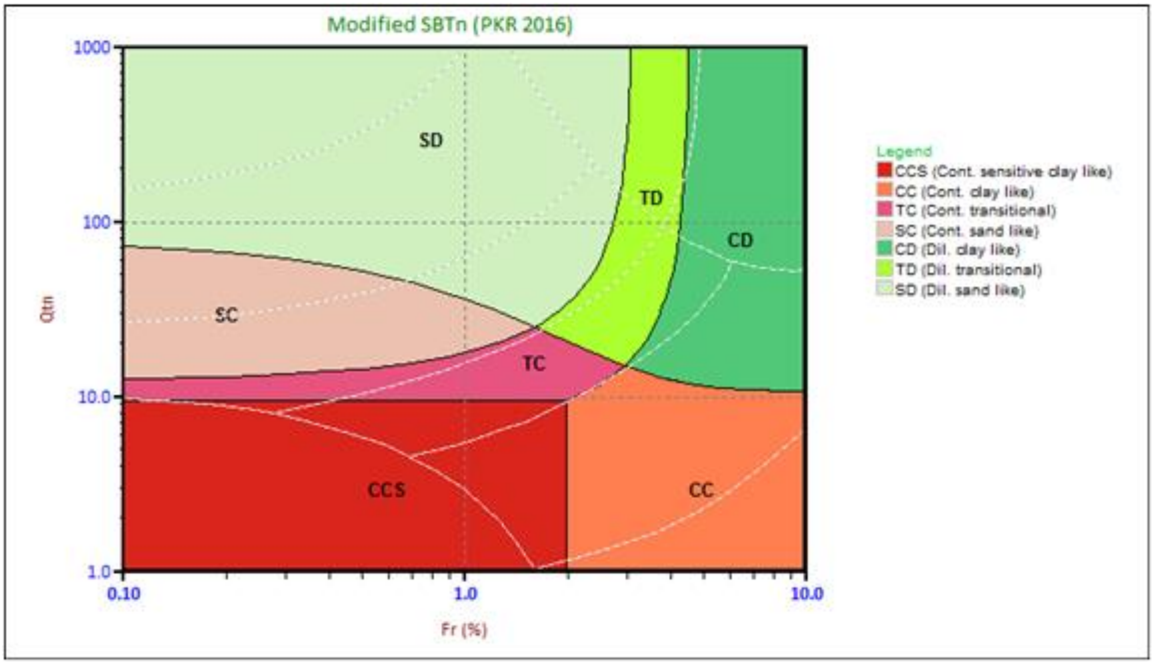


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

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

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

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

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

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

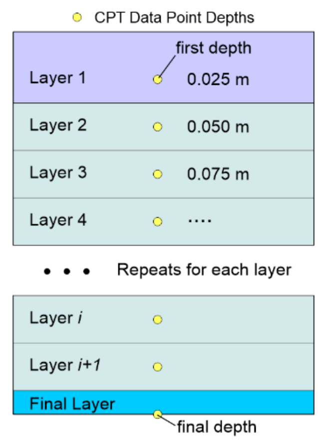
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

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

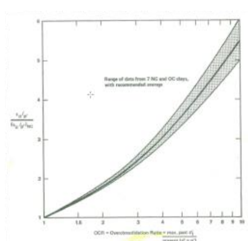
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure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

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

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

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

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

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

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

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

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

Table 2. References

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APPENDIX D

PREVIOUS EXPLORATIONS AND LAB TESTING

NOTES FOR THE BORING LOGS

FIELD NOTES:

1. The borings were drilled on April 14, 1994 with truck-mounted, power-driven, 4-inch diameter, continuous-flight auger drilling equipment.

2. All undisturbed samples were obtained with a 2.5-inch diameter, split-barrel sampler driven into the soil with a 140 pound hammer free-falling 30 inches. The numbers recorded under "Blows/Foot" are the number of blows, converted to "SPT" (Standard Penetration) blow counts, required to drive the sampler from 6 to 18 inches below the bottom of the boring.

3. Groundwater was encountered in the borings on the date and at the depth indicated on the boring logs.

LABORATORY NOTES:

1. The tabulated shear strength values are yield strength values.

2. DS = Strain controlled direct shear strength test at natural or field moisture content.

Job Name: 100 D Street
Location: Petaluma, California

Job Number: 9327
Boring Number: 1

Percent Fines (-#200)	Liquid Limit %	Plasticity Index	Type Strngth Test	Test Surch. Press. psf	Test Moist. Cont. %	Shear Strngth psf	Natural Moist. Cont. %	Dry Density pcf	Sampler Type-Blows/Foot	Feet	Visual Classification
											[GP] 6" SANDY GRAVEL Surfacing
										1	[ML] Gray-brown SANDY SILT medium stiff, moist, some
			DS	500	Natural	450	10.8	107	14	2	debris
										3	
			DS	1000	Natural	400	21.7	100	10	4	[CL] Gray-brown SILTY CLAY medium stiff, wet
										5	
			DS	2000	Natural	1250	22.0	102	10	6	Gray SANDY CLAY medium stiff, wet
										7	
										8	ground water end of drilling
										9	[SC] Lt. Brown CLAYEY SAND medium dense, wet
										10	[SM] Lt. Brown SILTY SAND loose to medium dense,
							21.7	105	13	11	saturated

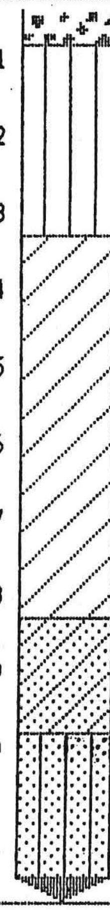


PLATE 2-A

Job Name: 100 D Street
Location: Petaluma, California

Job Number: 9327
Boring Number: 2

Percent Fines (-#200)	Liquid Limit %	Plasticity Index	Type Strngth Test	Test Surch. Press. psf	Test Moist. Cont. %	Shear Strngth psf	Natural Moist. Cont. %	Dry Density pcf	Sampler Type-Blows/Foot	Feet	Visual Classification
							16.7	111	28	1	[GC] Brown CLAYEY SANDY GRAVEL dense to very dense, moist some debris (FILL)
										2	
			DS	400	Natural	350	19.0	106	8	3	[CL] Gray SANDY CLAY medium stiff, wet
										4	
			DS	800	Natural	320	32.4	88	6	5	Gray SILTY CLAY soft to medium stiff, wet to saturated
										6	color change to dk gray
			DS	1500	Natural	800	21.7	100	8	7	
										8	
										9	Gray-brown SANDY CLAY medium stiff, saturated
							20.4	104	15	10	becomes stiff

Revisions: By ______ Date ______ By ______ Date ______

Location ______

By ______ Date ______ Checked By ______ Job Number ______ Name ______

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	[GW]	Well graded gravel or sand-gravel mixtures, no fines
		[GP]	Poorly graded gravel or sand-gravel mixtures, no fines
		[GM]	Silty gravel, gravel-sand-silt mixtures
		[GC]	Clayey gravel, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < no. 4 sieve size)	[SW]	Well graded sand or gravelly sand, no fines
		[SP]	Poorly graded sand, gravelly sand, no fines
		[SM]	Silty sand, sand-silt mixtures
		[SC]	Clayey sand, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	[ML]	Inorganic silts and very fine sand, silty/clayey fine sand
		[CL]	Inorganic clay of low plasticity, lean clay
		[OL]	Organic silt, organic silty clay of low plasticity
	SILTS & CLAYS LL > 50	[MH]	Inorganic silt, micaceous or diatomaceous or elastic silt
		[CH]	Inorganic clay of high plasticity, fat clay
		[OH]	Organic clay, silt, or silty clay of high plasticity
HIGHLY ORGANIC SOILS		[PT]	Peat and other highly organic soils

CLASSIFICATION CHART

CLASSIFICATION	U.S. STANDARD SIEVE SIZE	CLASSIFICATION	U.S. STANDARD SIEVE SIZE
BOULDERS	Above 12"	SAND	
COBBLES	12" to 3"	coarse	No. 4 to No. 10
GRAVEL		medium	No. 10 to No. 40
coarse	3" to 3/4"	fine	No. 40 to No. 200
fine	3/4" to No.4	SILT & CLAY	Below No. 200

GRAIN SIZE CHART

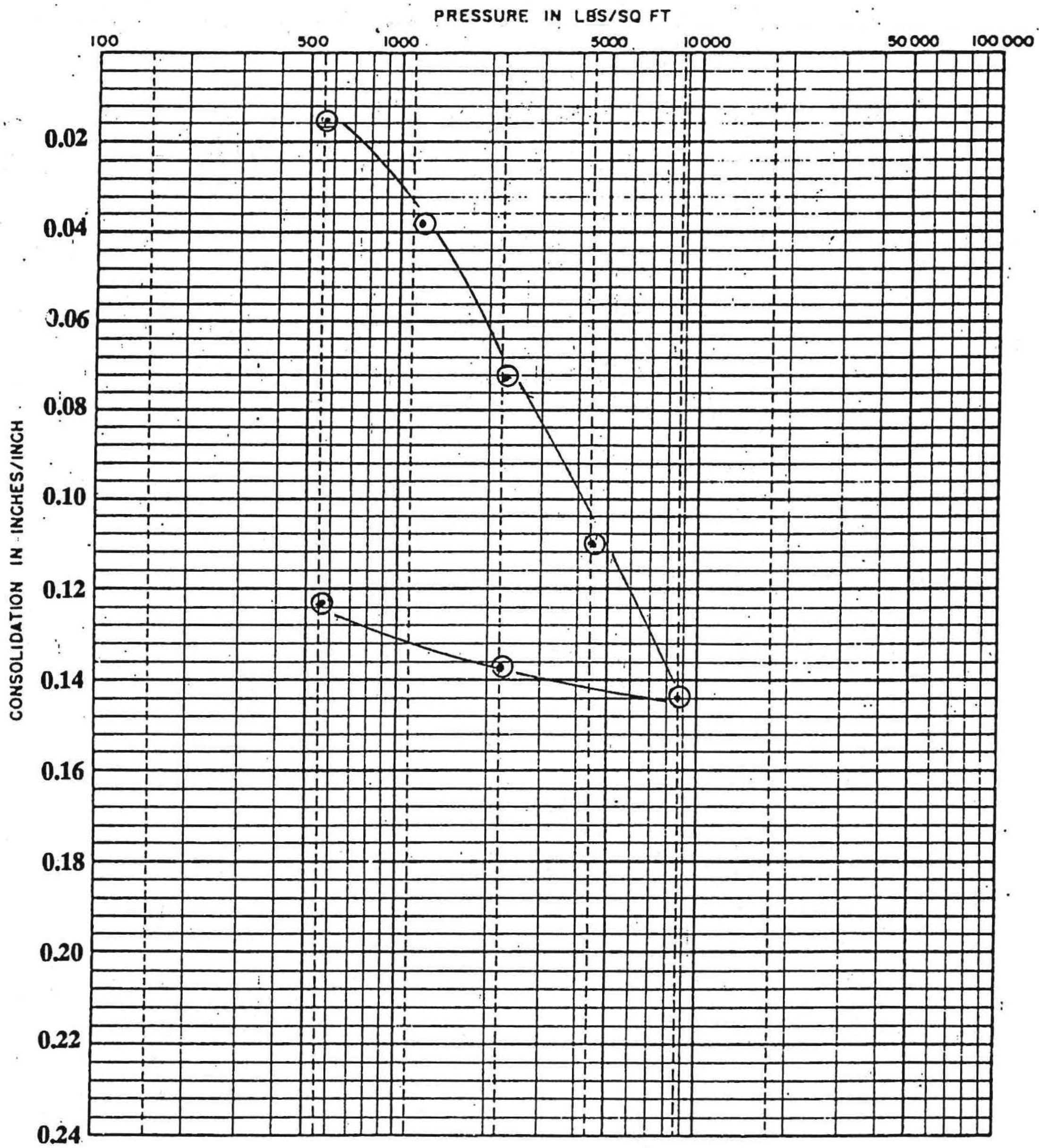
METHOD OF SOIL CLASSIFICATION

(Unified Soil Classification System)

PLATE 3

Revisions:
By______ Date______
By______ Date______
Location______

By______ Date______
Checked By______
Job Number______ Name______



KEY	BORING	DEPTH (FT)	SOIL DESCRIPTION	SOIL TYPE	NATURAL MOISTURE CONTENT %	NATURAL DRY DENSITY (P.C.F.)	SPECIFIC GRAVITY
⊙	1	4	Greyish-brown SANDY SILTY CLAY	CL	28.3	98	

CONSOLIDATION TEST DATA

PLATE 4

Berlogar, Stevens & Associates, Inc.
Pleasanton, California

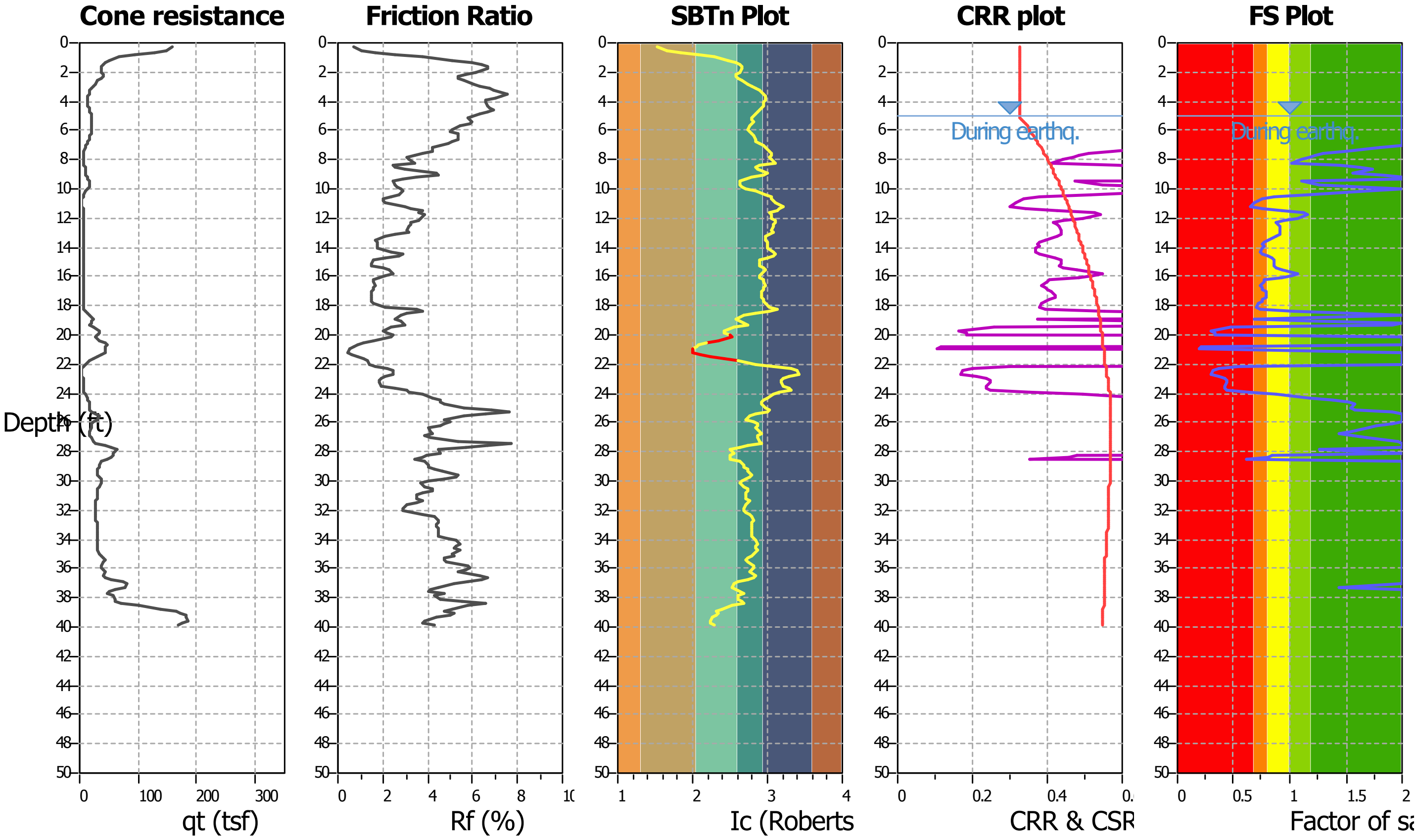
LIQUEFACTION ANALYSIS REPORT

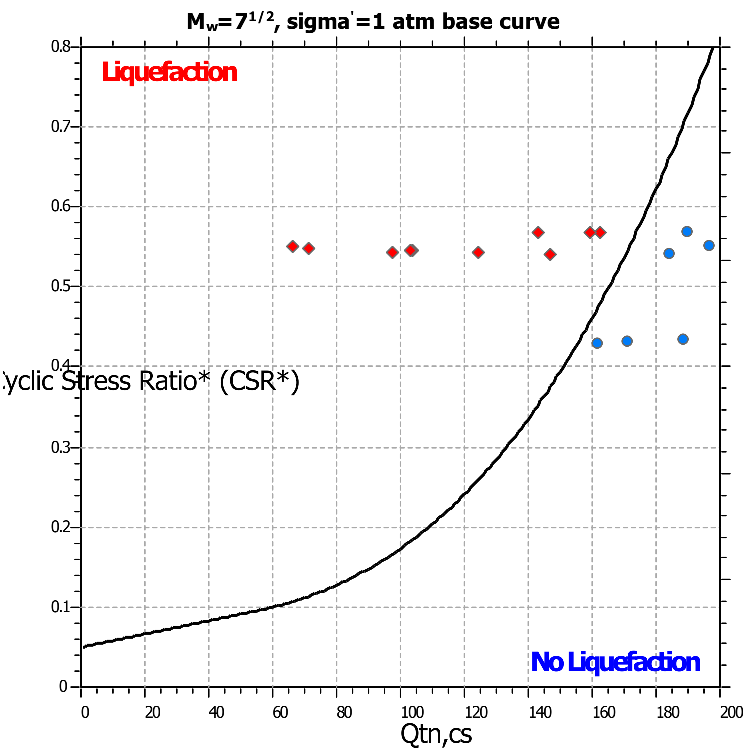
Project title : East D Street **Location : Petaluma, CA**

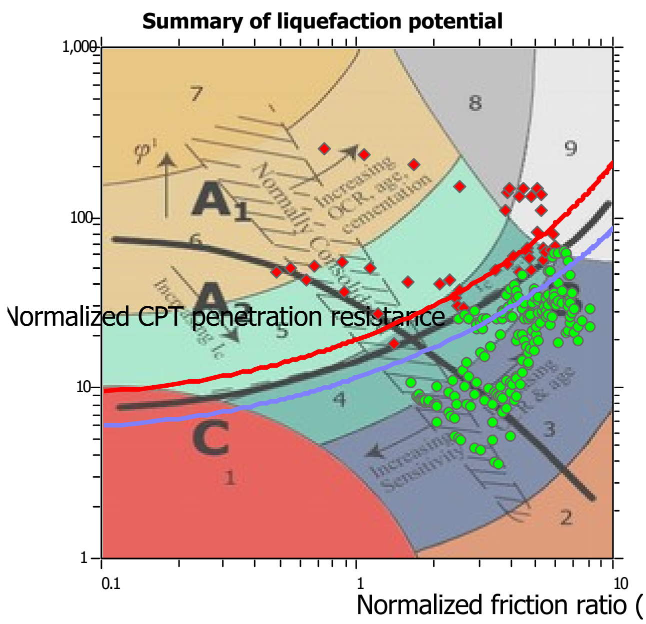
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

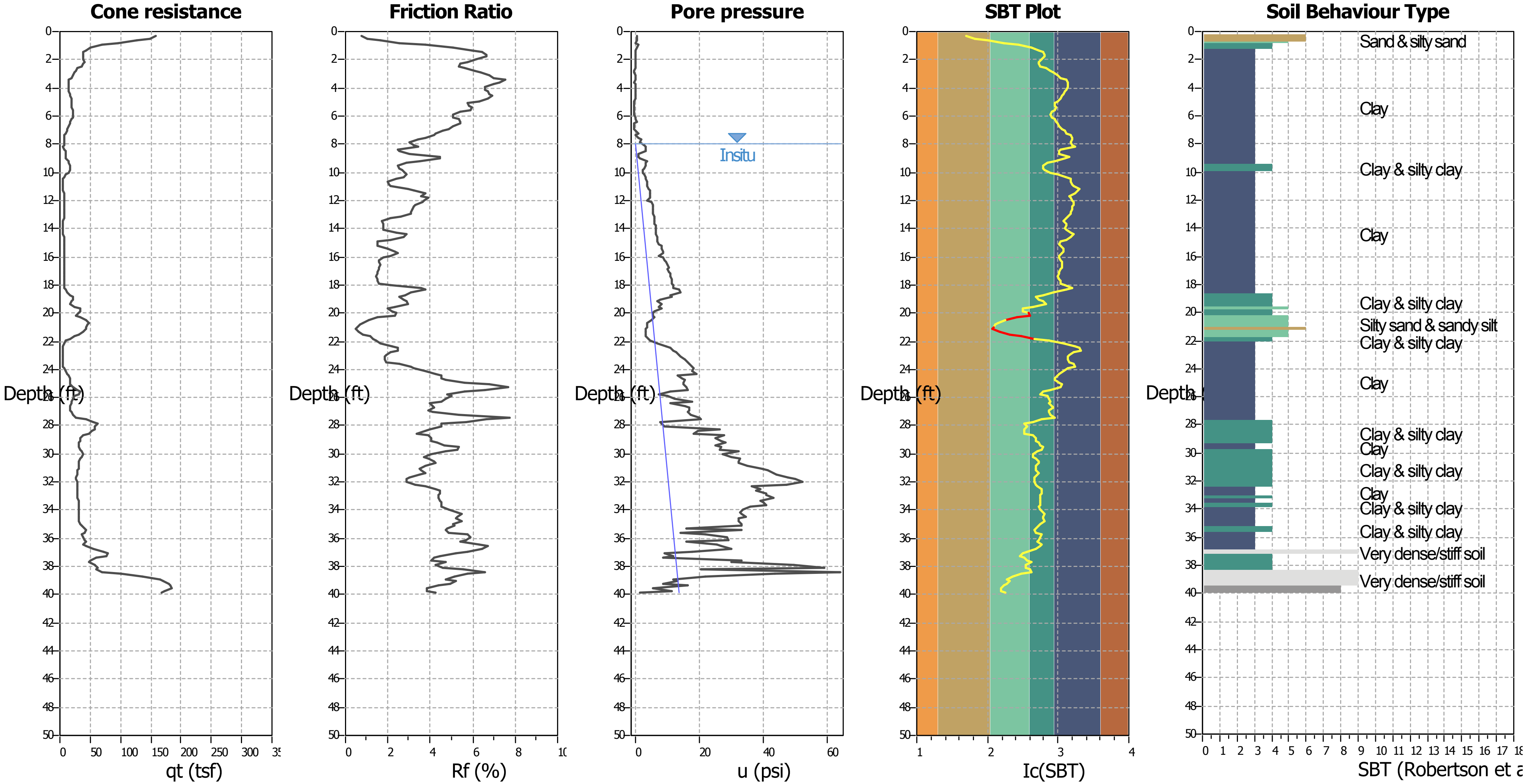






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



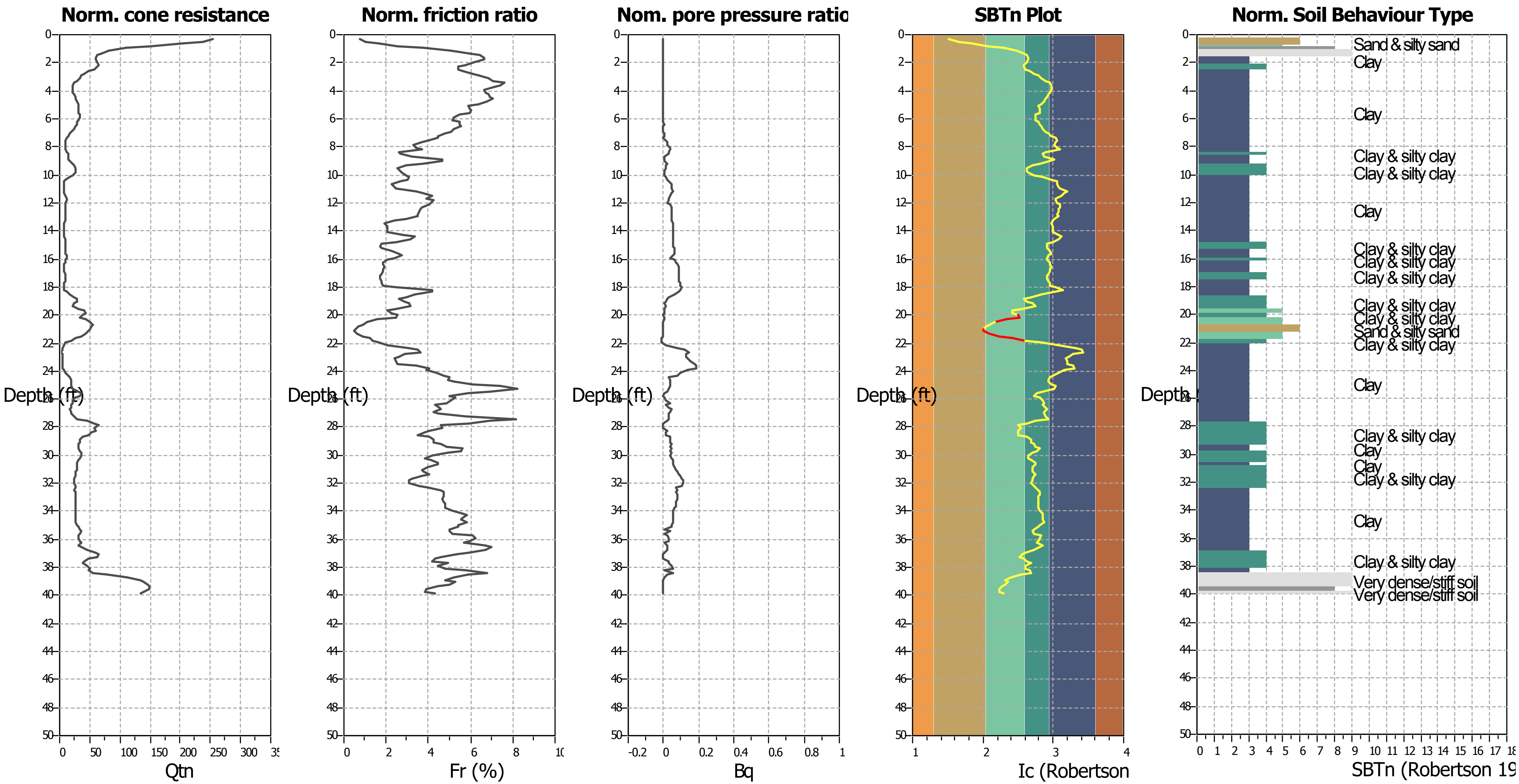
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



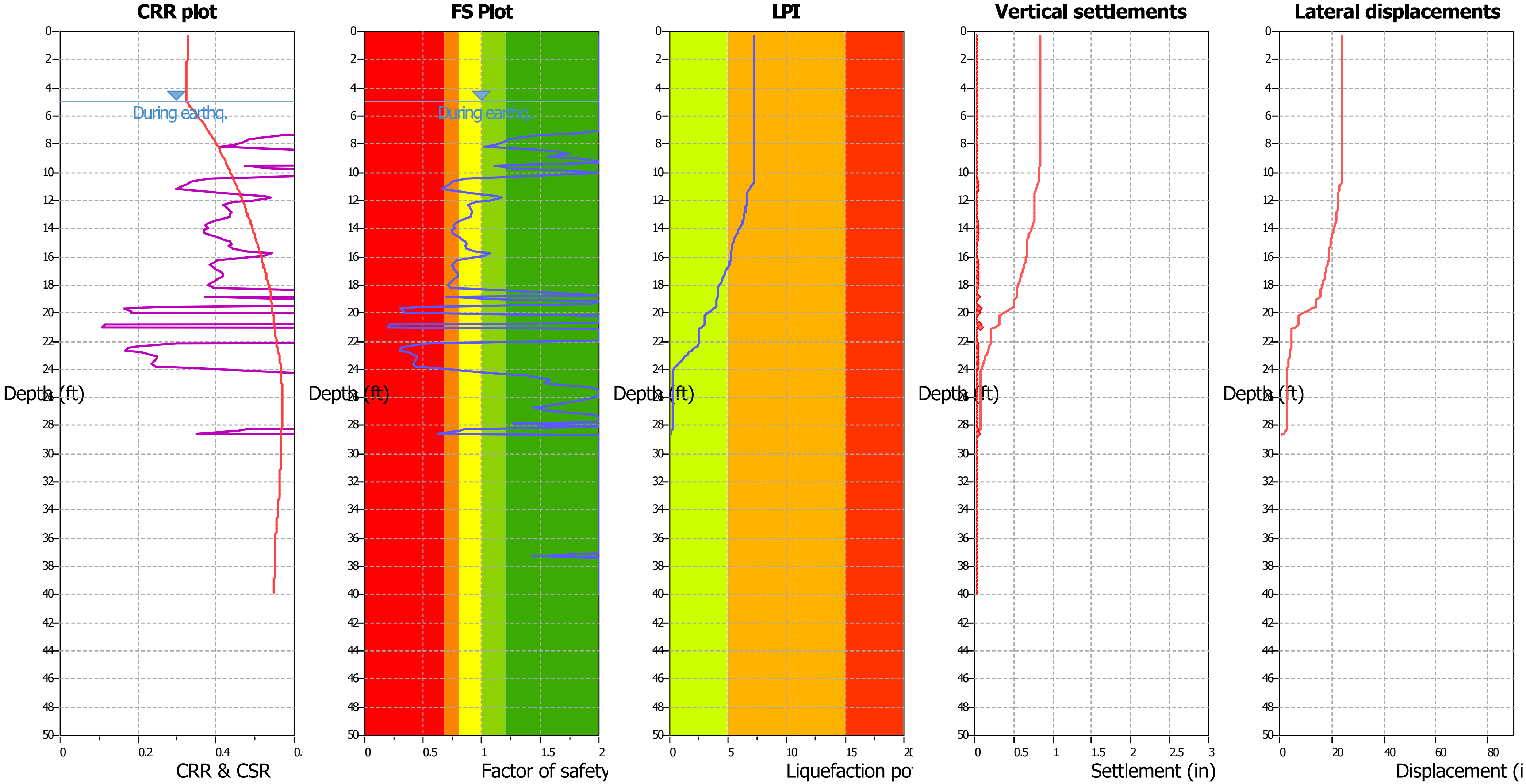
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

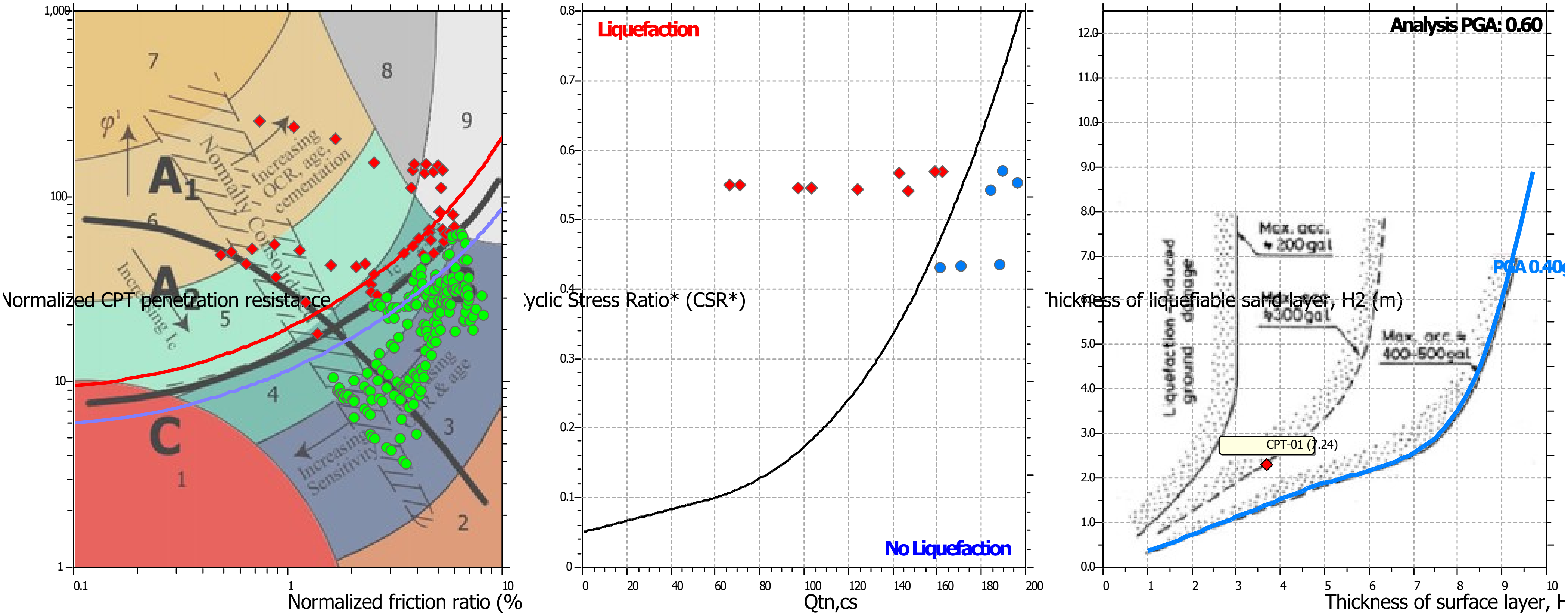
F.S. color scheme

- 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

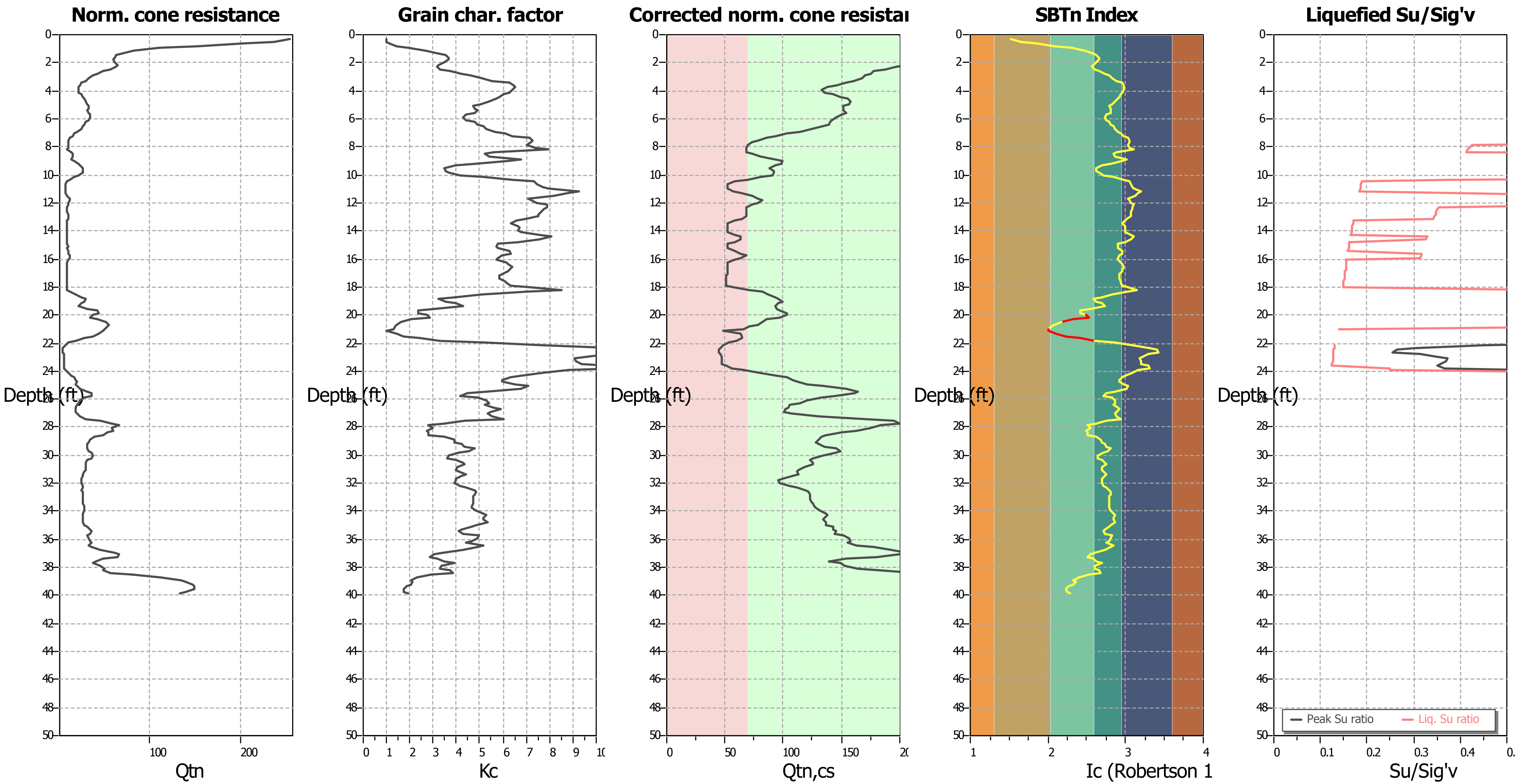
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Berlogar, Stevens & Associates, Inc.
Pleasanton, California

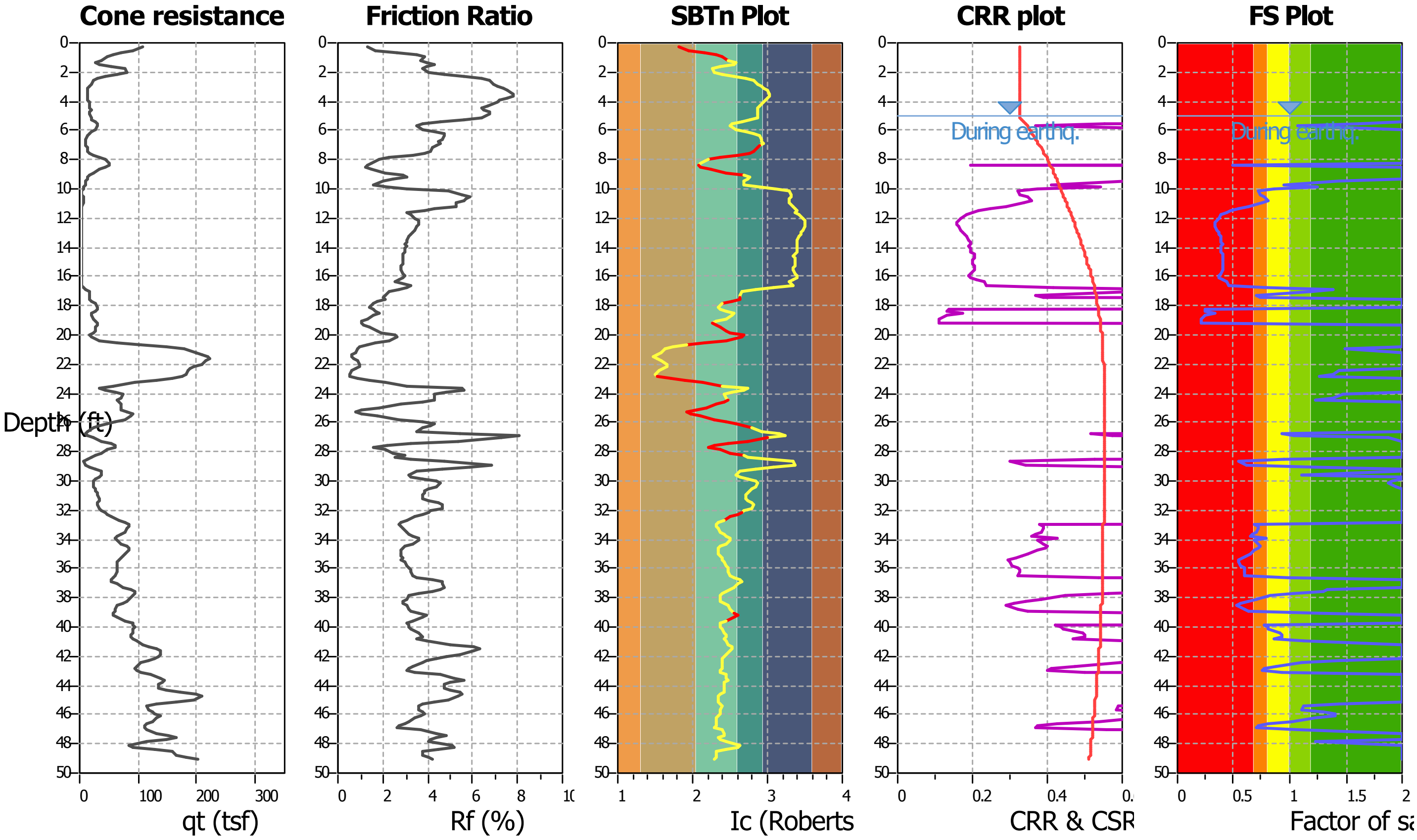
LIQUEFACTION ANALYSIS REPORT

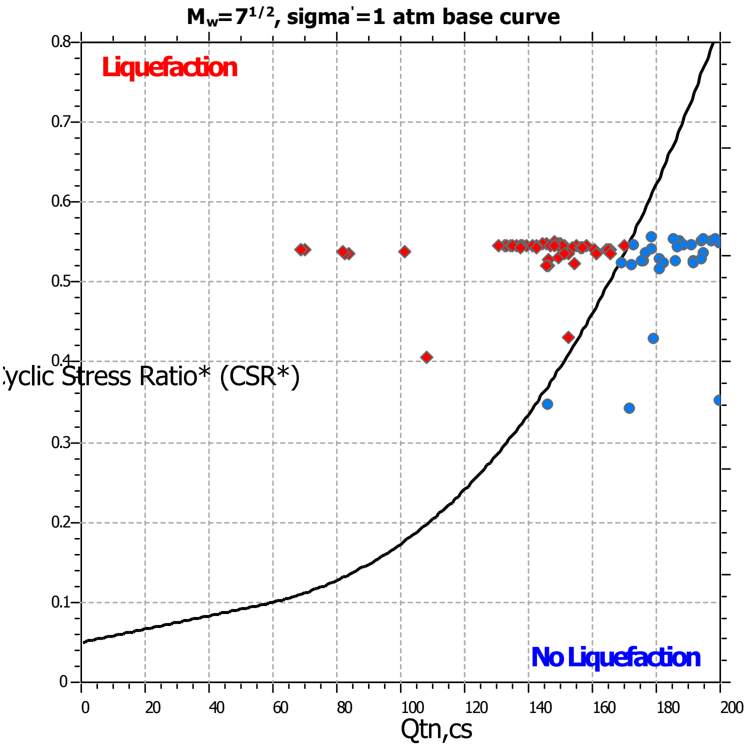
Project title : East D Street **Location : Petaluma, CA**

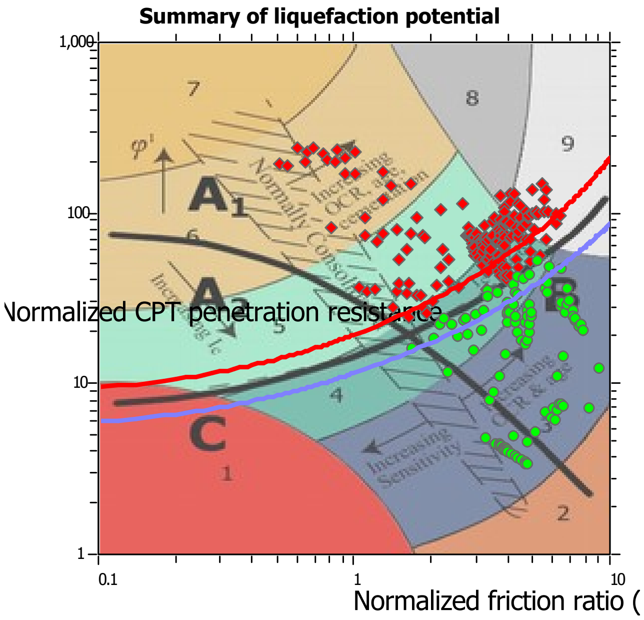
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

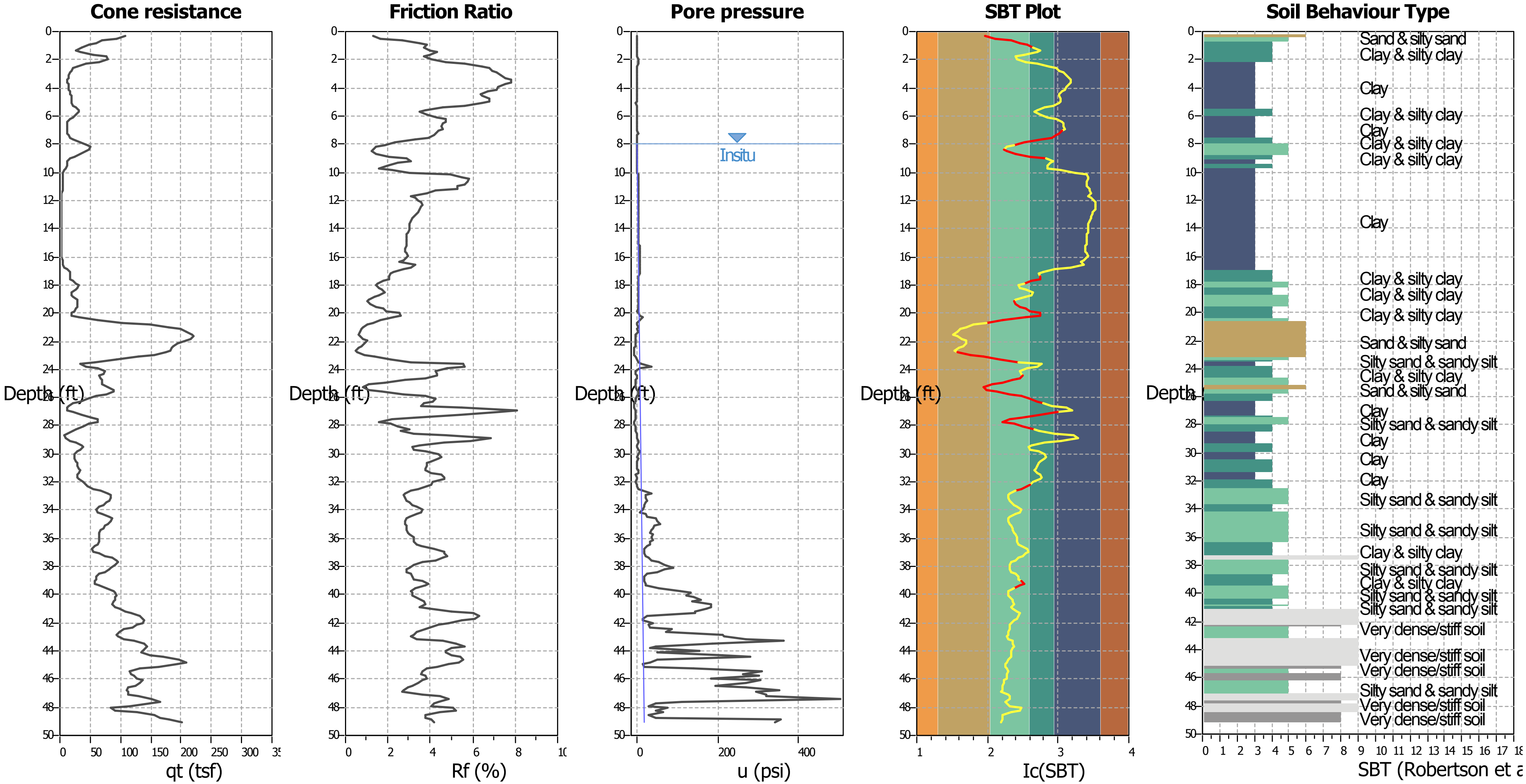






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



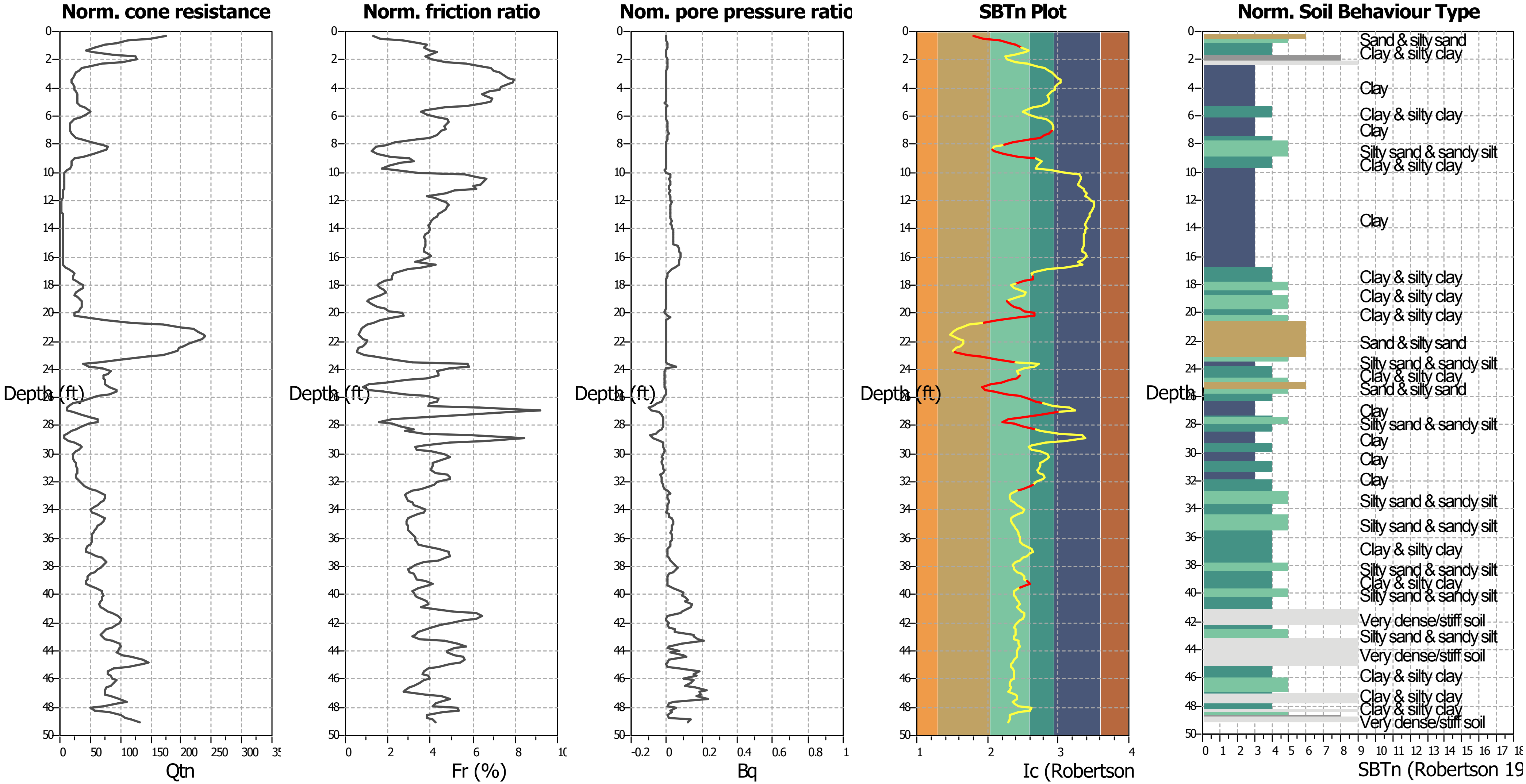
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



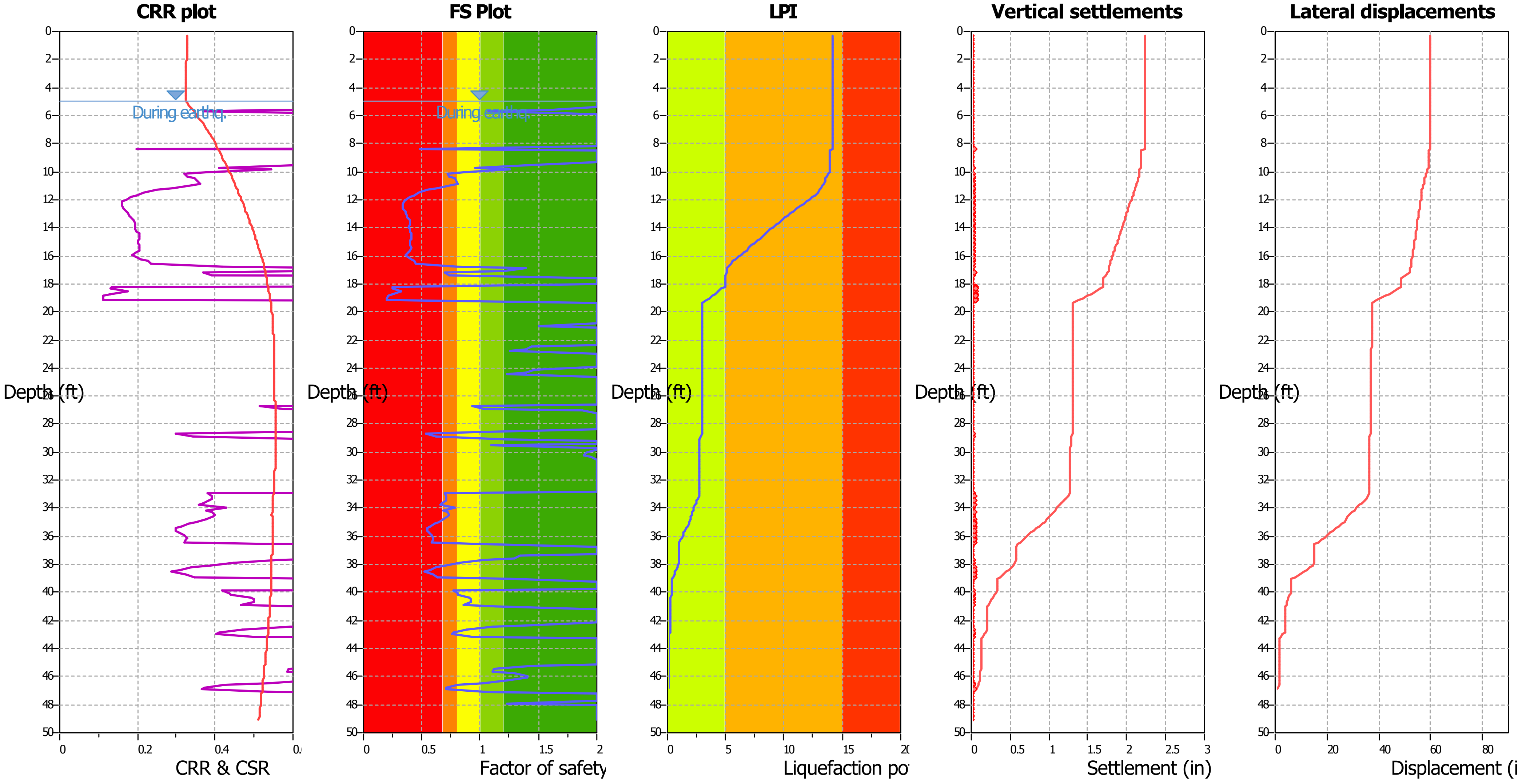
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

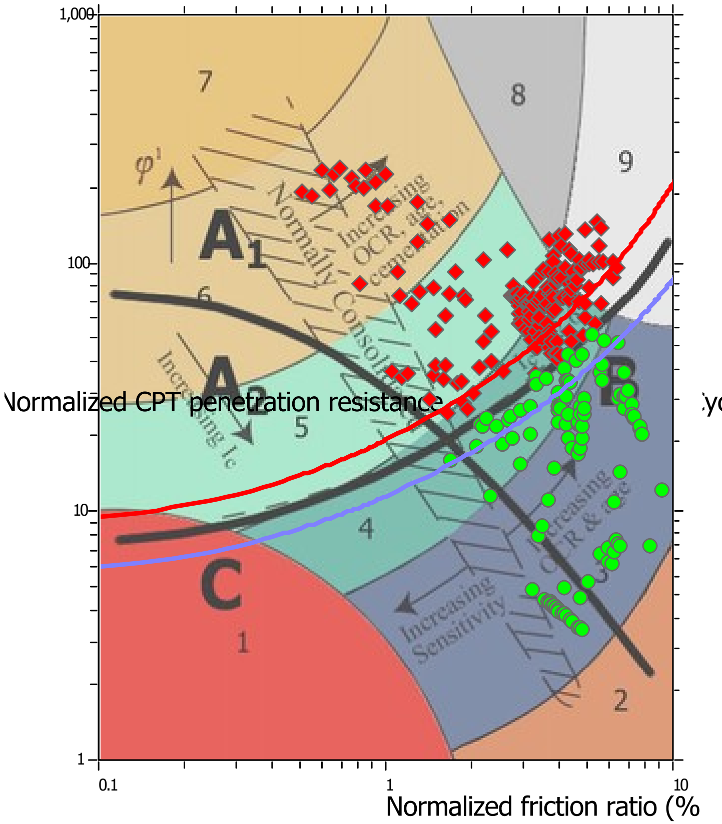
F.S. color scheme

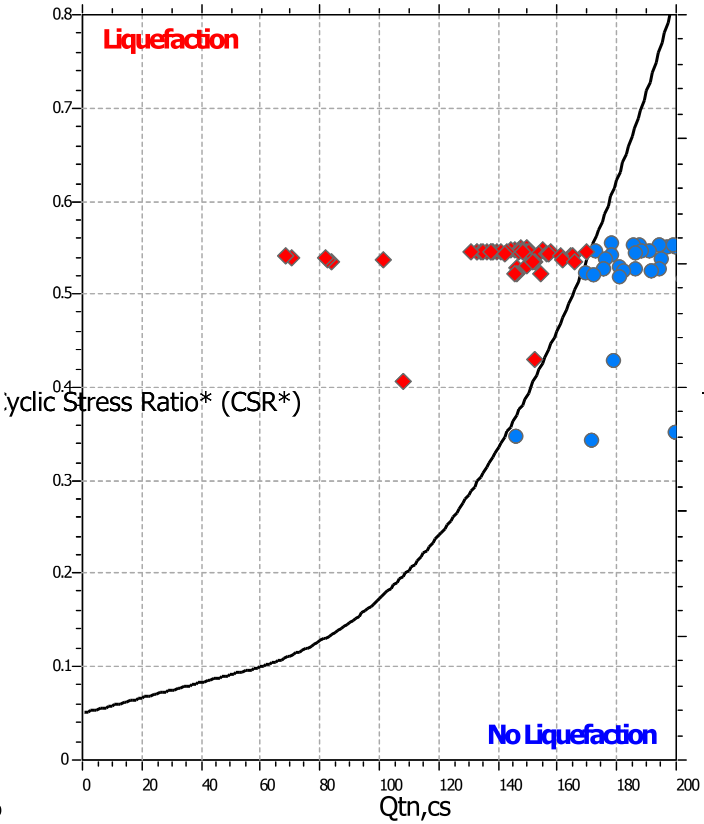
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

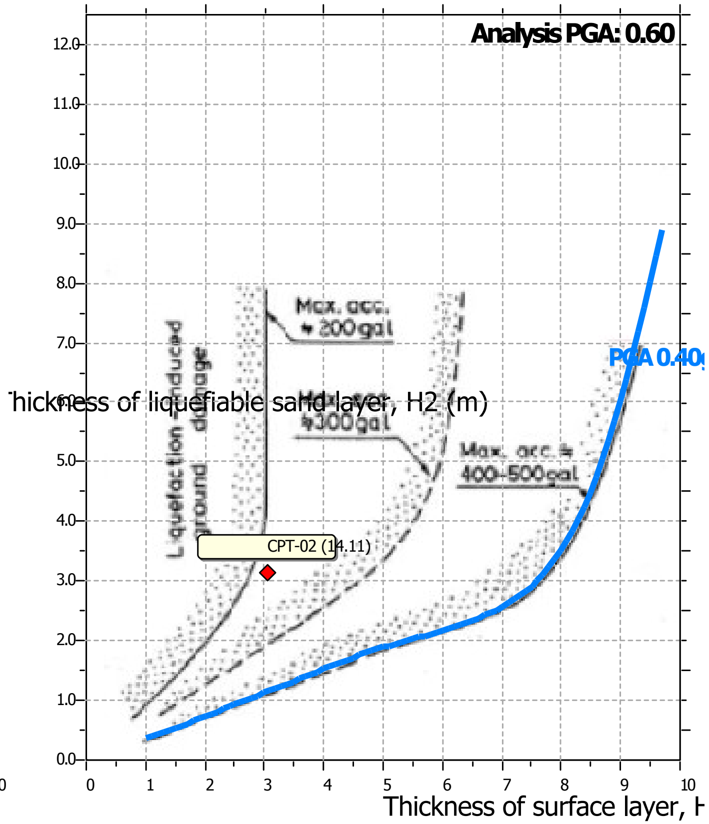
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



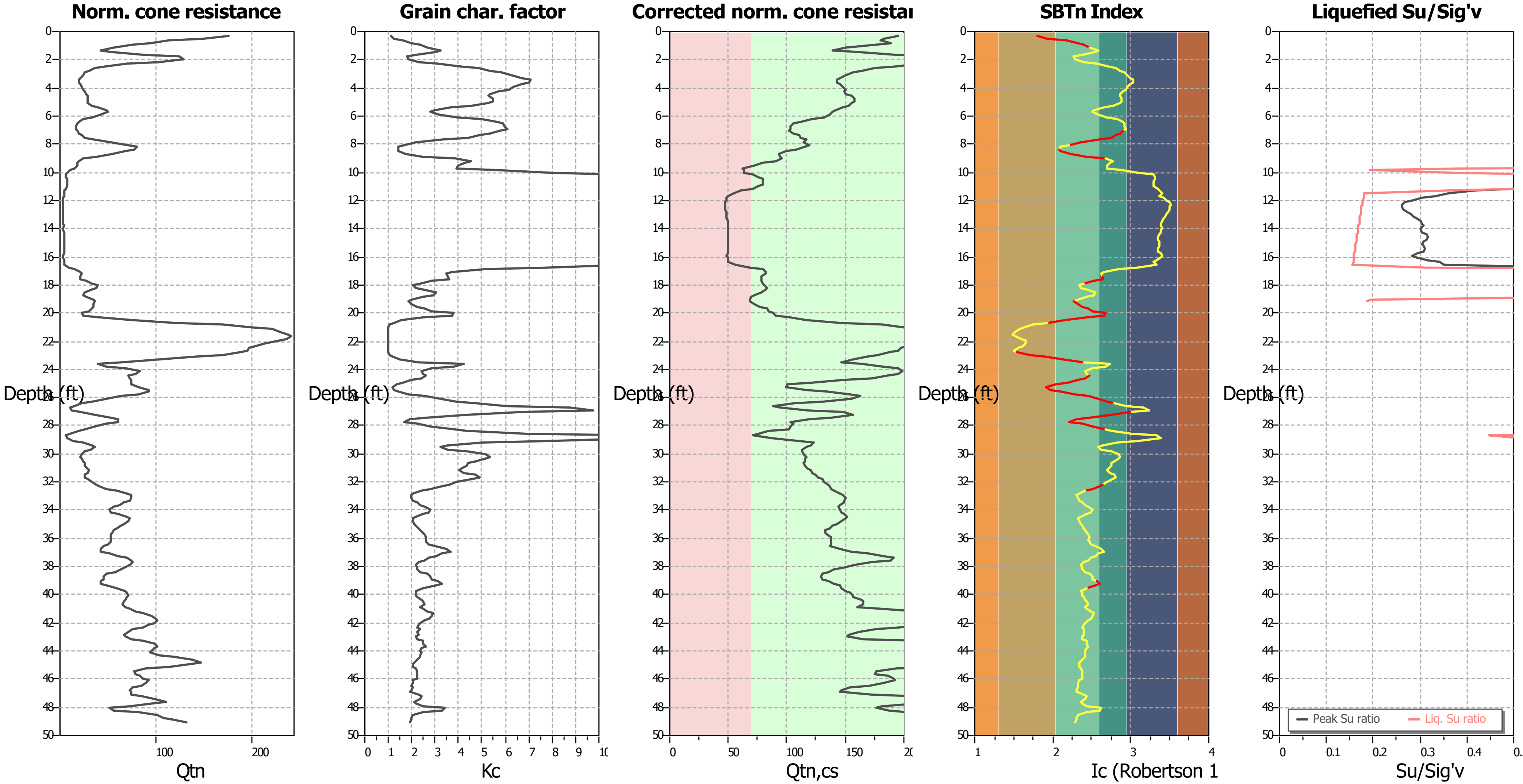




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Berlogar, Stevens & Associates, Inc.
Pleasanton, California

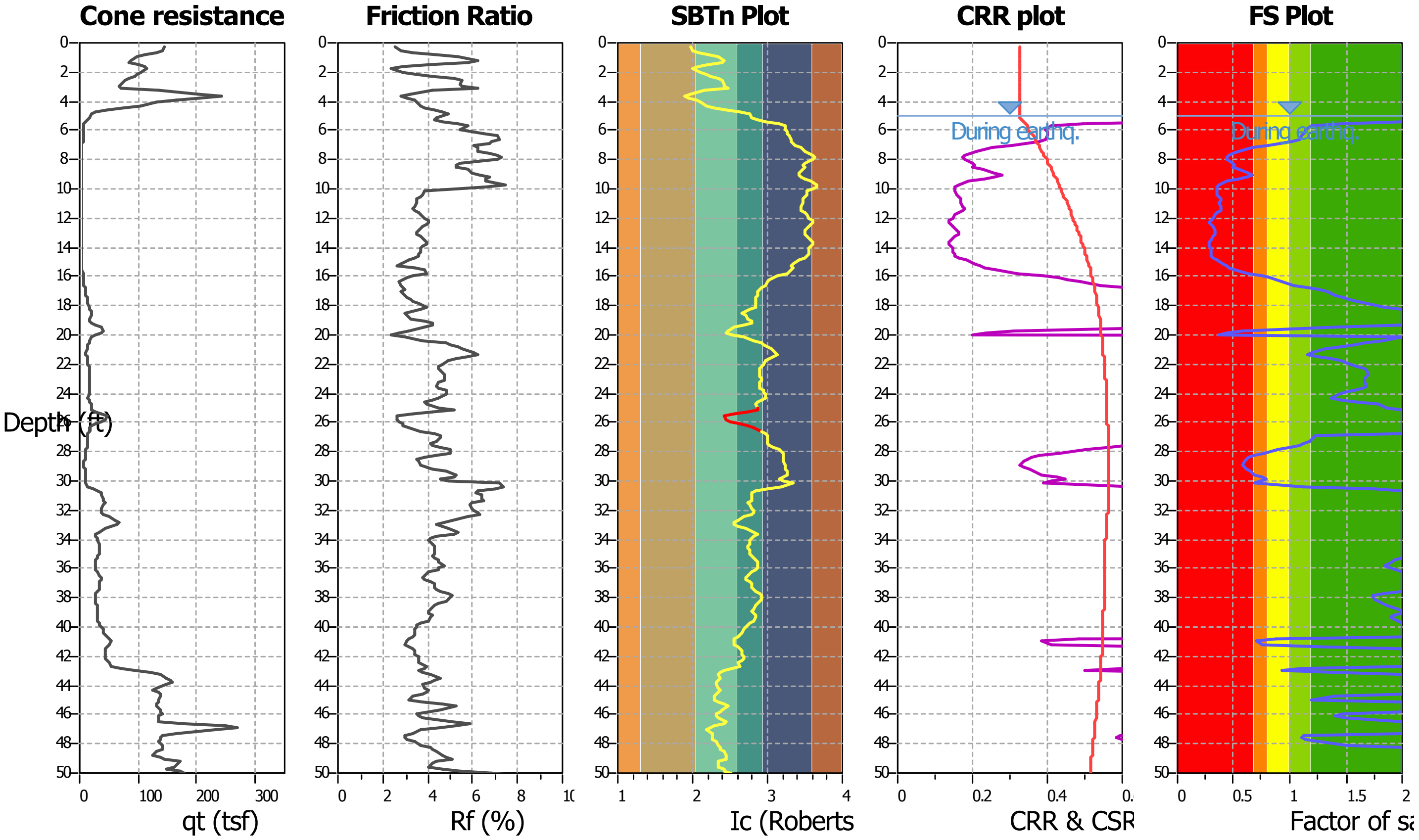
LIQUEFACTION ANALYSIS REPORT

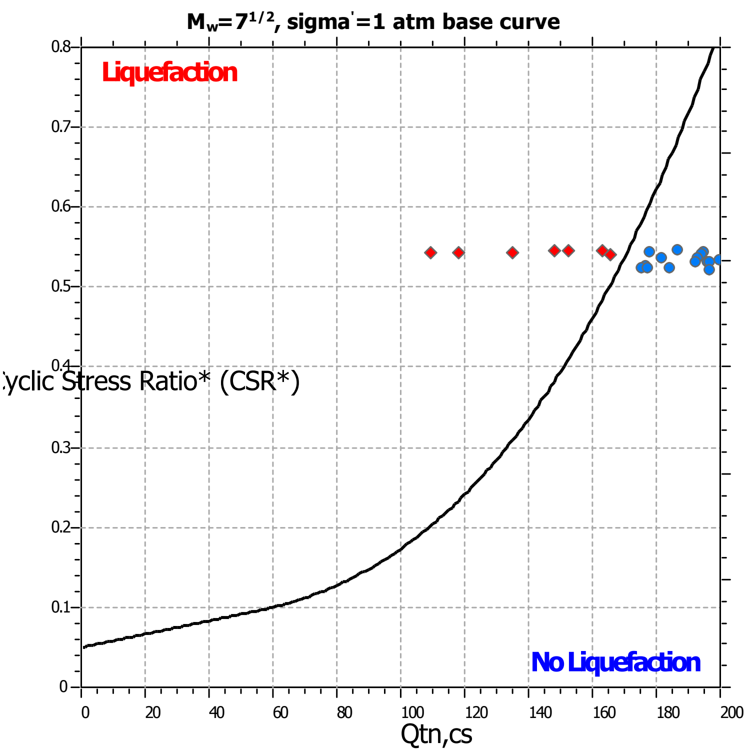
Project title : East D Street **Location : Petaluma, CA**

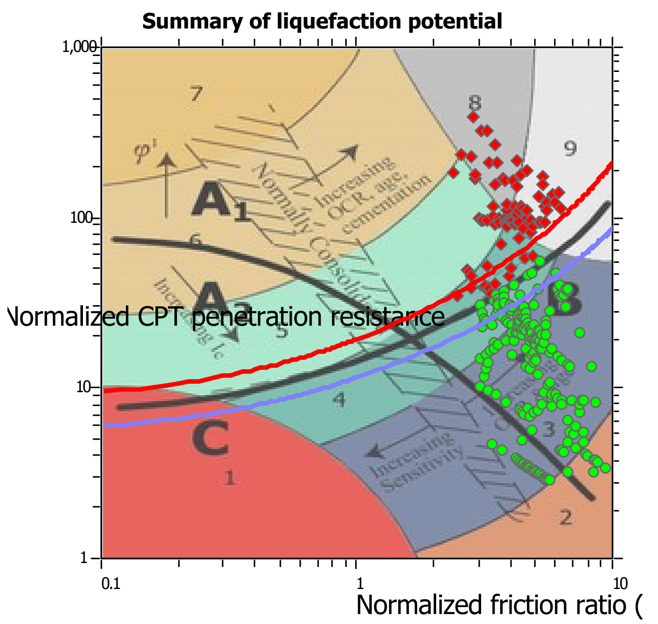
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

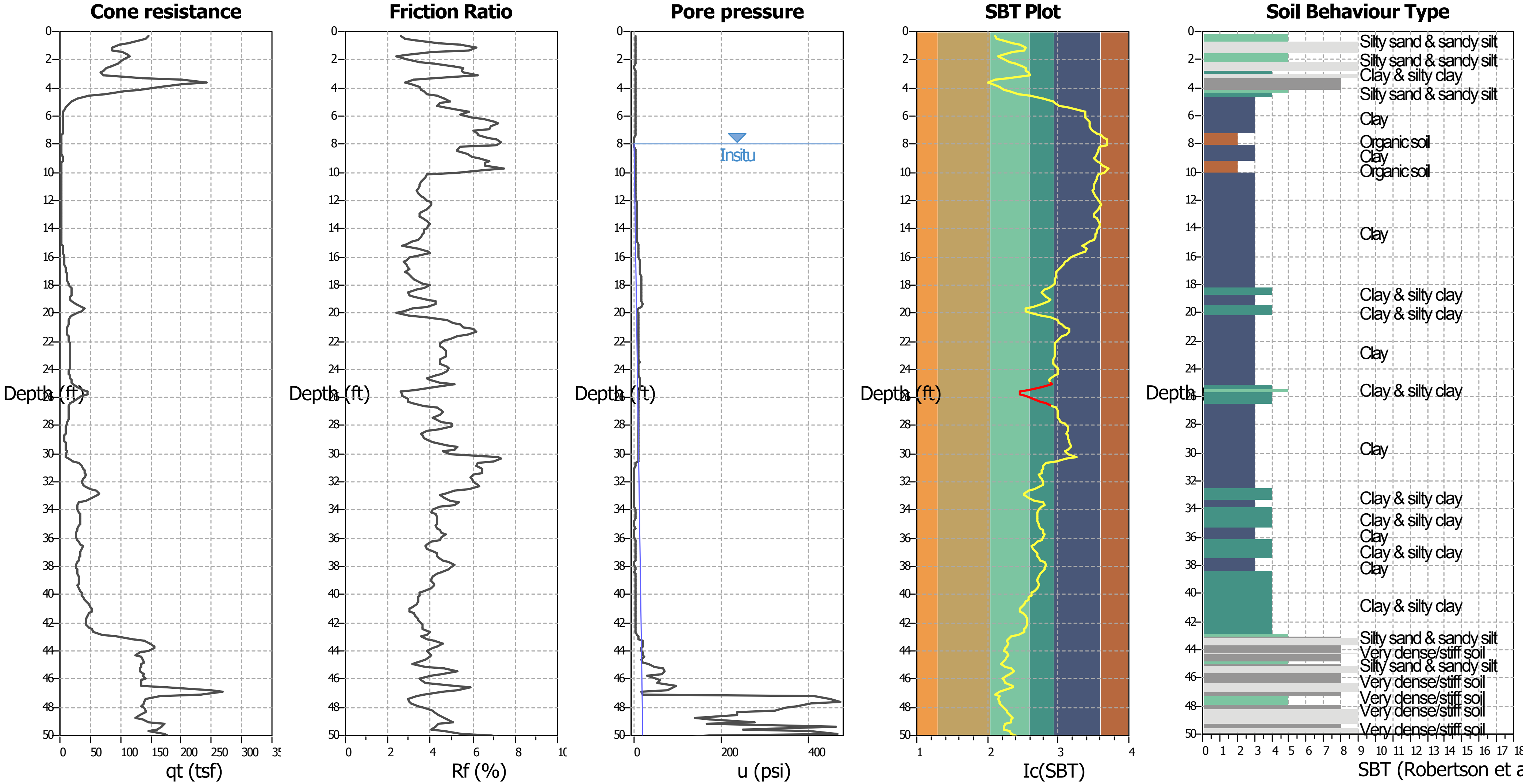






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



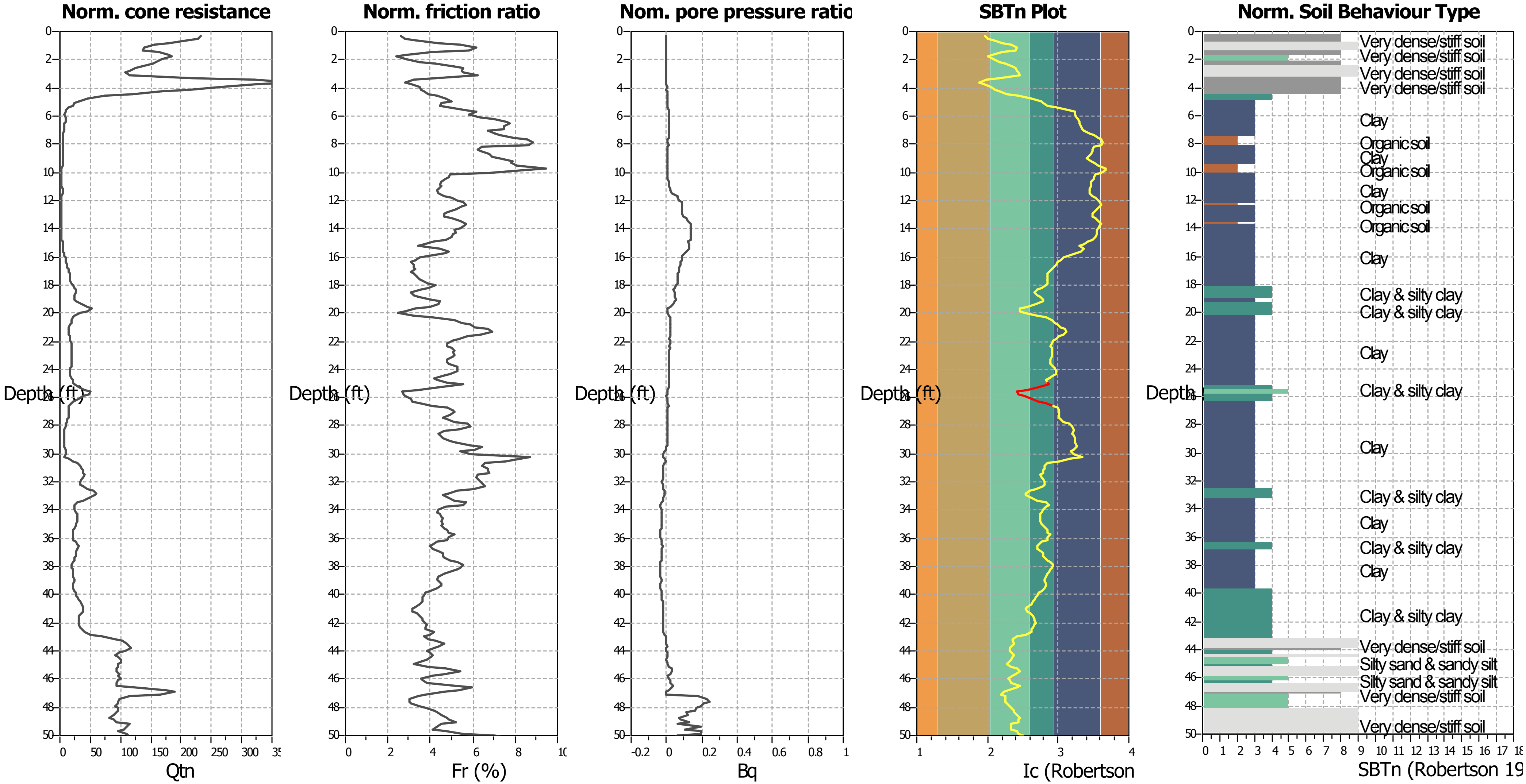
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



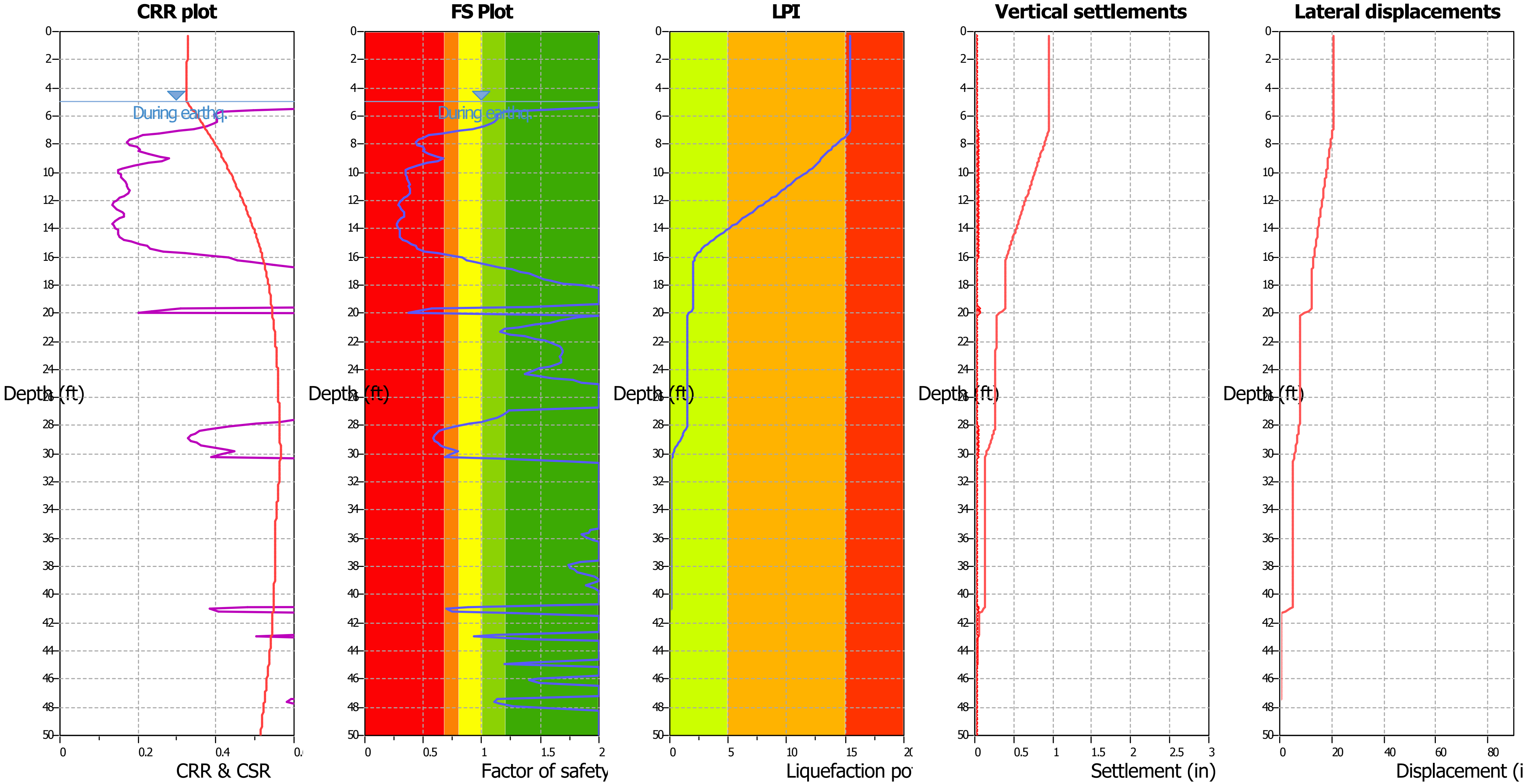
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

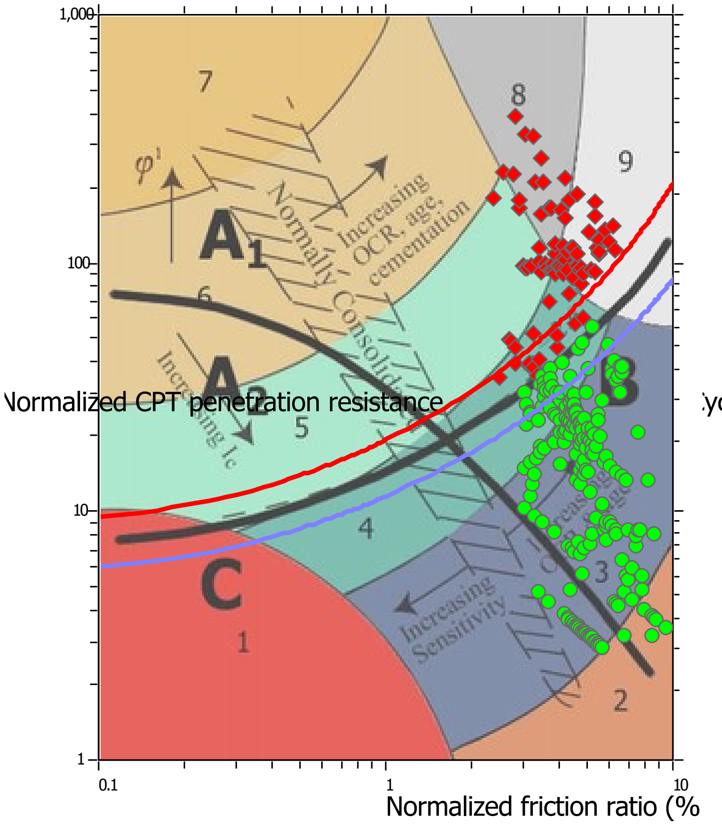
F.S. color scheme

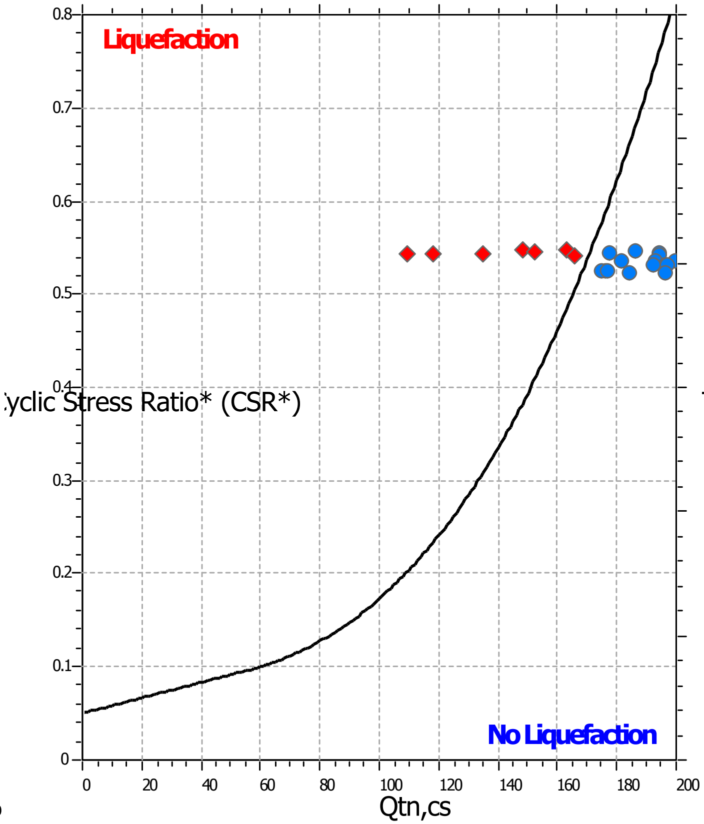
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

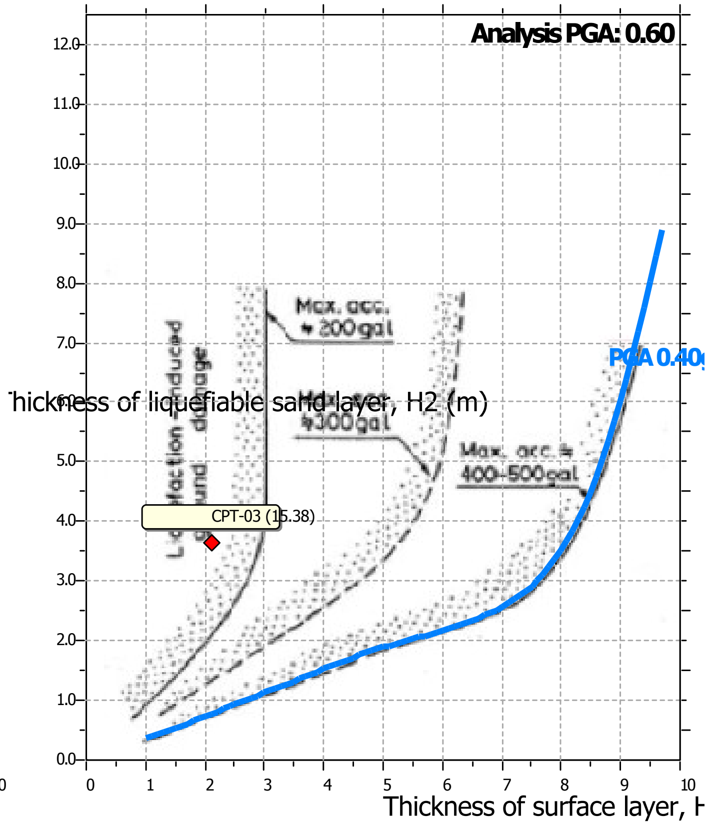
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



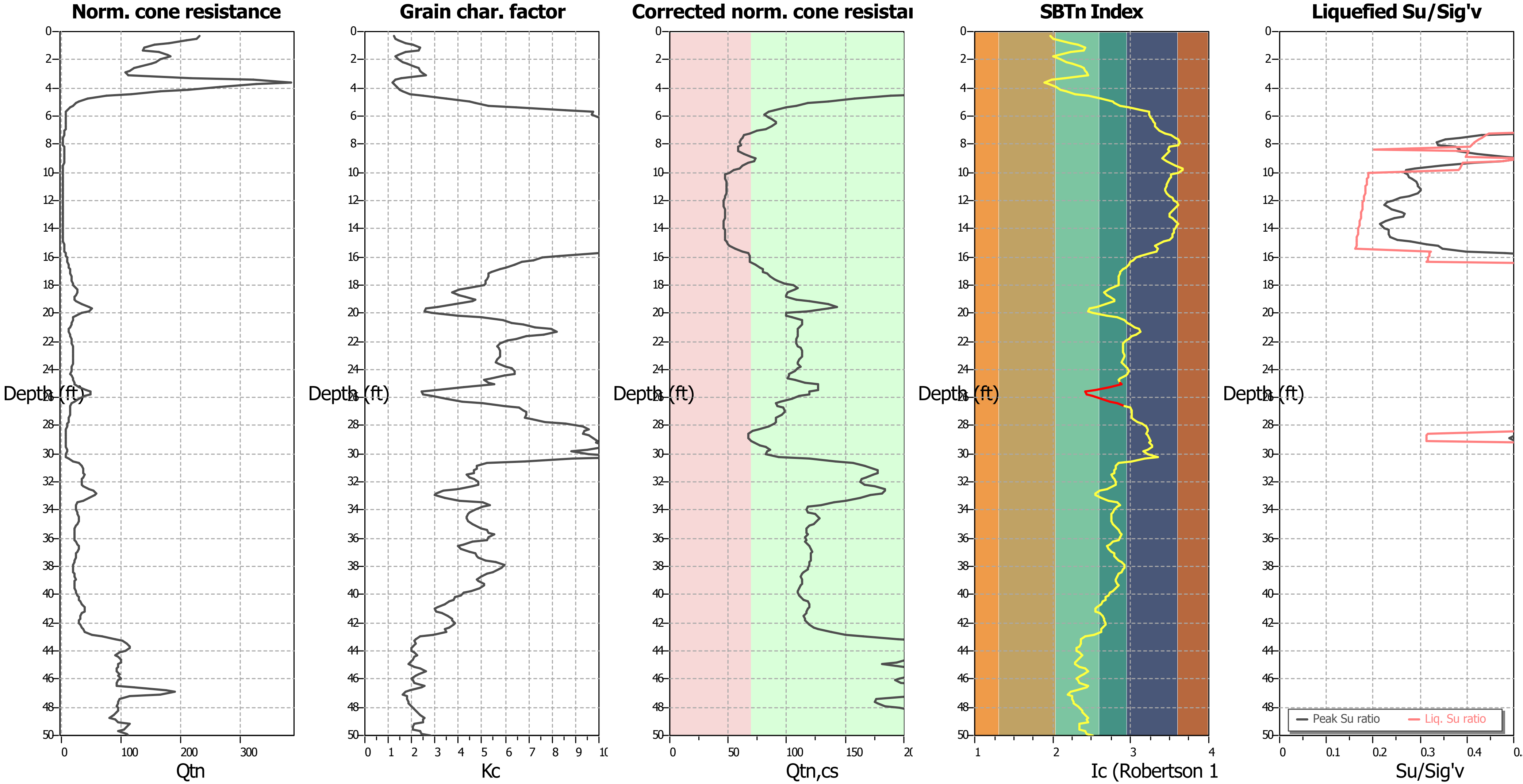




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

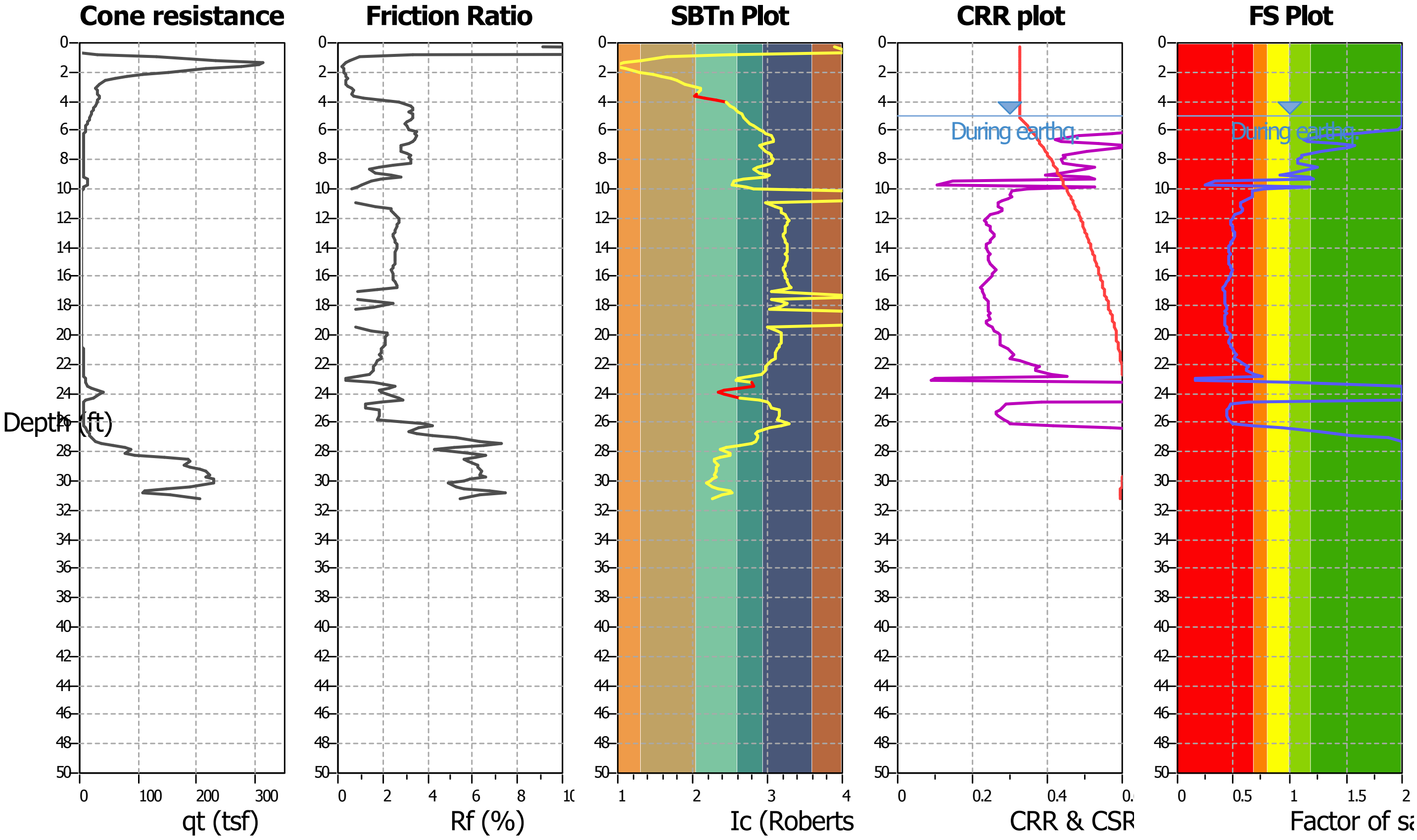
LIQUEFACTION ANALYSIS REPORT

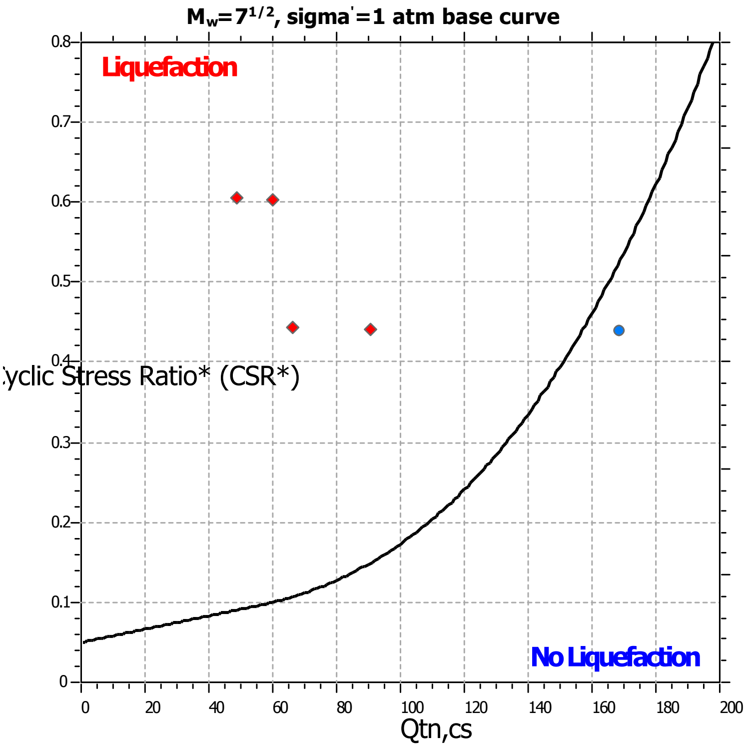
Project title : East D Street **Location : Petaluma, CA**

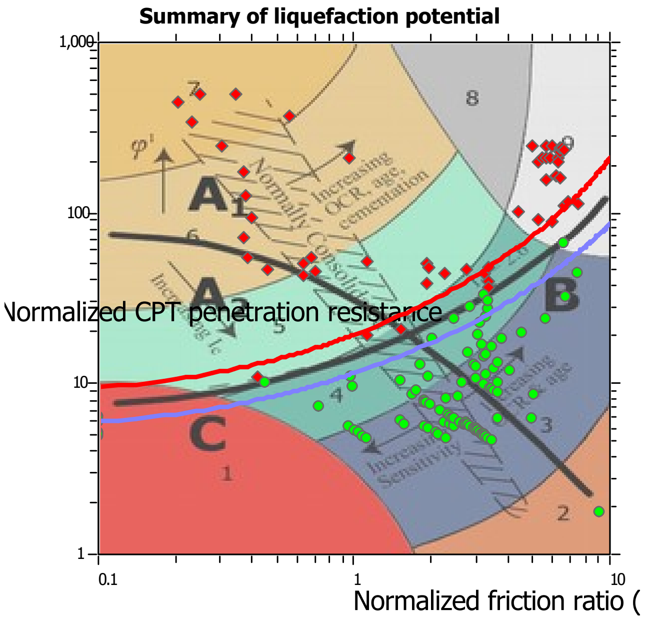
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

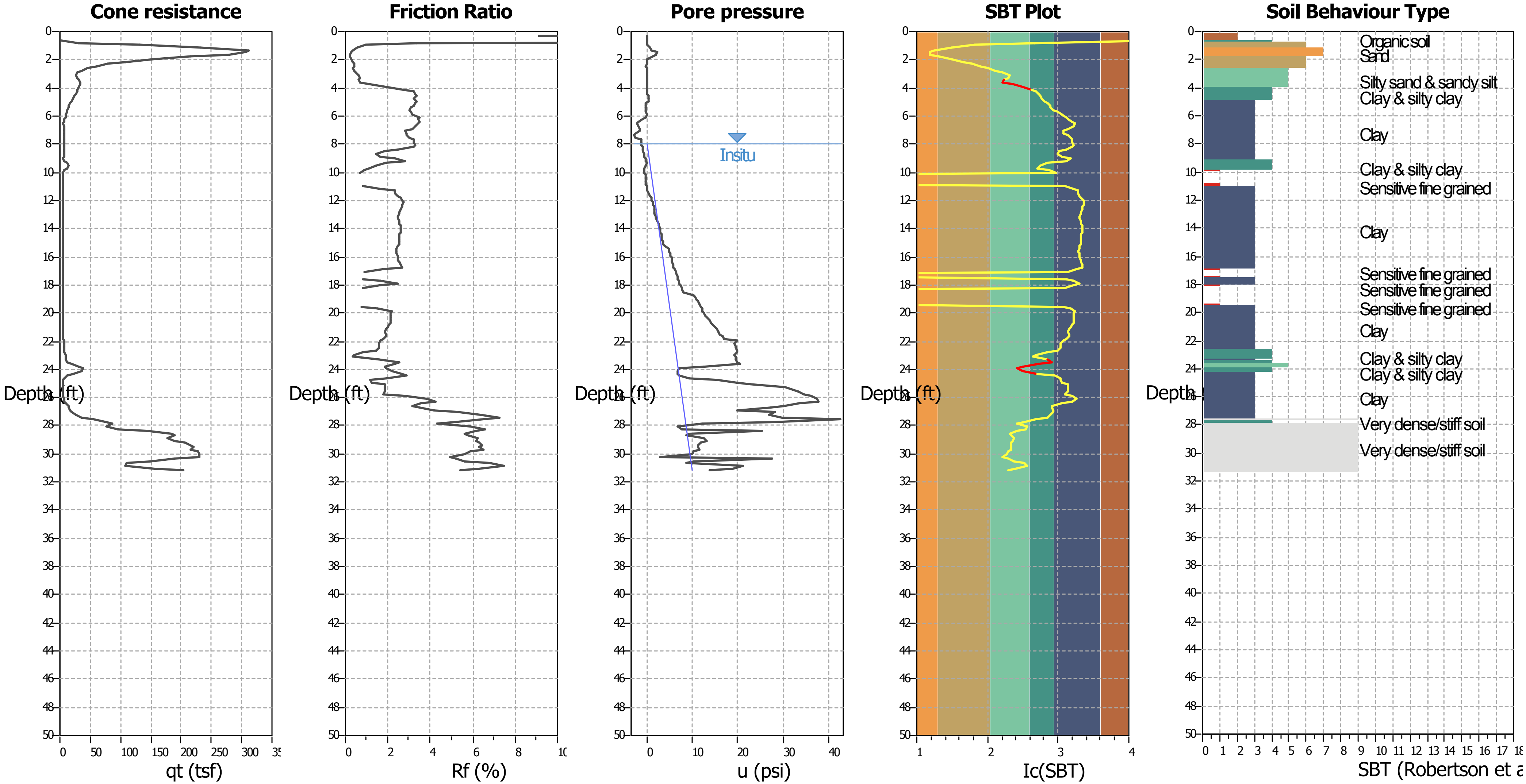






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



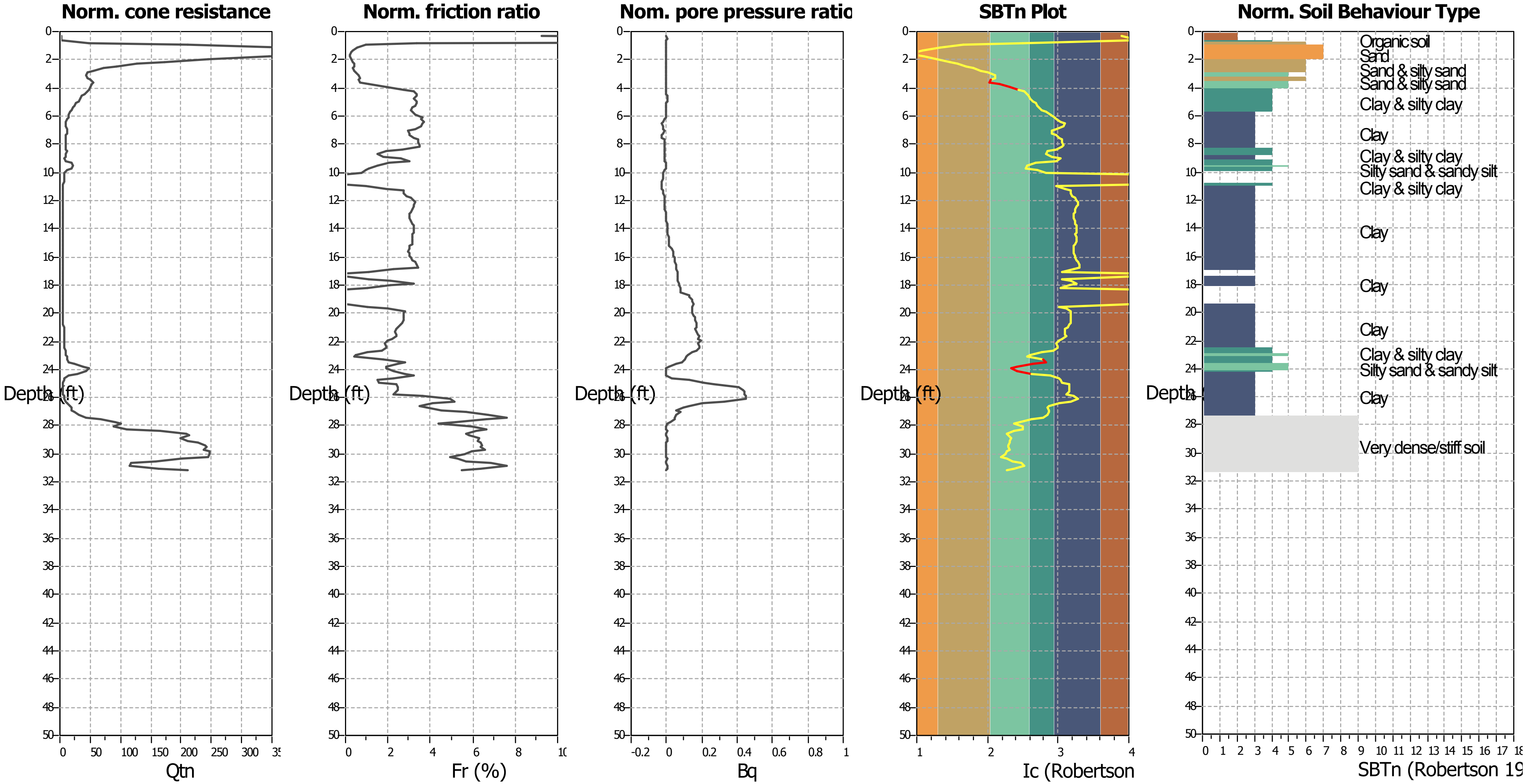
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



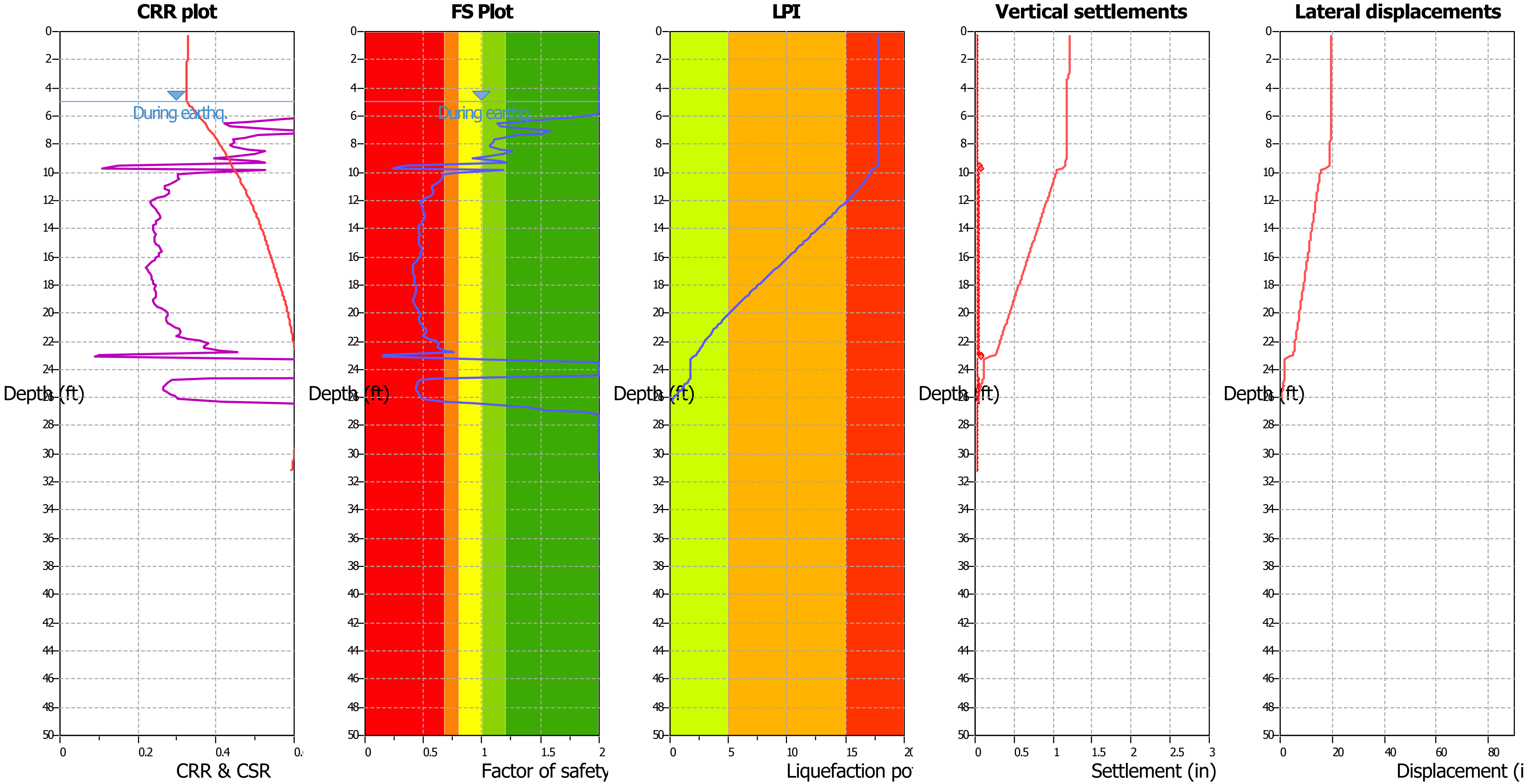
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

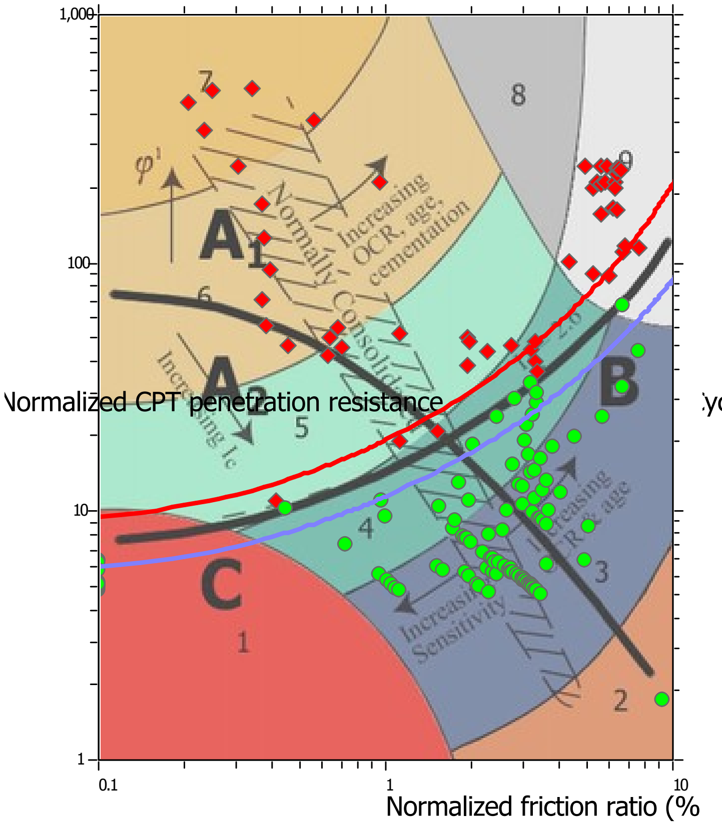
F.S. color scheme

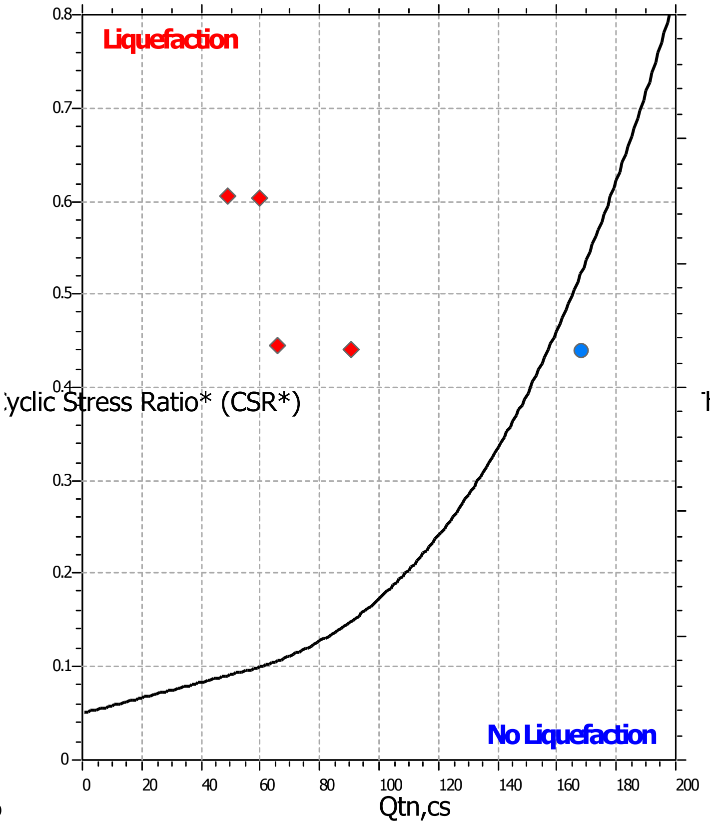
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

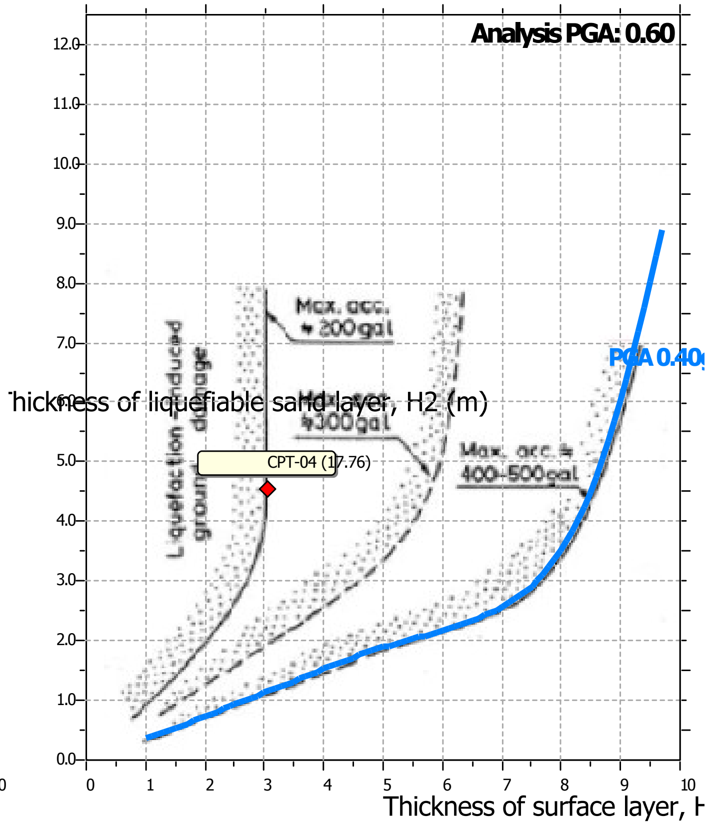
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



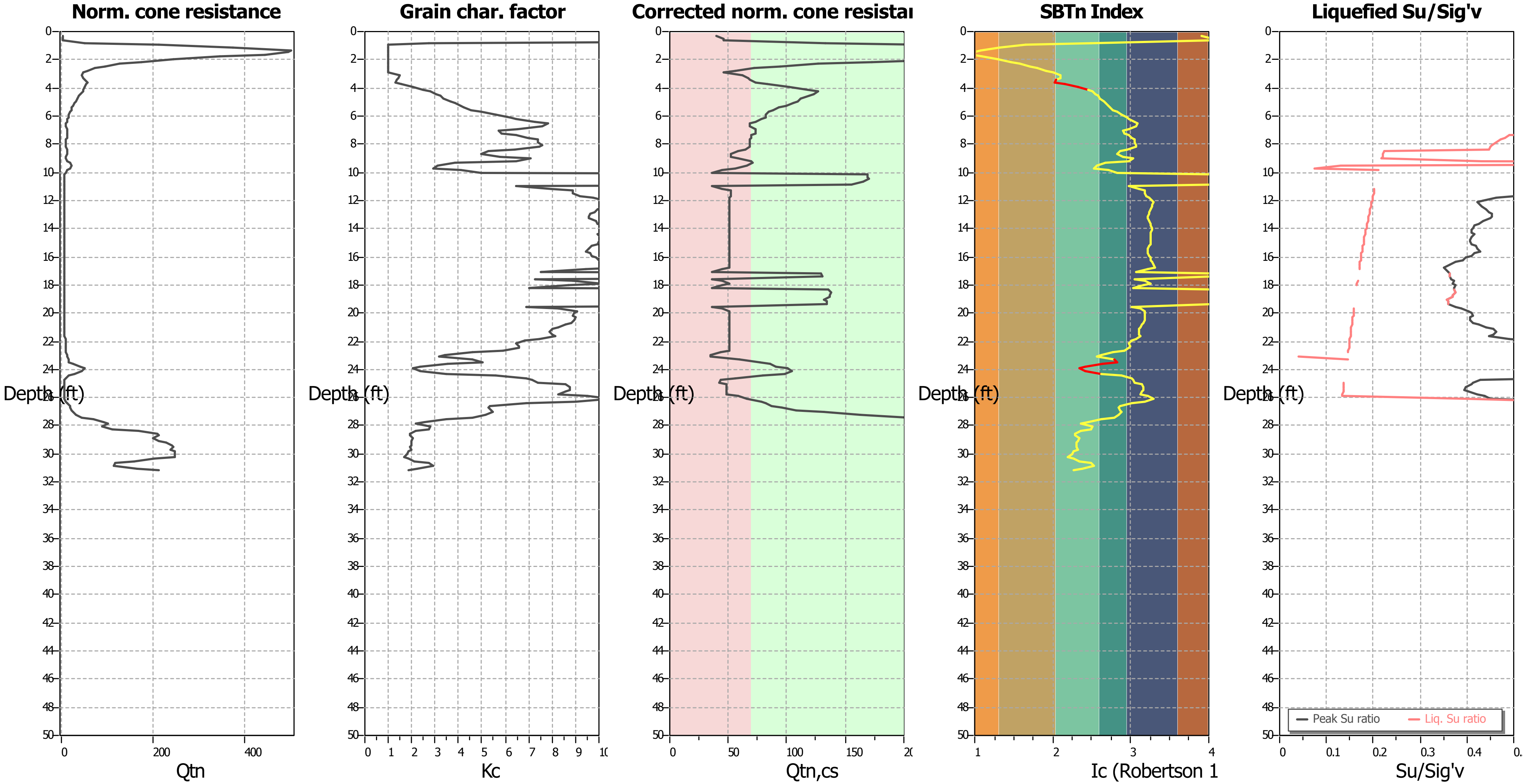




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Berlogar, Stevens & Associates, Inc.
Pleasanton, California

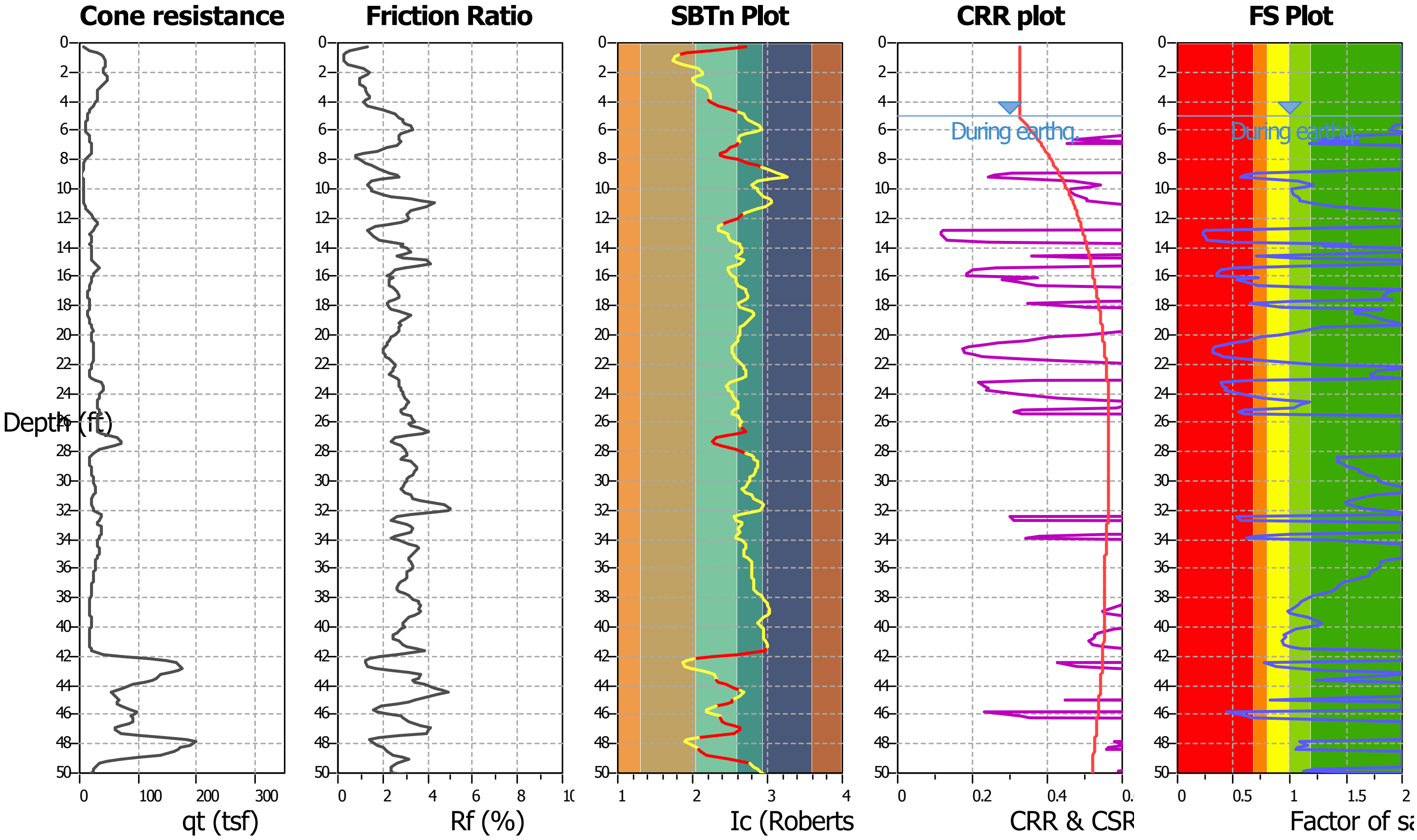
LIQUEFACTION ANALYSIS REPORT

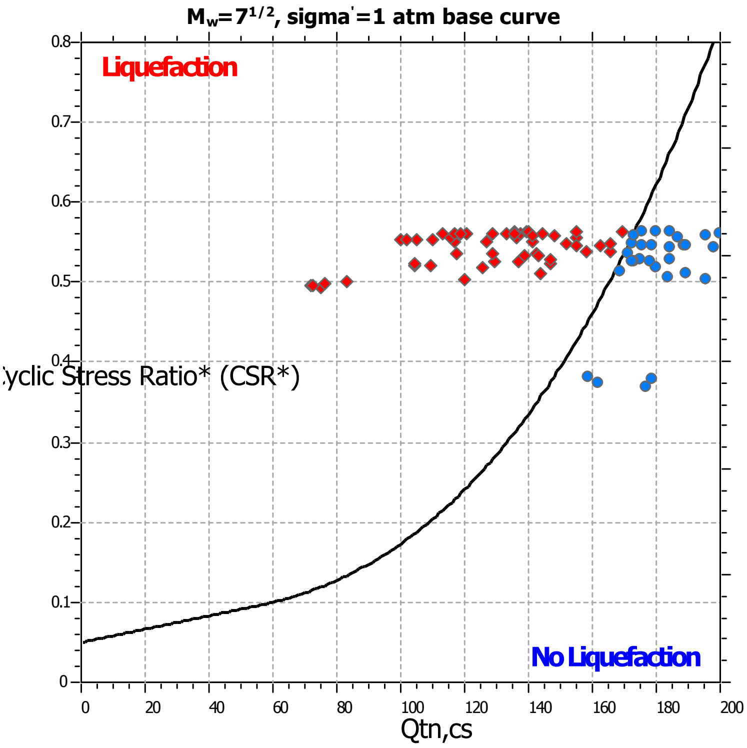
Project title : East D Street **Location : Petaluma, CA**

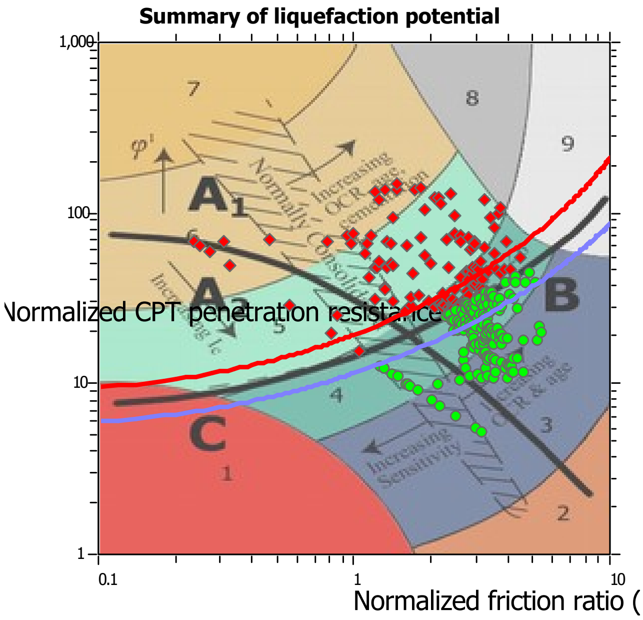
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

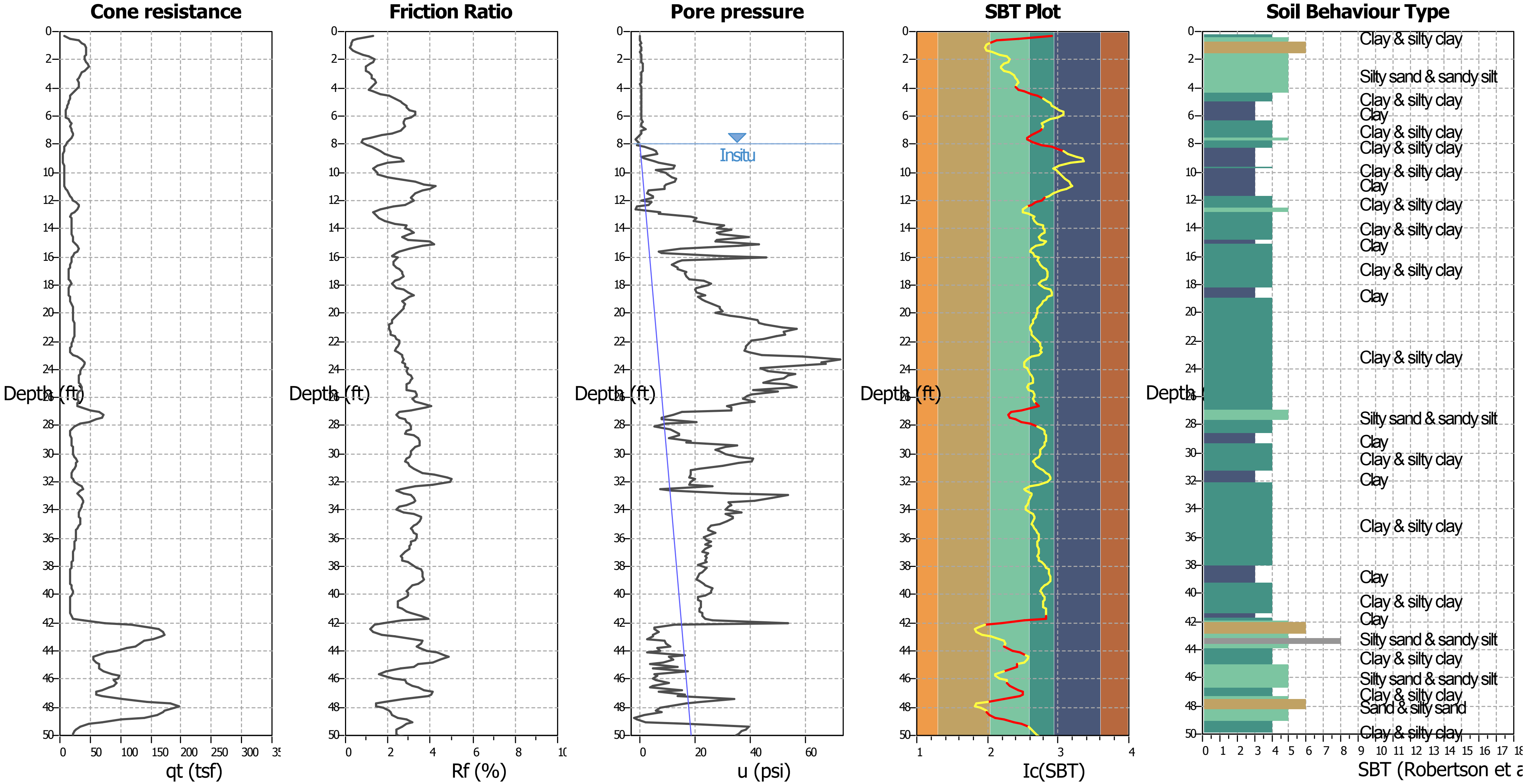






Zone A_1 : Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



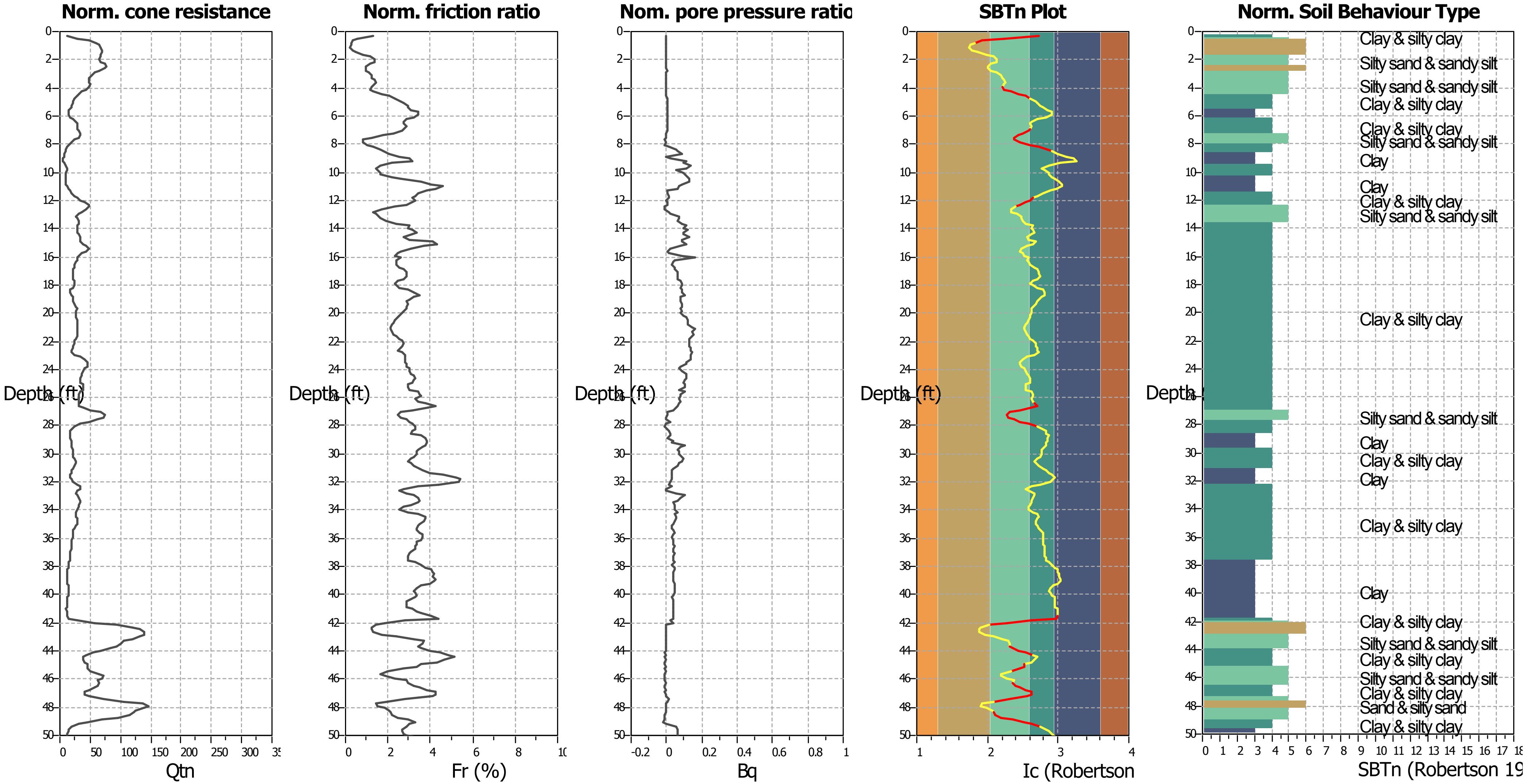
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A		
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils		
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes		
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft		

SBT legend

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

CPT basic interpretation plots (normalized)



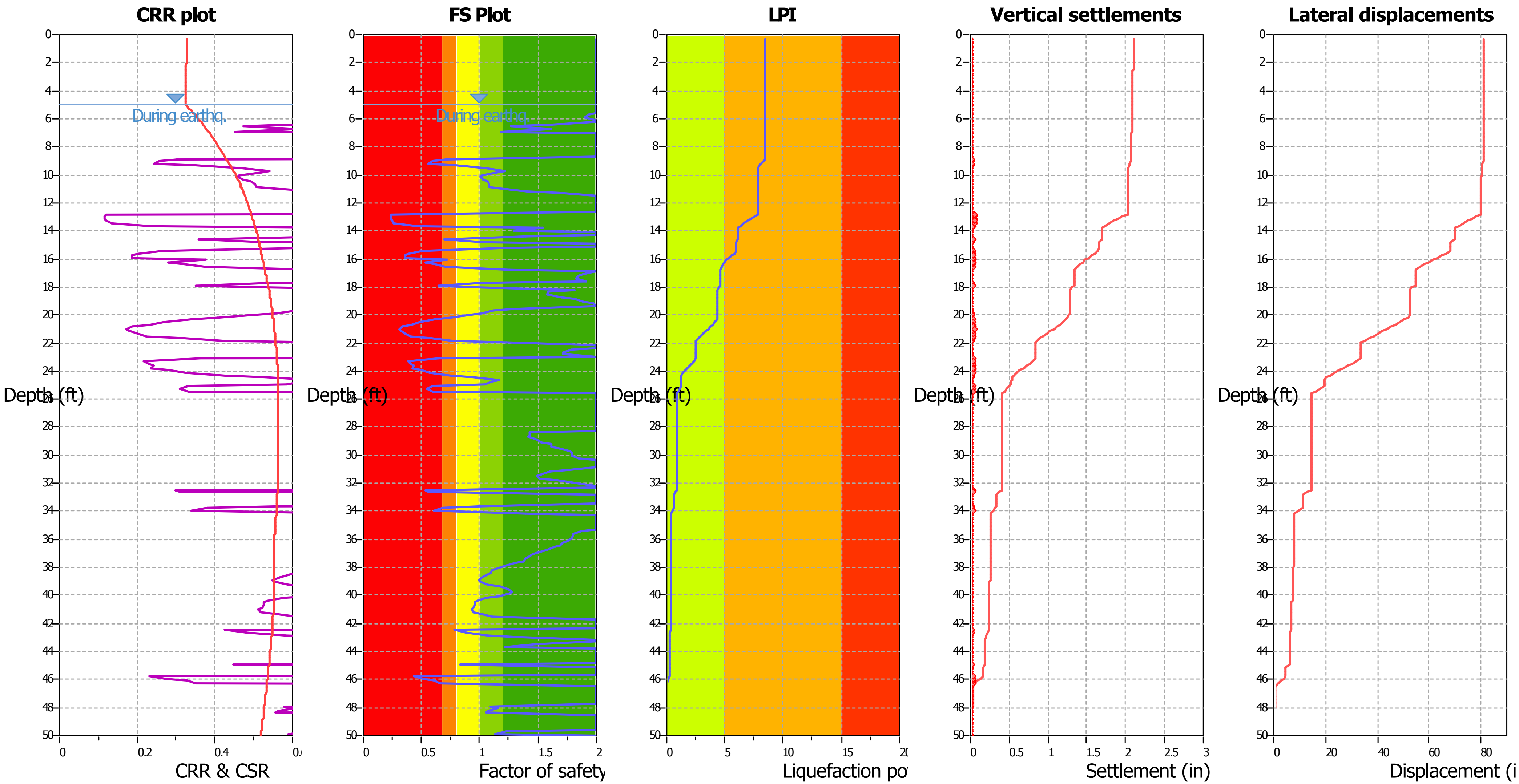
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

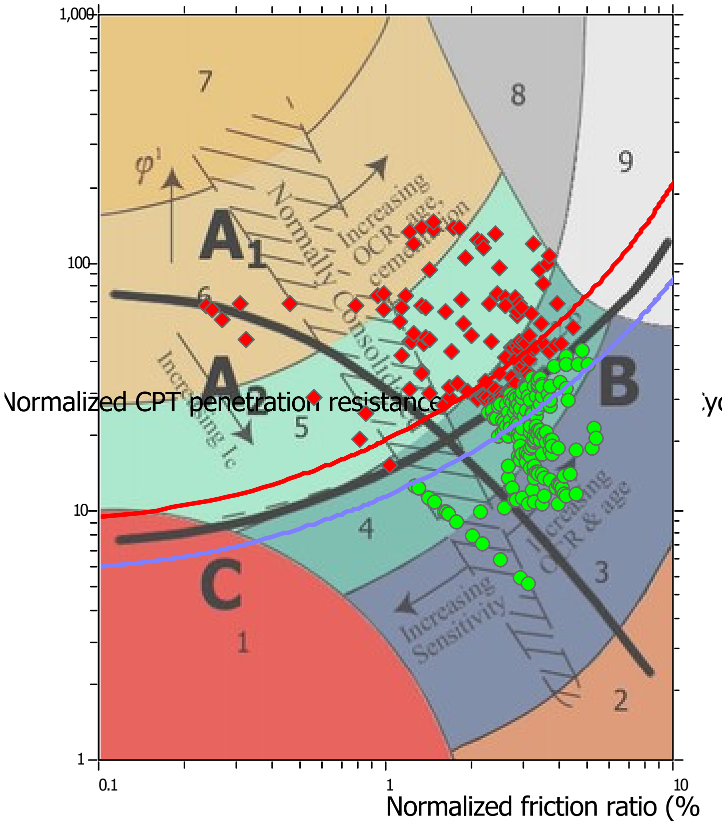
F.S. color scheme

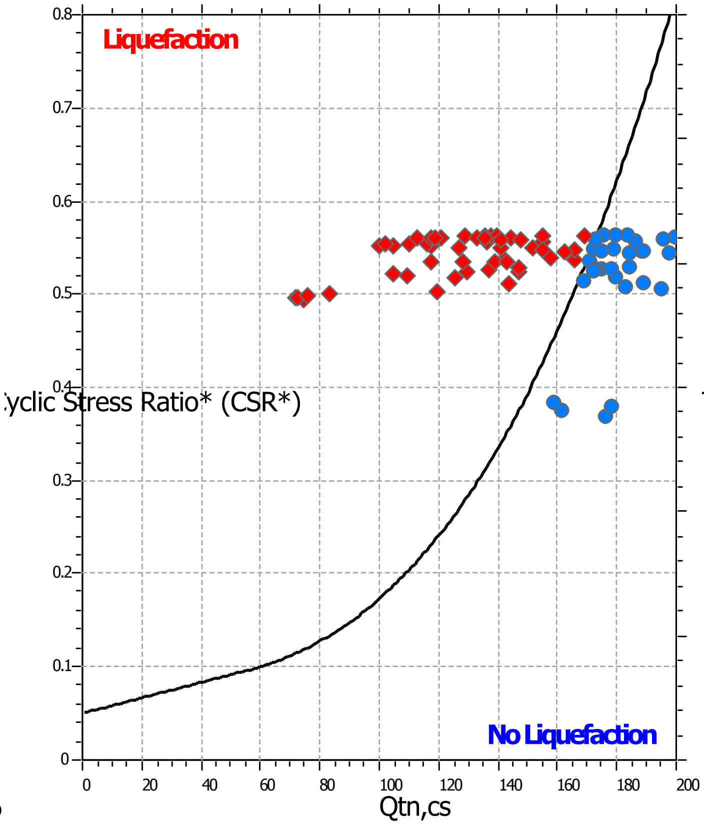
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

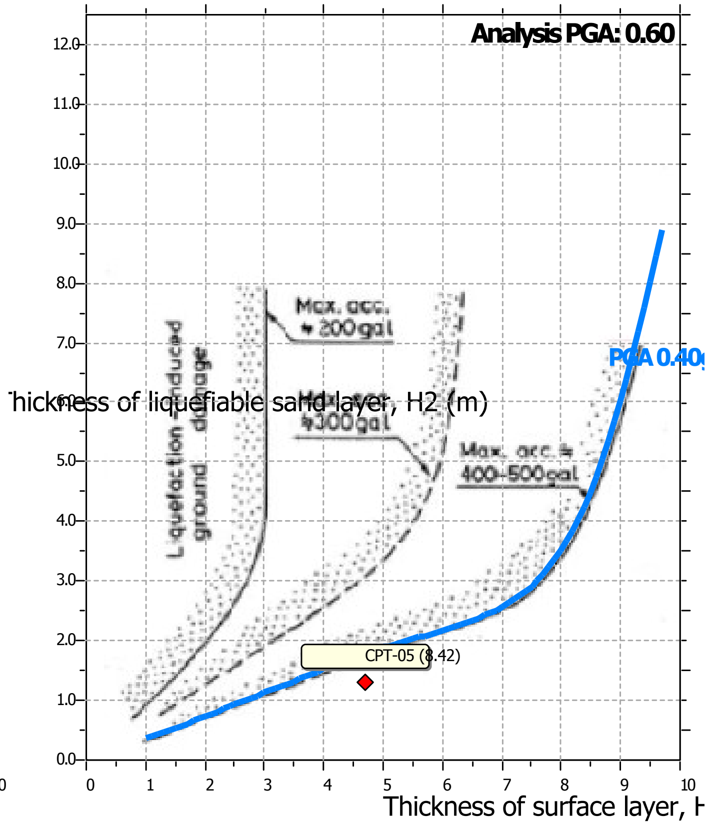
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



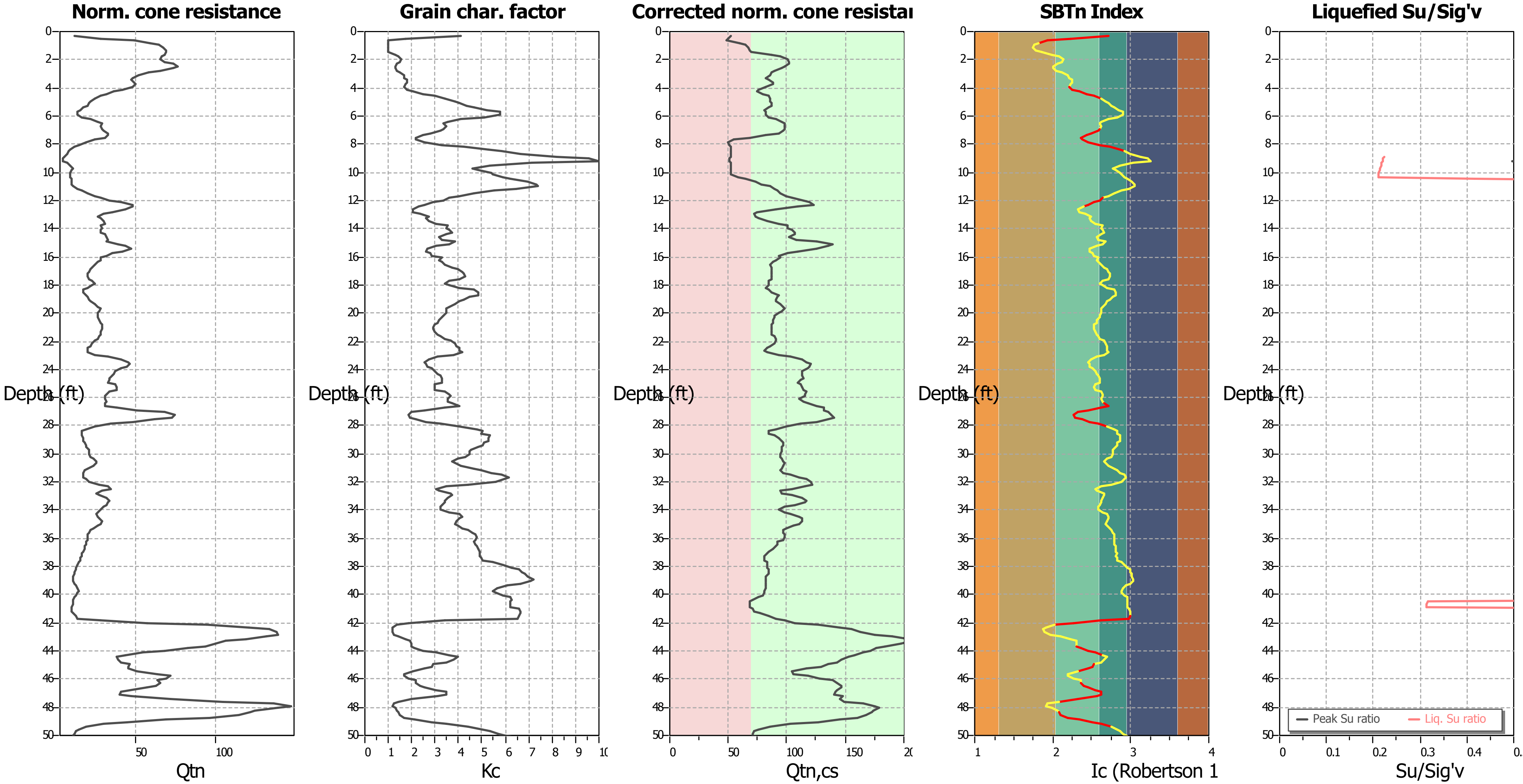




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Berlogar, Stevens & Associates, Inc.
Pleasanton, California

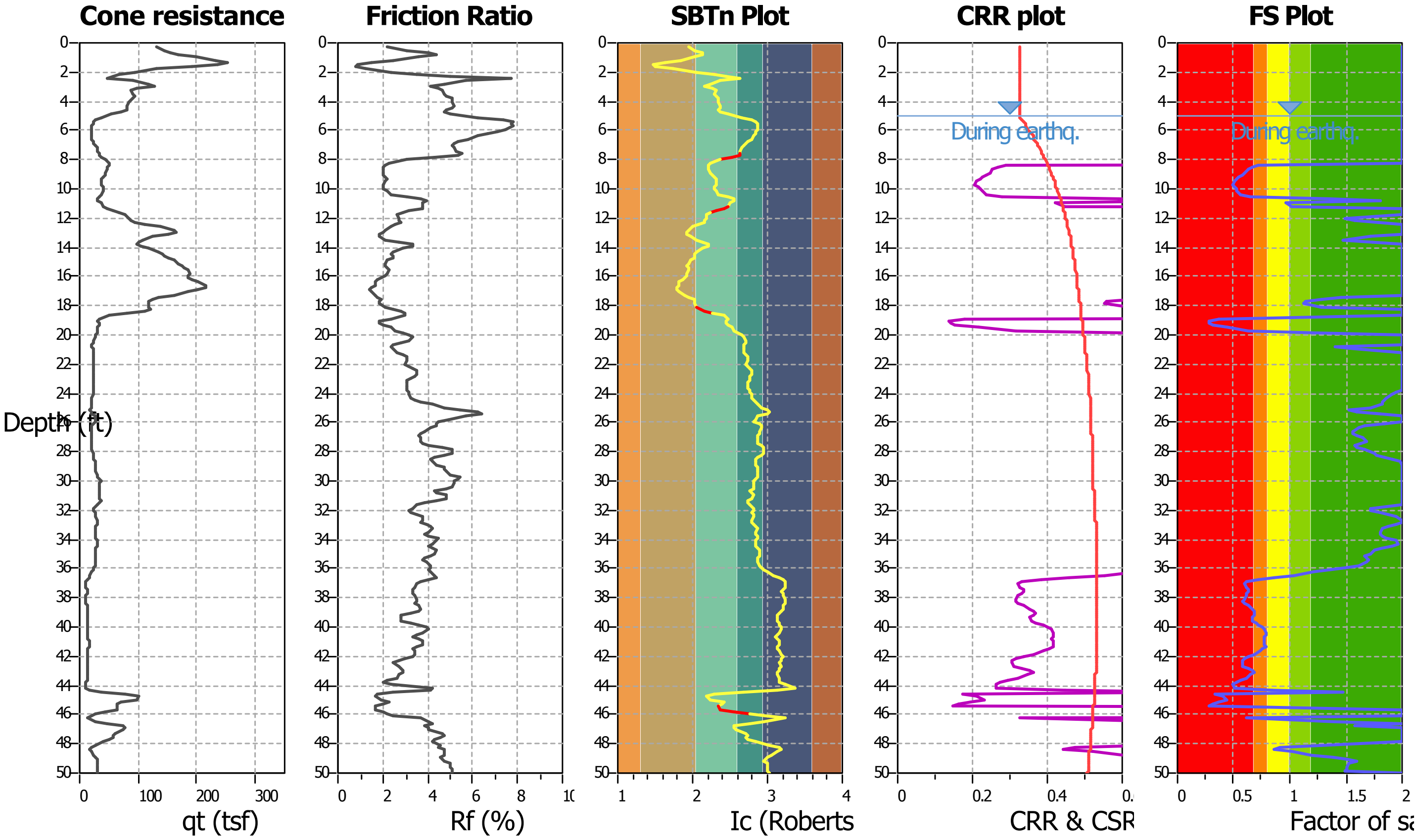
LIQUEFACTION ANALYSIS REPORT

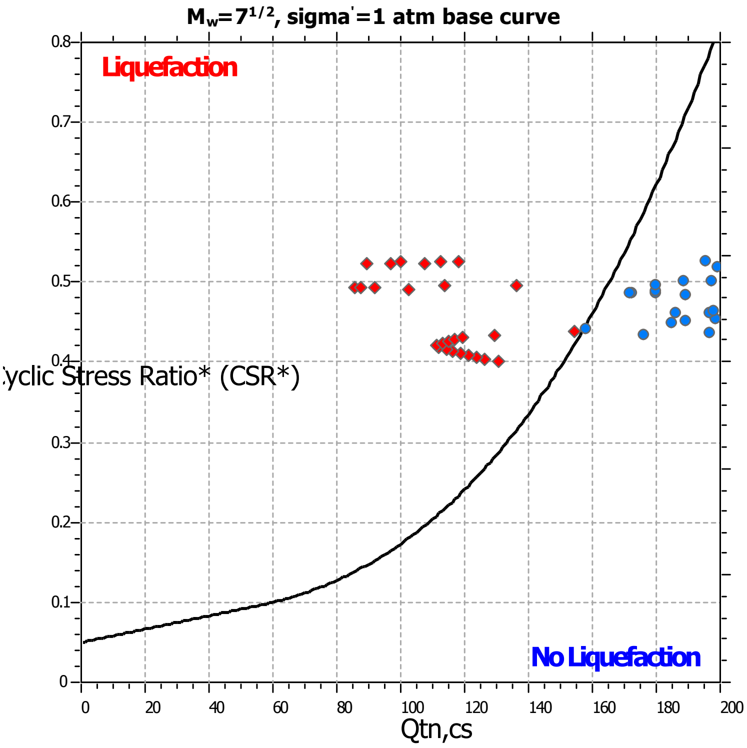
Project title : East D Street **Location : Petaluma, CA**

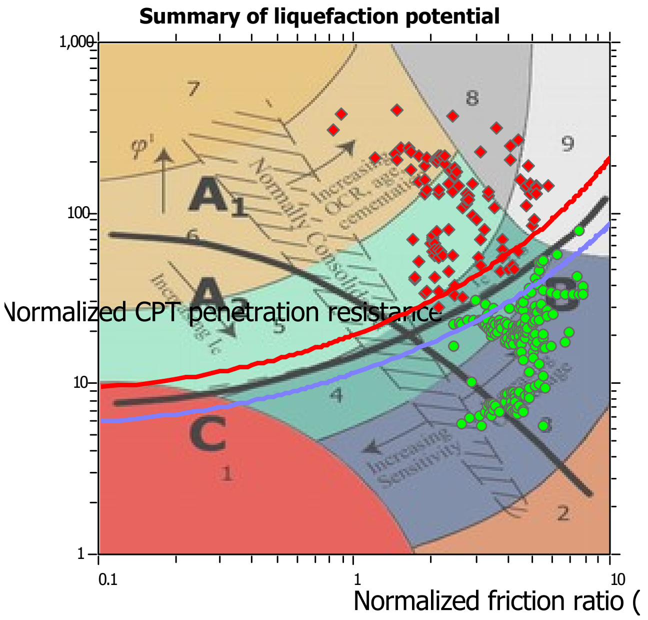
CPT file : CPT-06

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

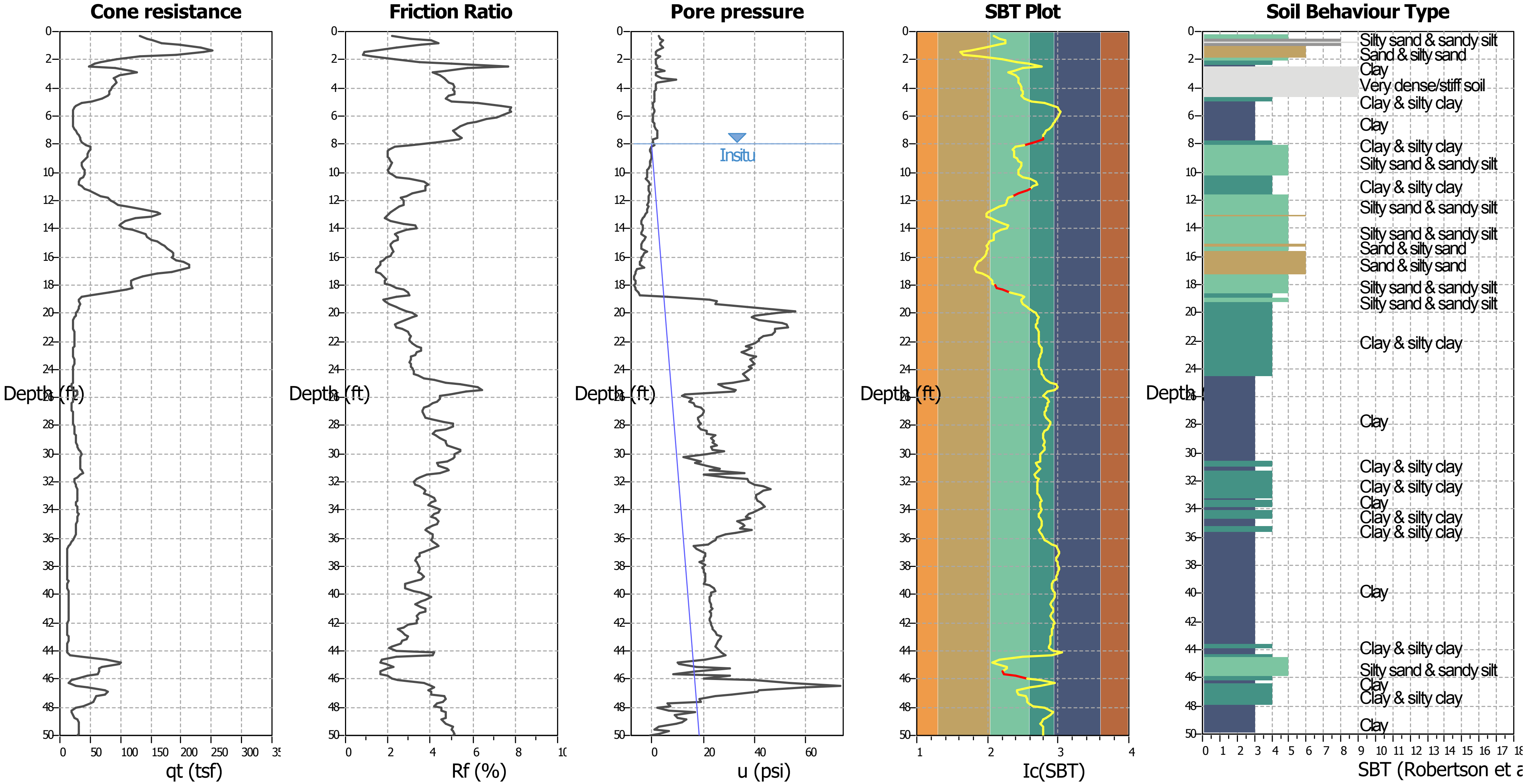






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



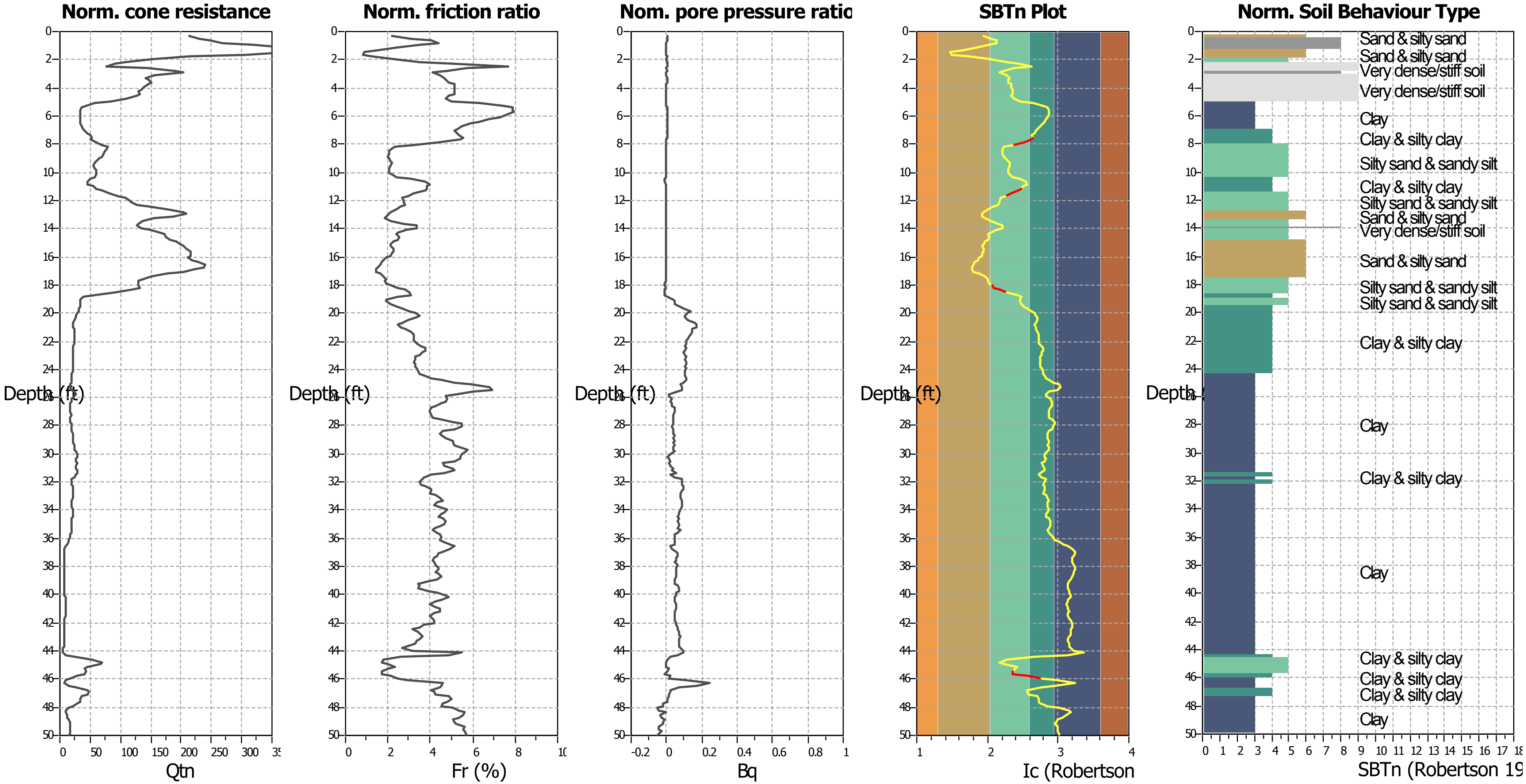
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



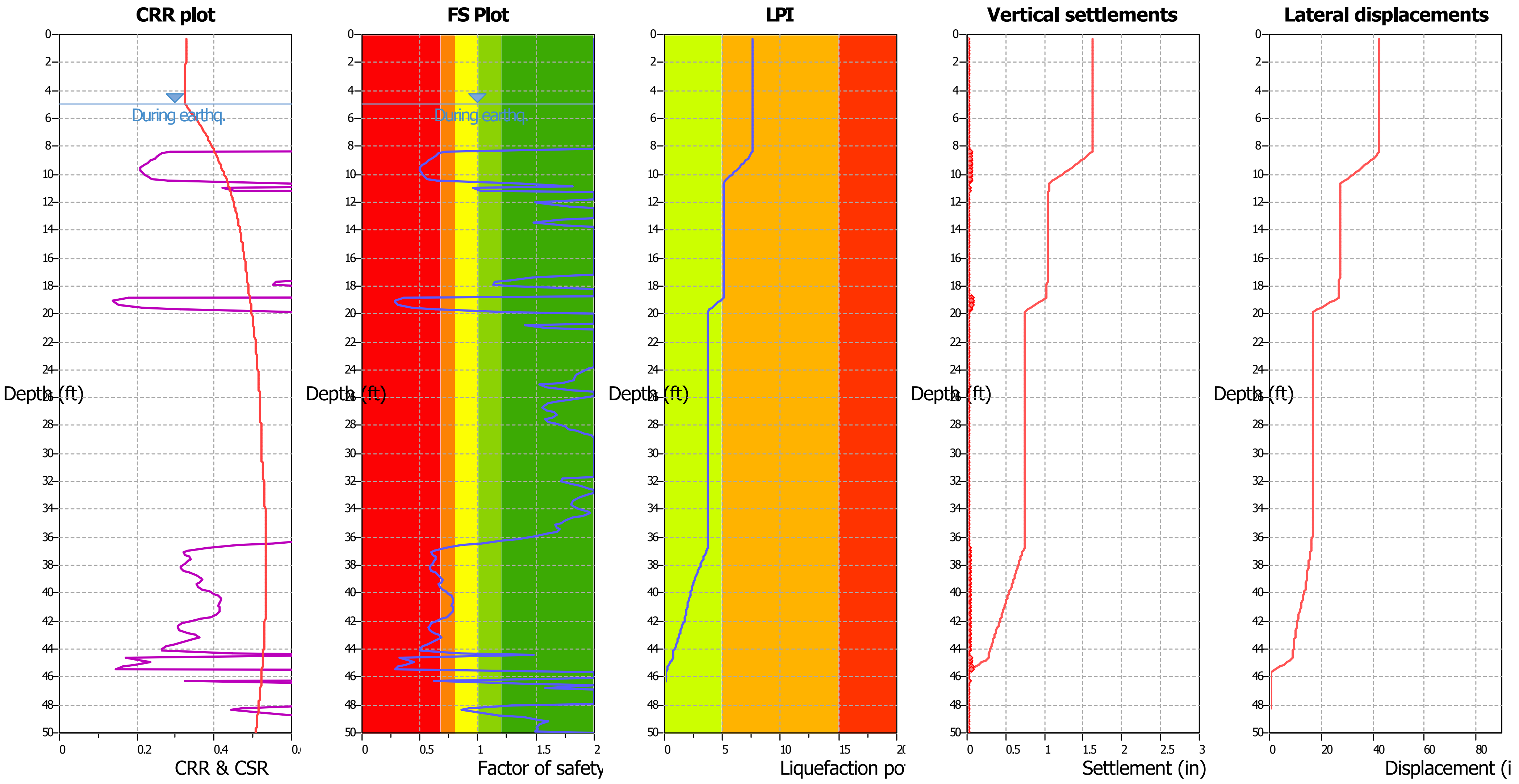
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

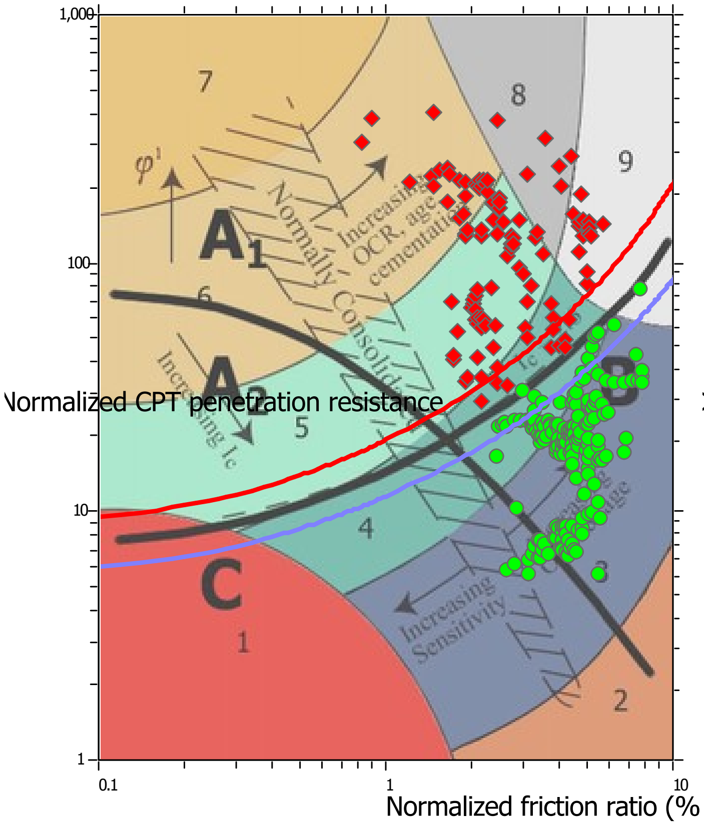
F.S. color scheme

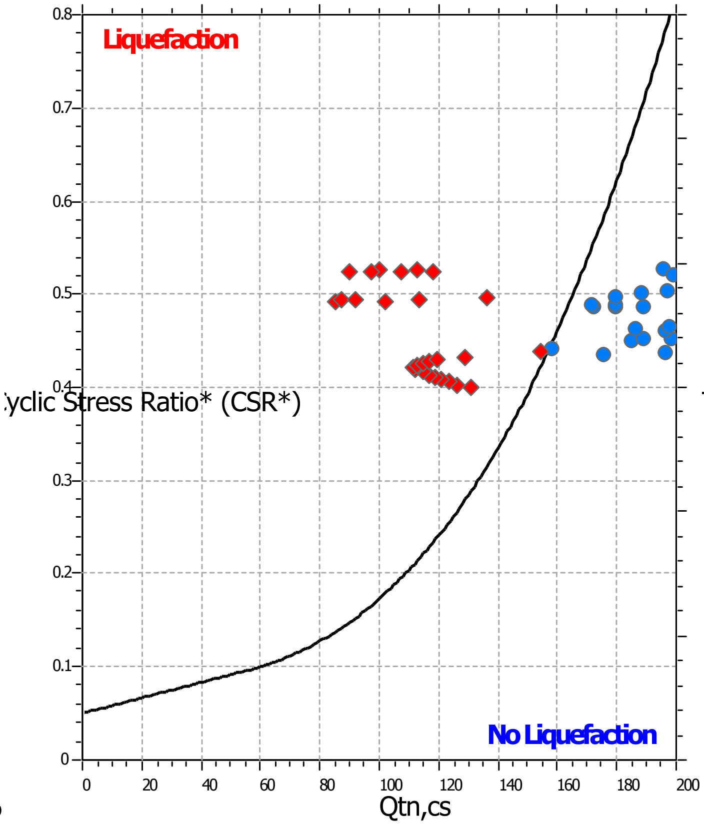
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

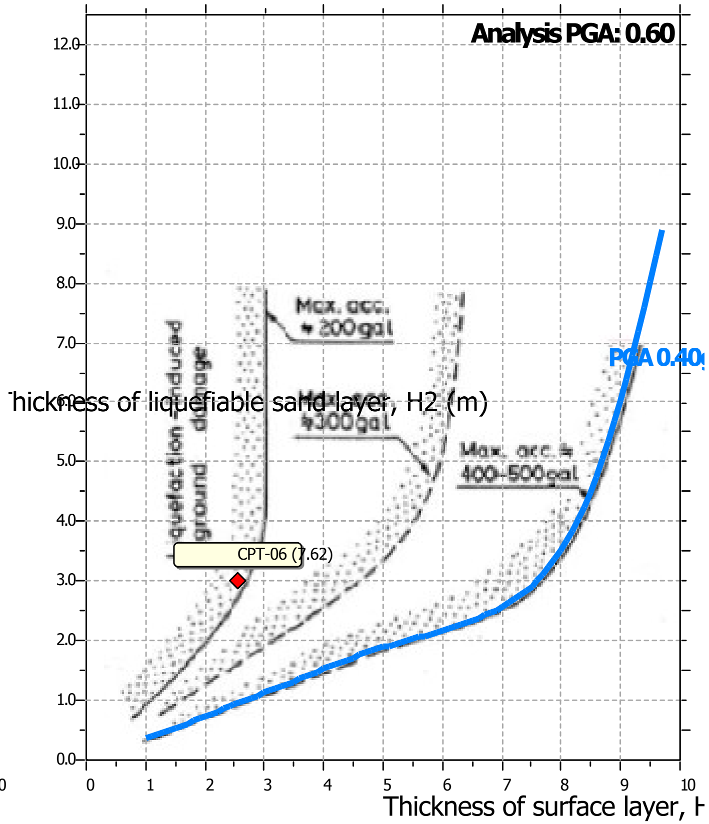
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



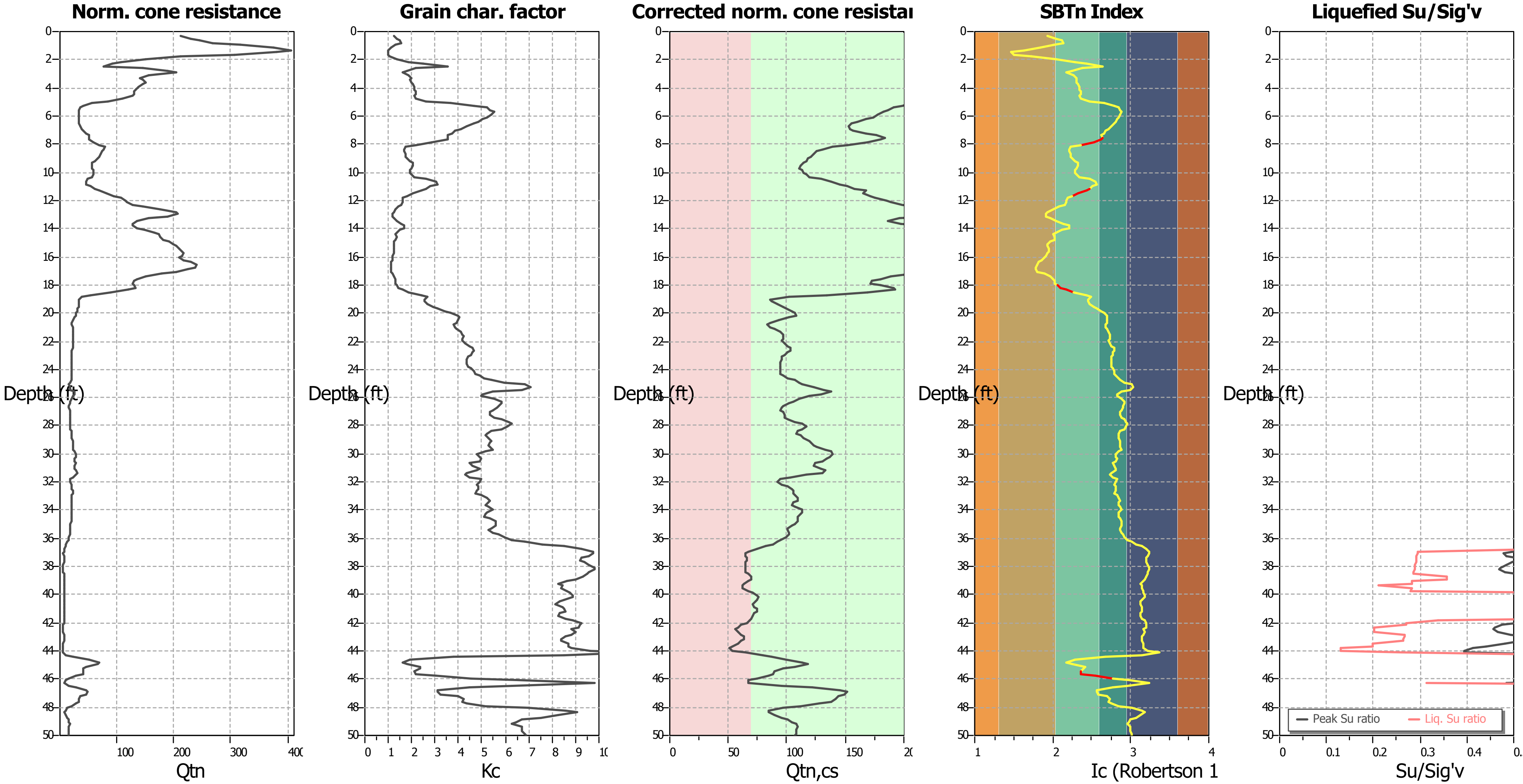




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

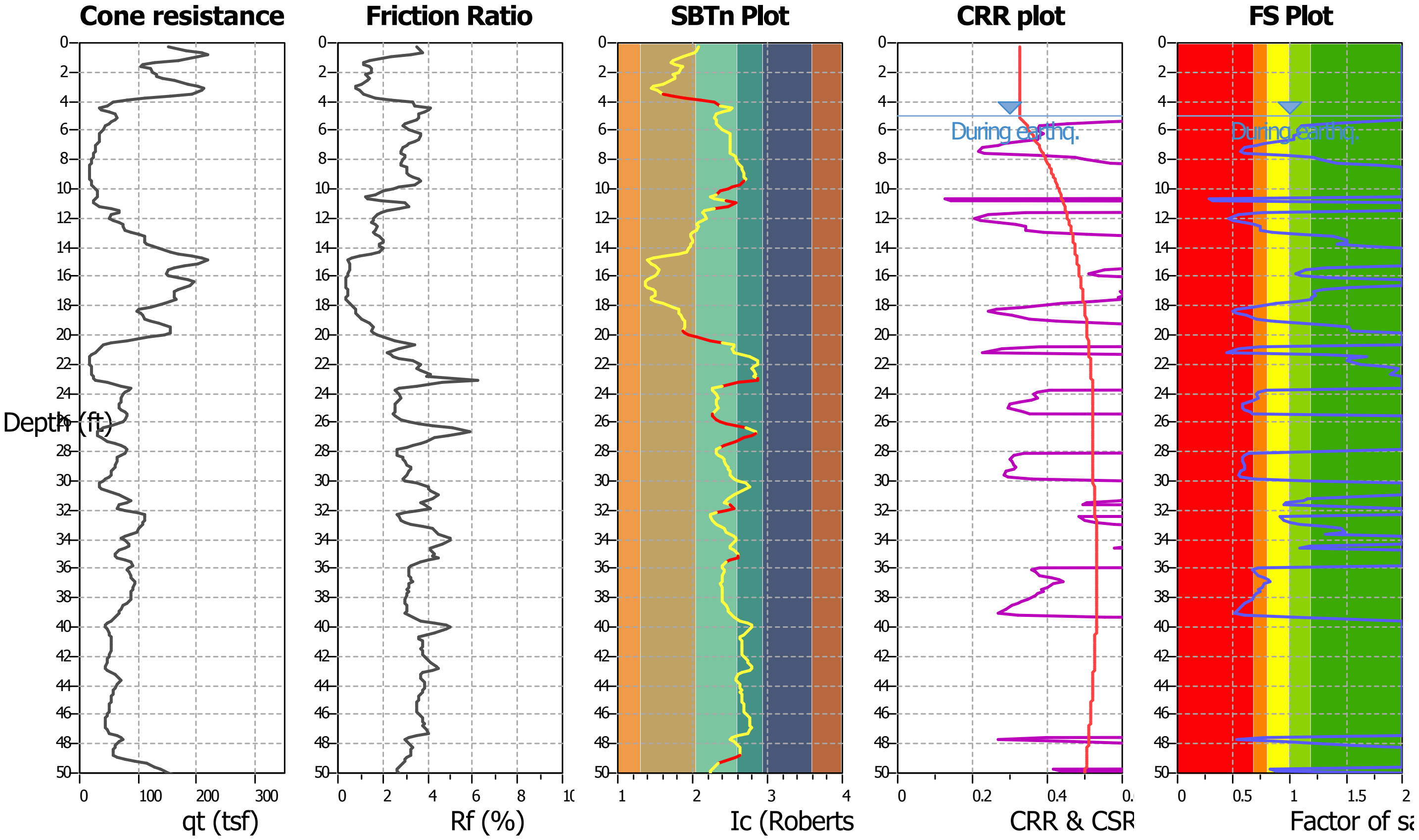
LIQUEFACTION ANALYSIS REPORT

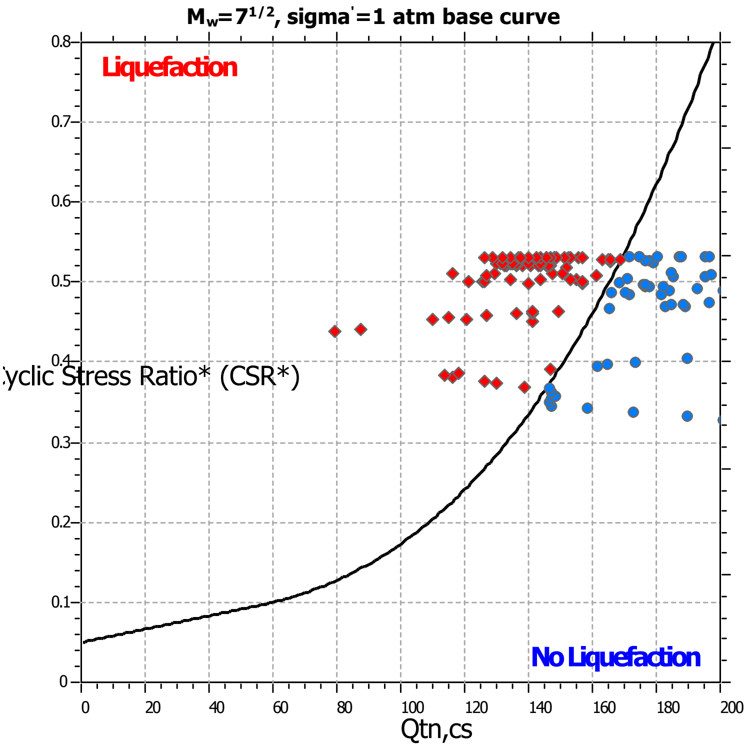
Project title : East D Street **Location : Petaluma, CA**

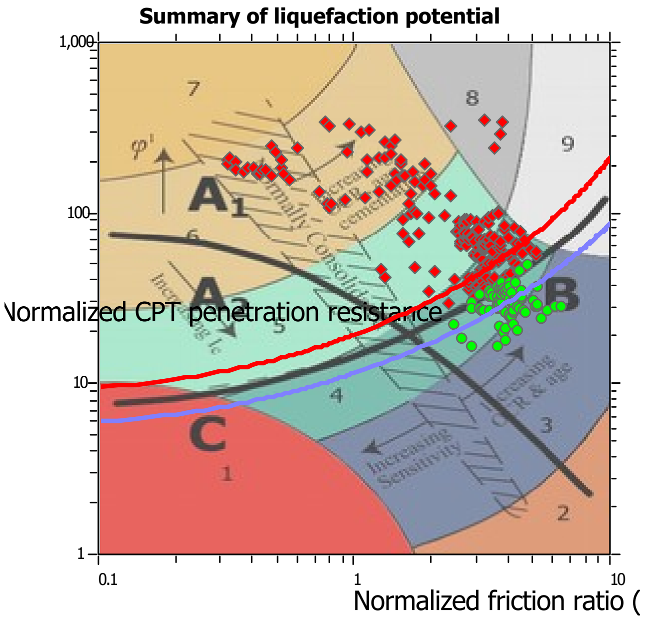
CPT file : CPT-07

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

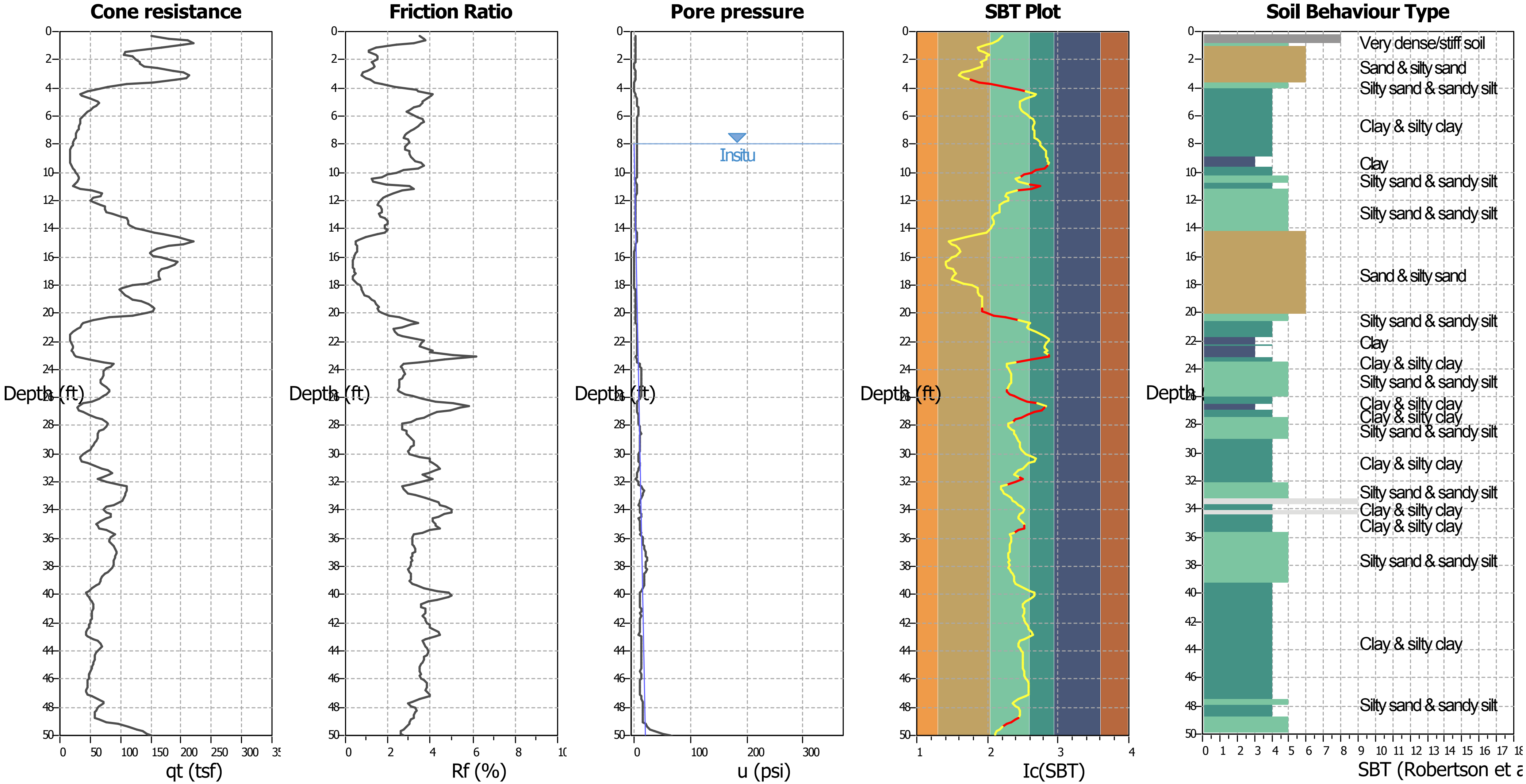






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



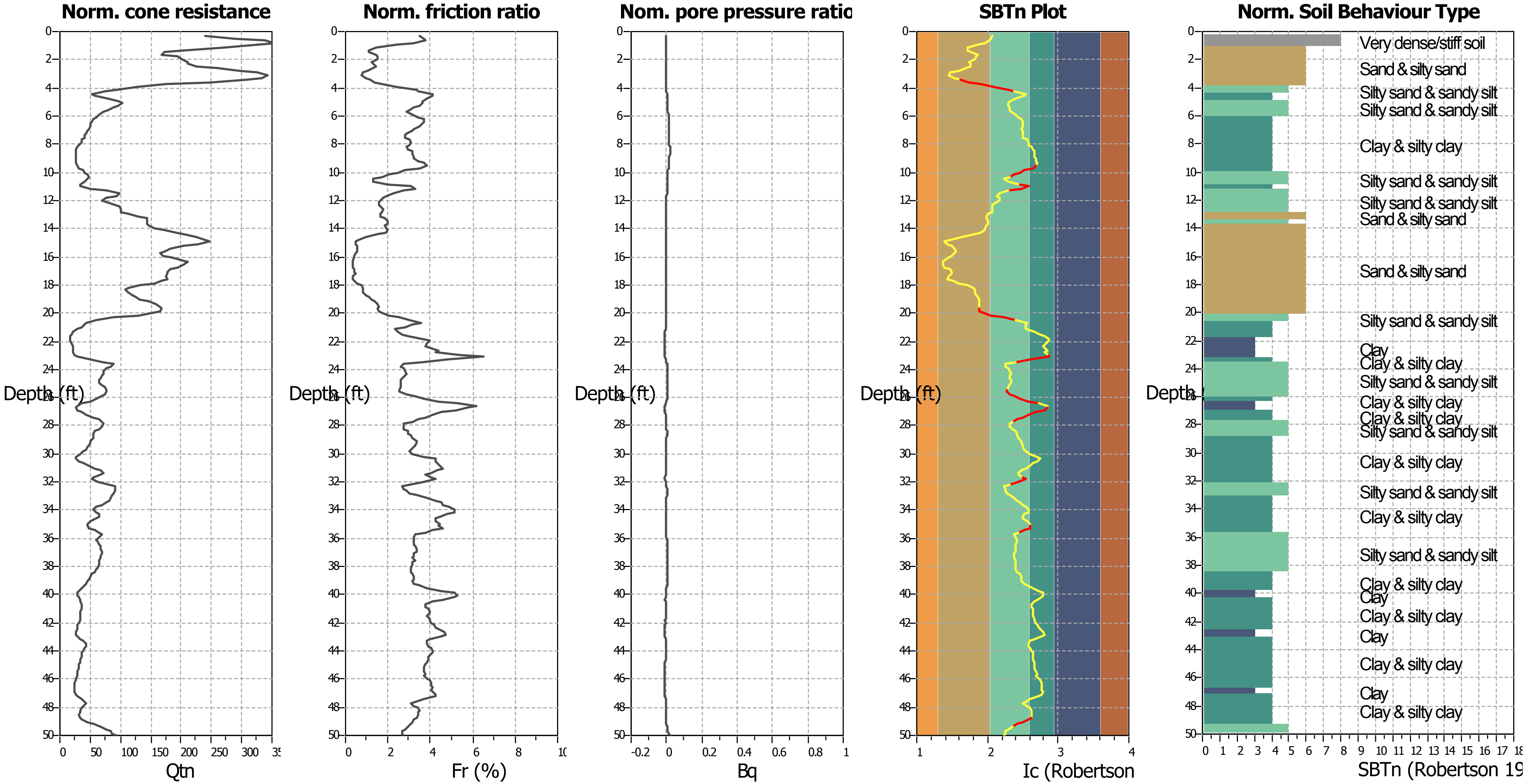
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



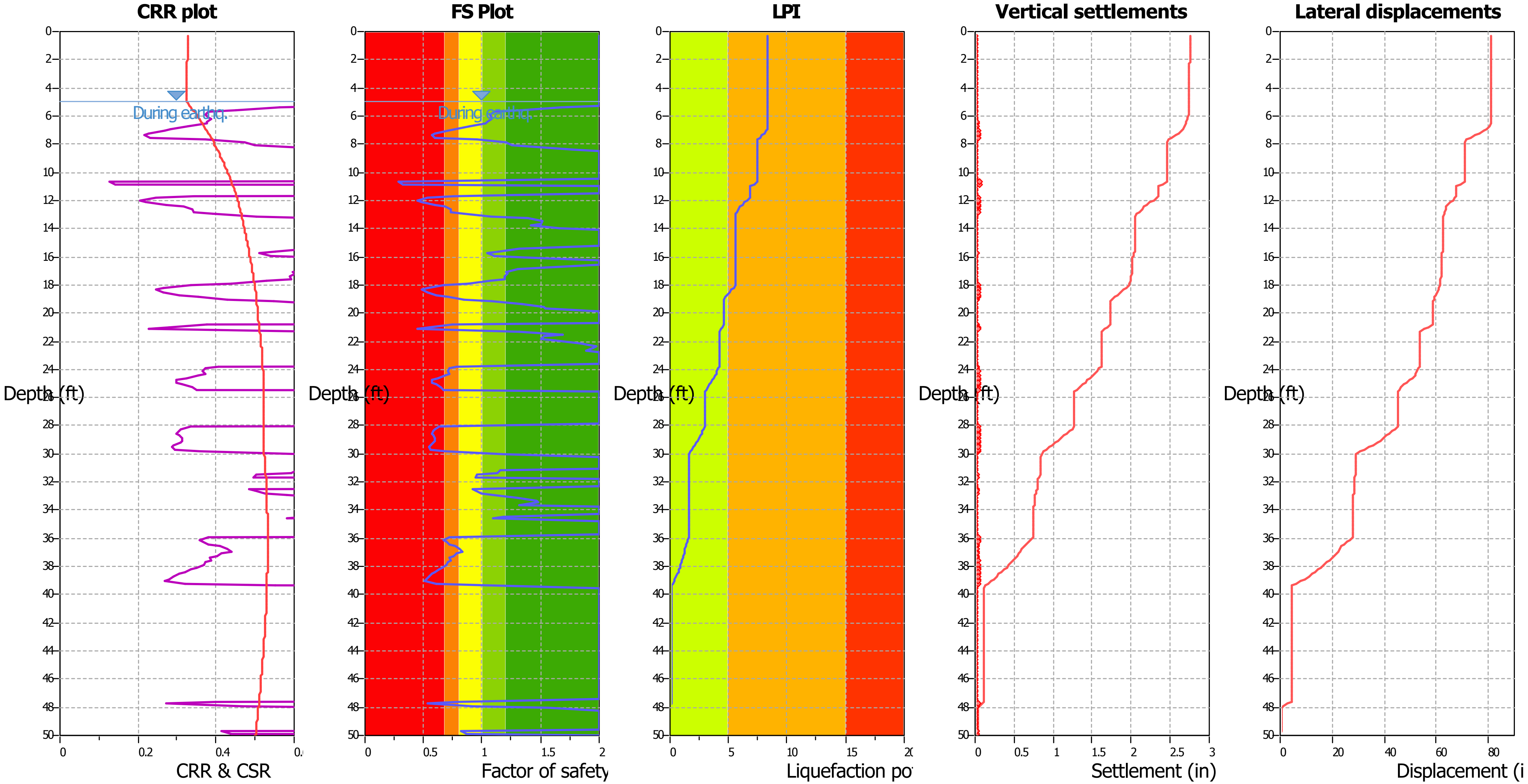
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

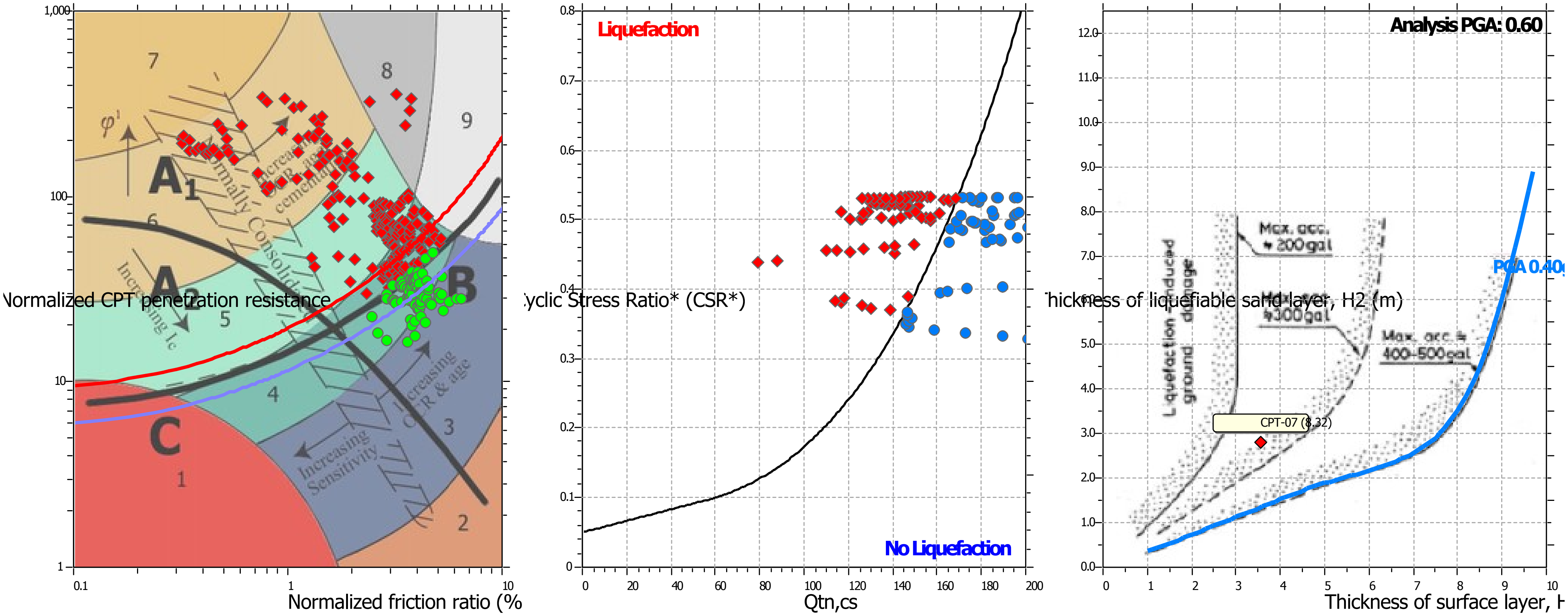
F.S. color scheme

- 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

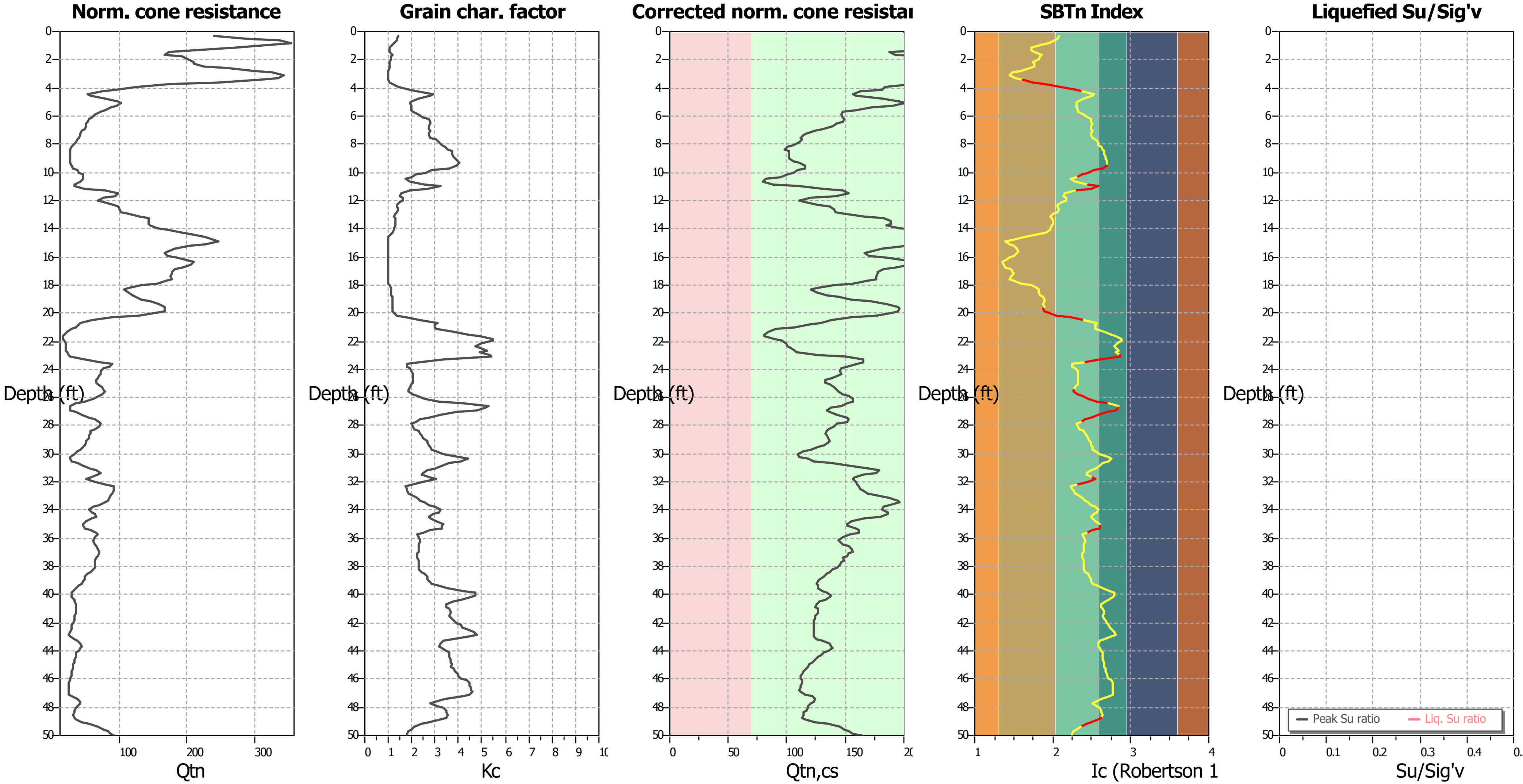
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

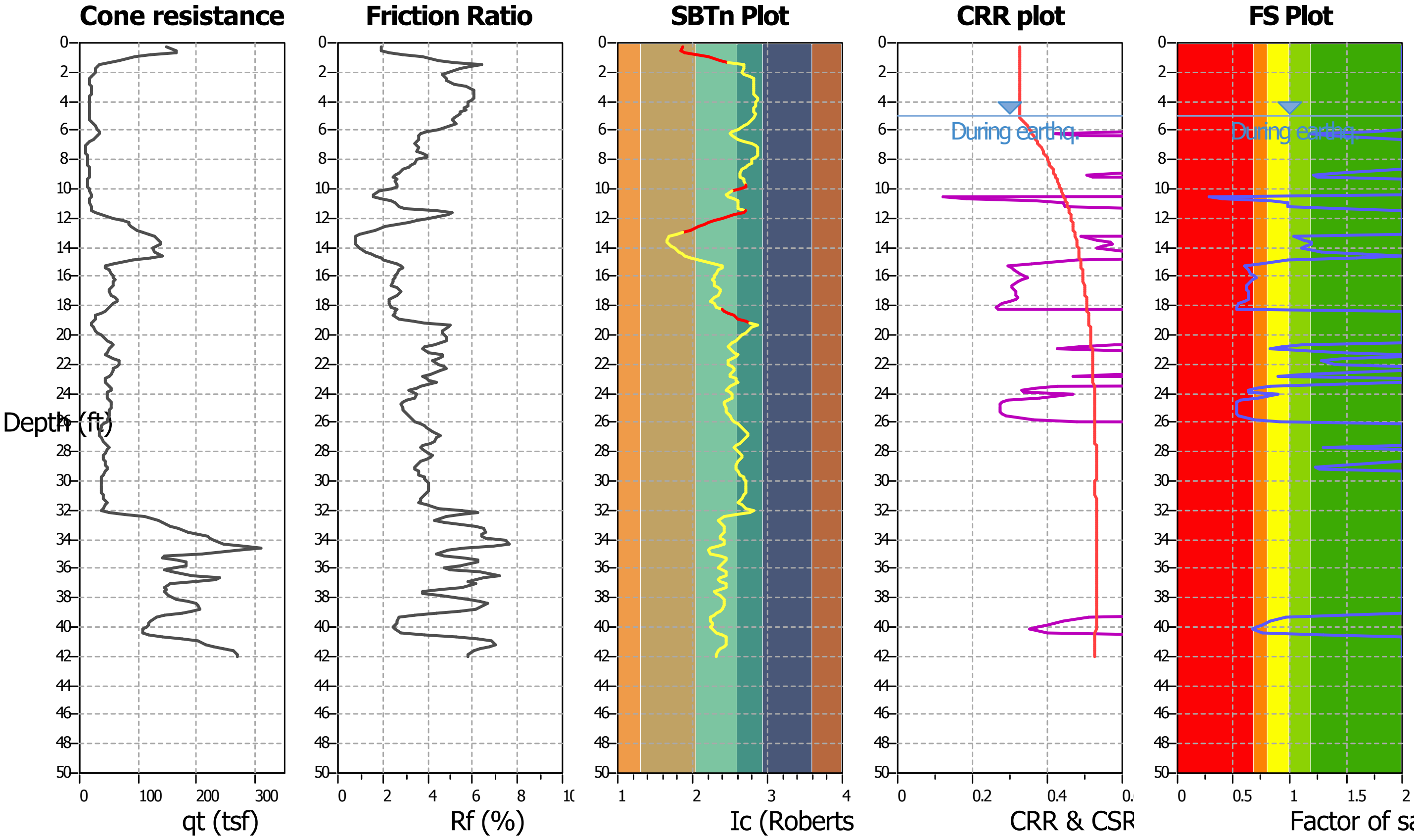
LIQUEFACTION ANALYSIS REPORT

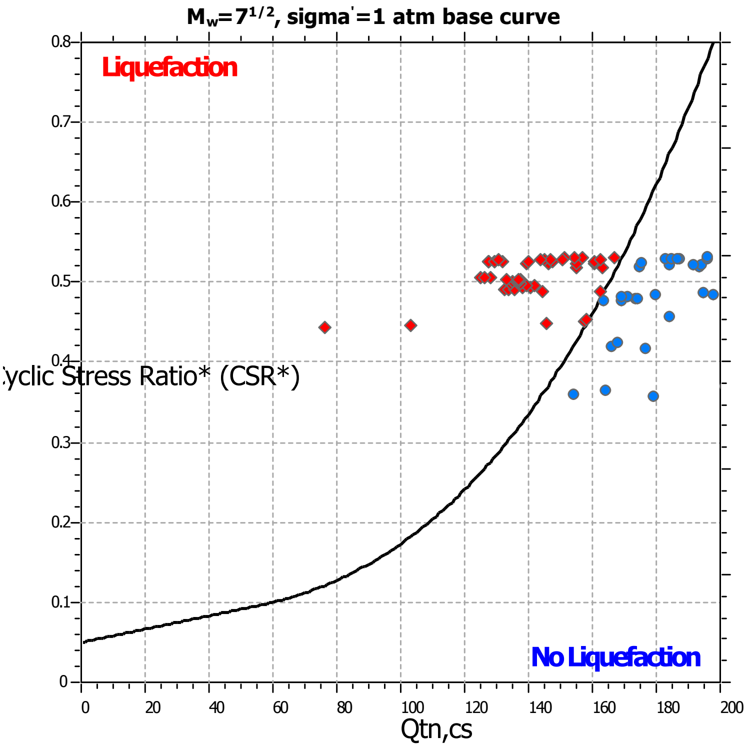
Project title : East D Street **Location : Petaluma, CA**

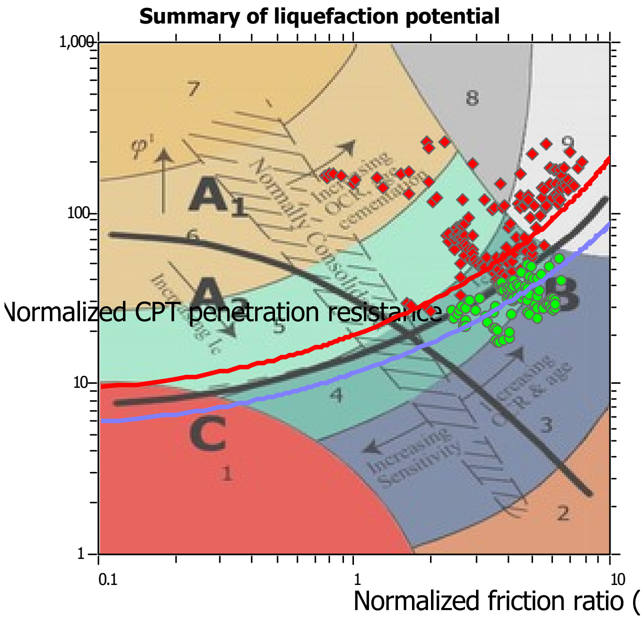
CPT file : CPT-08

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

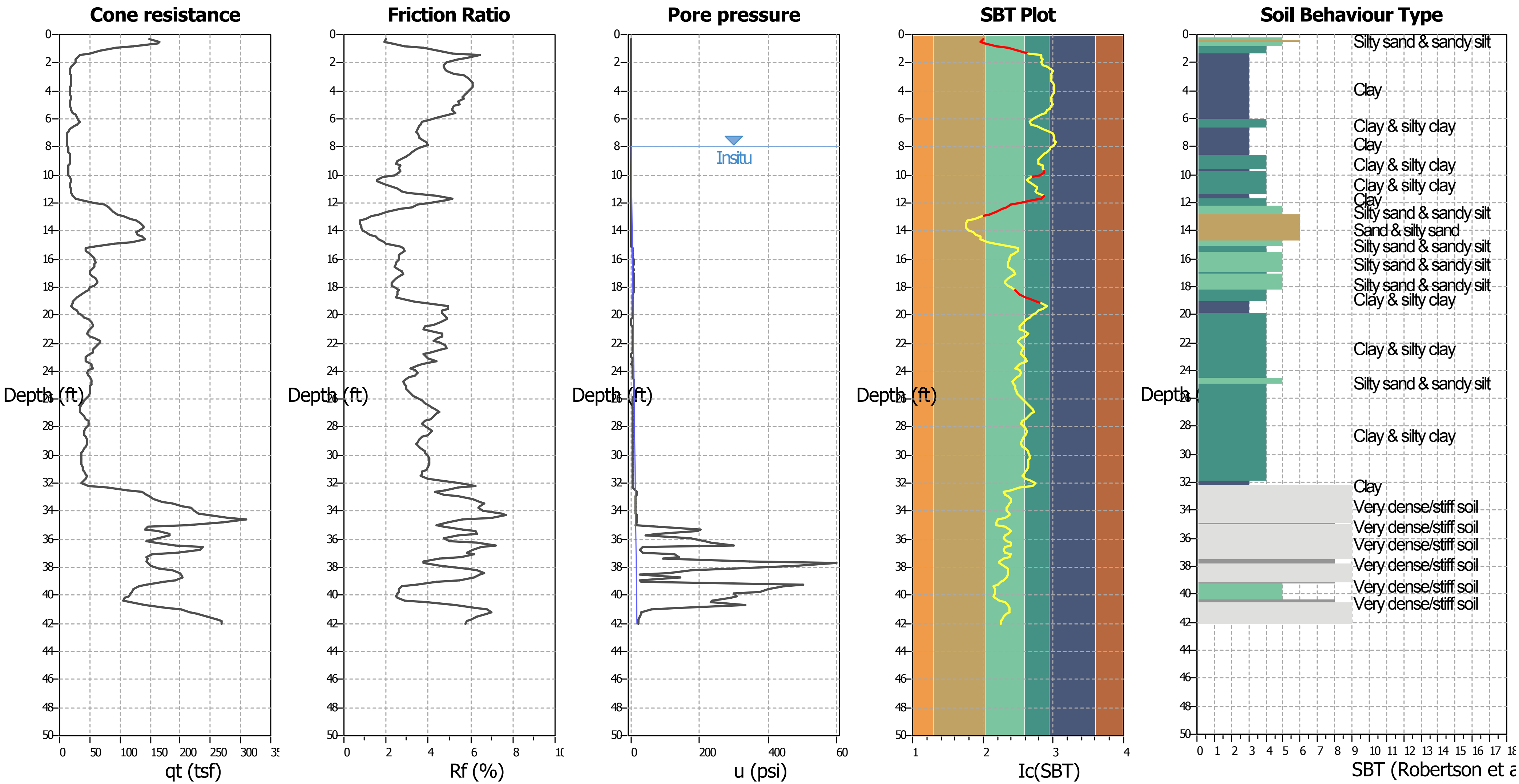






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



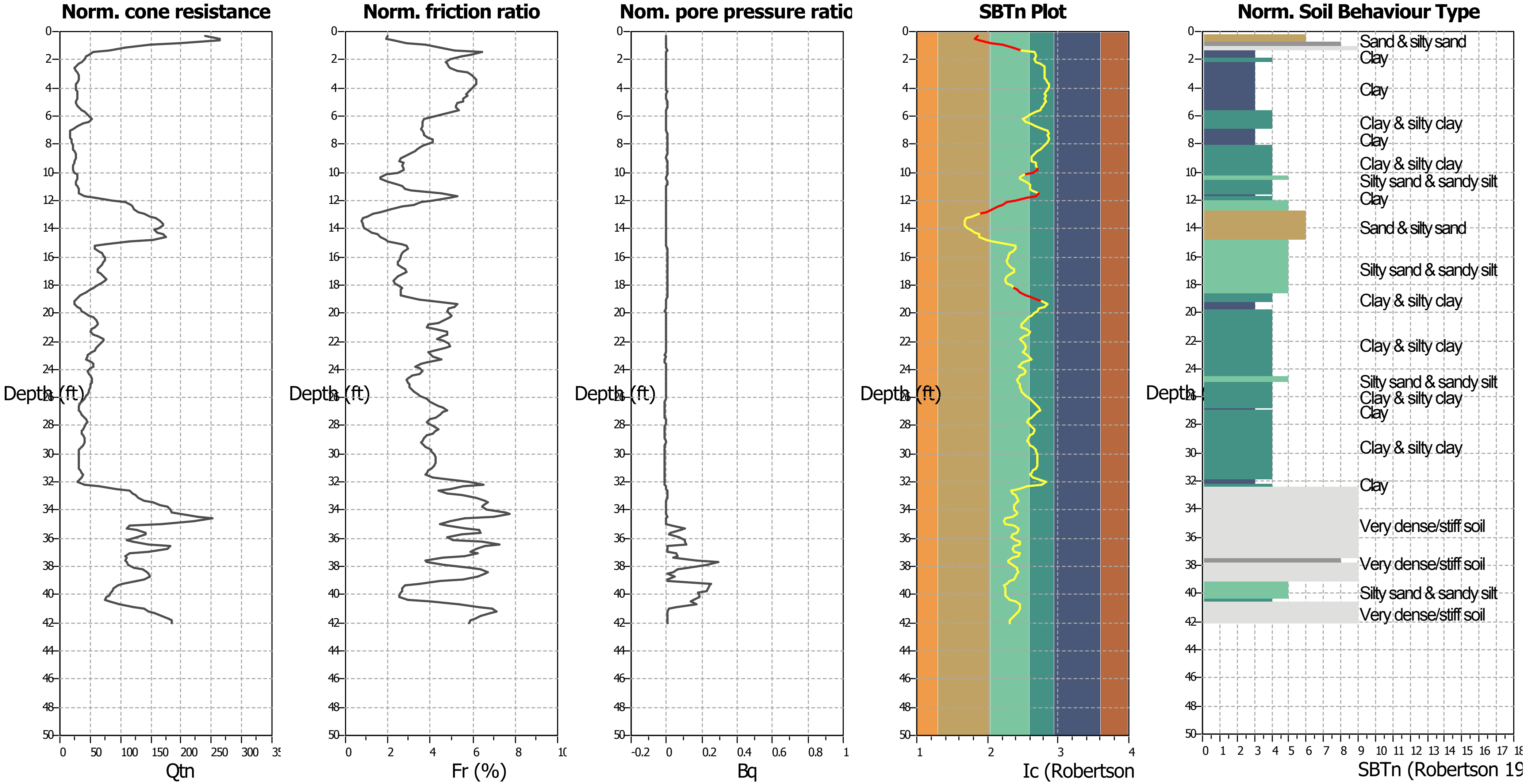
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
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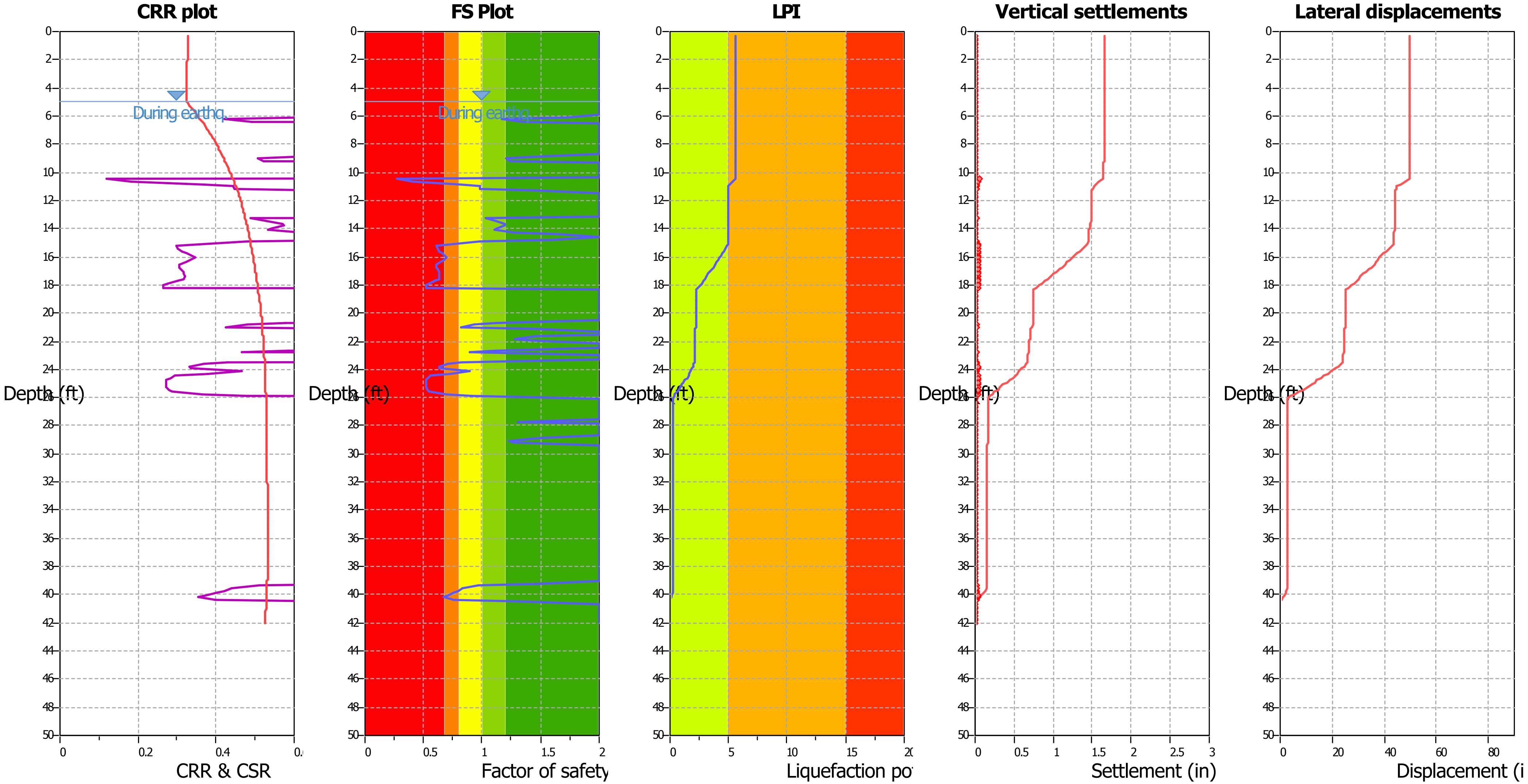
Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

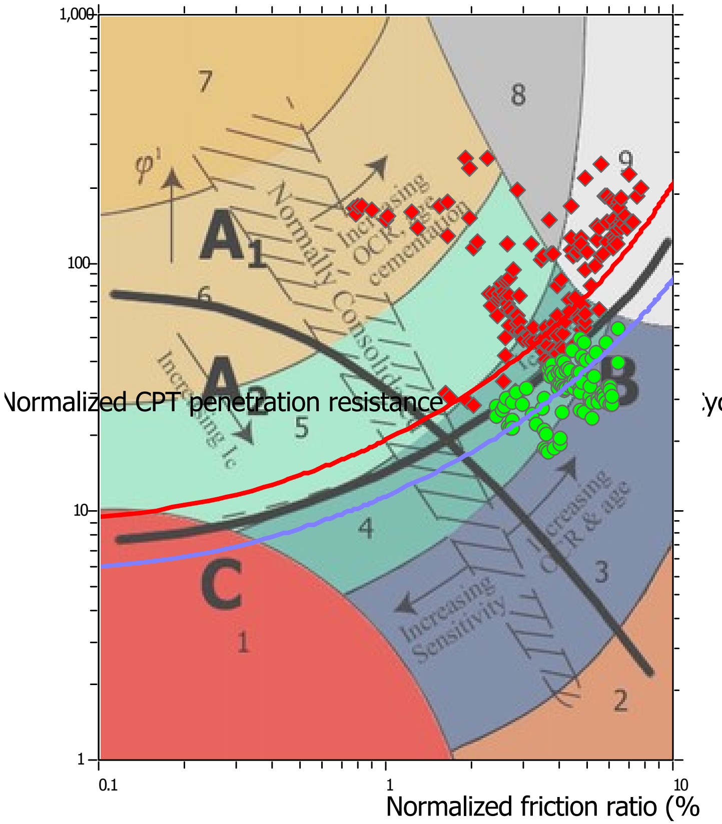
F.S. color scheme

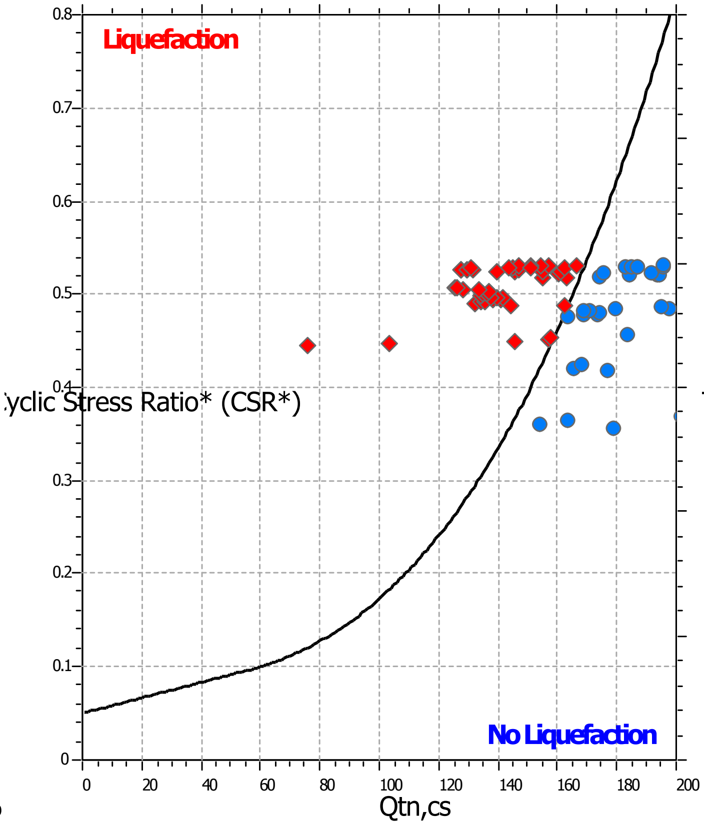
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

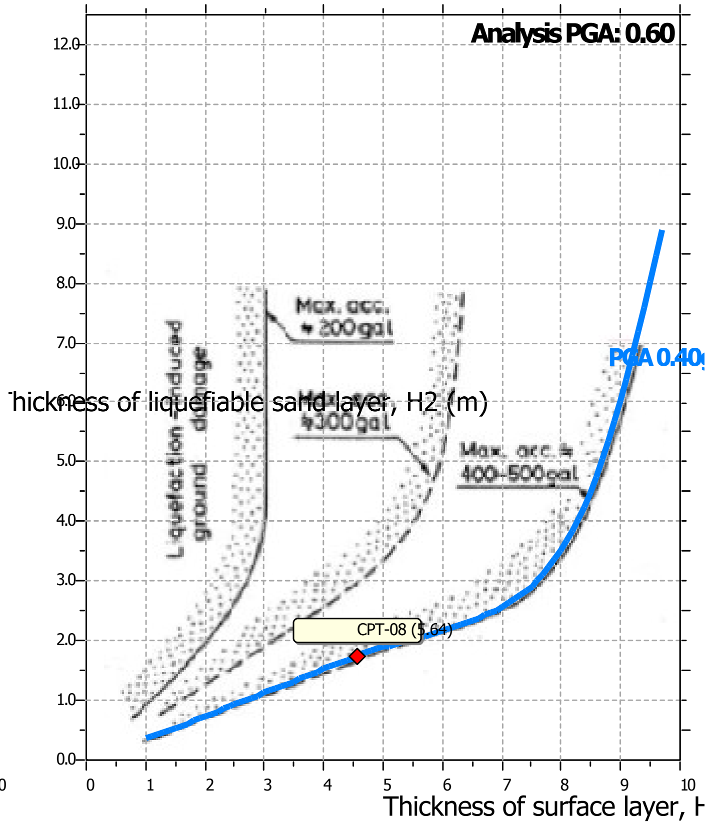
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

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Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
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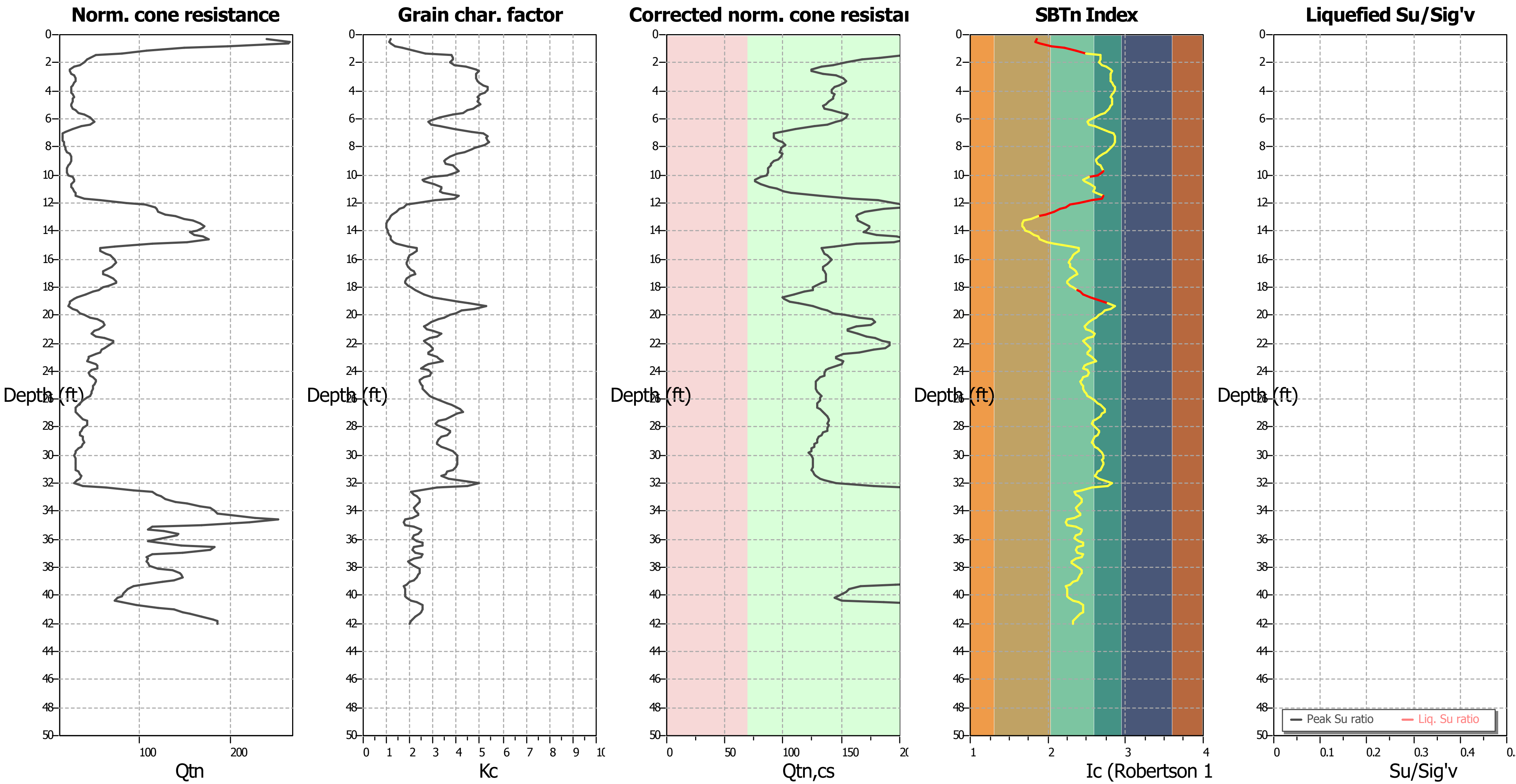
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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: 200 — 40 — 10 — 4

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

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

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
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

MOISTURE CONDITION

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

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

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

ENGEO
Expect Excellence

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 38.234186111   LONGITUDE: -122.633325

Geotechnical Exploration
East D street
Petaluma
15571.002.000

DATE DRILLED: 5/20/2021
HOLE DEPTH: Approx. 48 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: K. McFadden / TB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet: 5, 10, 15, 20, 25
Elevation in Feet: 10, 5, 0, -5, -10

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	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 WITH GRAVEL (CL), reddish brown, dry, medium plasticity, rounded gravel, trace organics [FILL]	20									
CLAYEY SAND (SC), reddish brown, loose, moist, subangular [NATIVE]										
LEAN CLAY (CL), brown, stiff, moist, medium plasticity	13								2.5*	PP
POORLY GRADED SAND (SP), reddish brown, loose, moist										
FAT CLAY (CH), dark gray, soft, moist, high plasticity [YBM]	3									
		119	37	82				738		LVS
								60*	0.25*	PP+TV
	1									
POORLY GRADED SAND WITH CLAY (SP-SC), dark gray, loose, moist	13									
Dense	33				14					
SANDY LEAN CLAY (CL), brown light gray, stiff, moist, iron oxide staining, blocky										

LOG - GEOTECHNICAL_SU+QU W/ ELEV GINT.GPJ ENGEO INC.GDT 7/13/21

LOG OF BORING 1-B1

ENGEO Expect Excellence

LATITUDE: 38.234186111 LONGITUDE: -122.633325

Geotechnical Exploration
East D street
Petaluma
15571.002.000

DATE DRILLED: 5/20/2021
HOLE DEPTH: Approx. 48 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: K. McFadden / TB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, 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	Plastic Limit	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), brown light gray, stiff, moist, iron oxide staining, blocky												
	-15														
30						16					25.7	101.6	961.6		UU
	-20														
35			Yellowish brown, hard			87								4.5+*	PP
	-25		LEAN CLAY (CL), yellowish brown, hard, moist, trace subangular gravel												
40						45									
	-30														
45			Gray and reddish brown, iron oxide staining			50 for 4"					27.5	96.5	5038.9		UU
						64									
	-35		Boring terminated at 48 feet beneath ground surface. Groundwater not observed due to drilling method.												

LOG OF BORING 1-B1A

ENGEO Expect Excellence

LATITUDE: 38.234186111 LONGITUDE: -122.633325

Geotechnical Exploration
East D street
Petaluma
15571.002.000

DATE DRILLED: 5/20/2021
HOLE DEPTH: Approx. 15½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: K. McFadden / TB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
0–10.5	10, 5			NR											
10			SANDY LEAN CLAY (CL), reddish brown, moist, medium plasticity, fine sand												
	0		FAT CLAY (CH), dark gray, soft, moist, high plasticity, organics (rootlets)				66	26	40				1233		LVS
15							119	37	82		90	48.8	402.5 500*	0*	UU PP+TV
			Boring terminated at 15.5 feet beneath ground surface. Groundwater not encountered.												

LOG OF BORING 1-DP1

ENGEO
Expect Excellence

LATITUDE: 38.235669 LONGITUDE: -122.634519

Geotechnical Exploration
East D street
Petaluma
15571.002.000

DATE DRILLED: 5/20/2021
HOLE DEPTH: Approx. 40 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 15 ft.

LOGGED / REVIEWED BY: K. McFadden / TB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Direct Push
HAMMER TYPE: N/A

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
				NR											
			SANDY SILT (ML), light olive gray, moist, fine-grain sand												
5	10														
			CLAYEY SAND (SC-CL), olive gray, moist, fine-grain sand												
10	5														
			CLAYEY SAND (SC), pale olive, moist, fine-grain sand												
			POORLY GRADED SAND WITH SILT (SP-SM), olive, moist, fine-grain sand												
15	0														
20	-5														
			FAT CLAY WITH SAND (CL), olive gray, moist, medium plasticity												
							59	24	35		30.3				
25	-10														

LOG - GEOTECHNICAL_SU+QU W/ ELEV GINT.GPJ ENGEO INC.GDT 7/13/21

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LOG OF BORING 1-DP1

LATITUDE: 38.235669 LONGITUDE: -122.634519

Geotechnical Exploration
East D street
Petaluma
15571.002.000

DATE DRILLED: 5/20/2021
HOLE DEPTH: Approx. 40 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 15 ft.

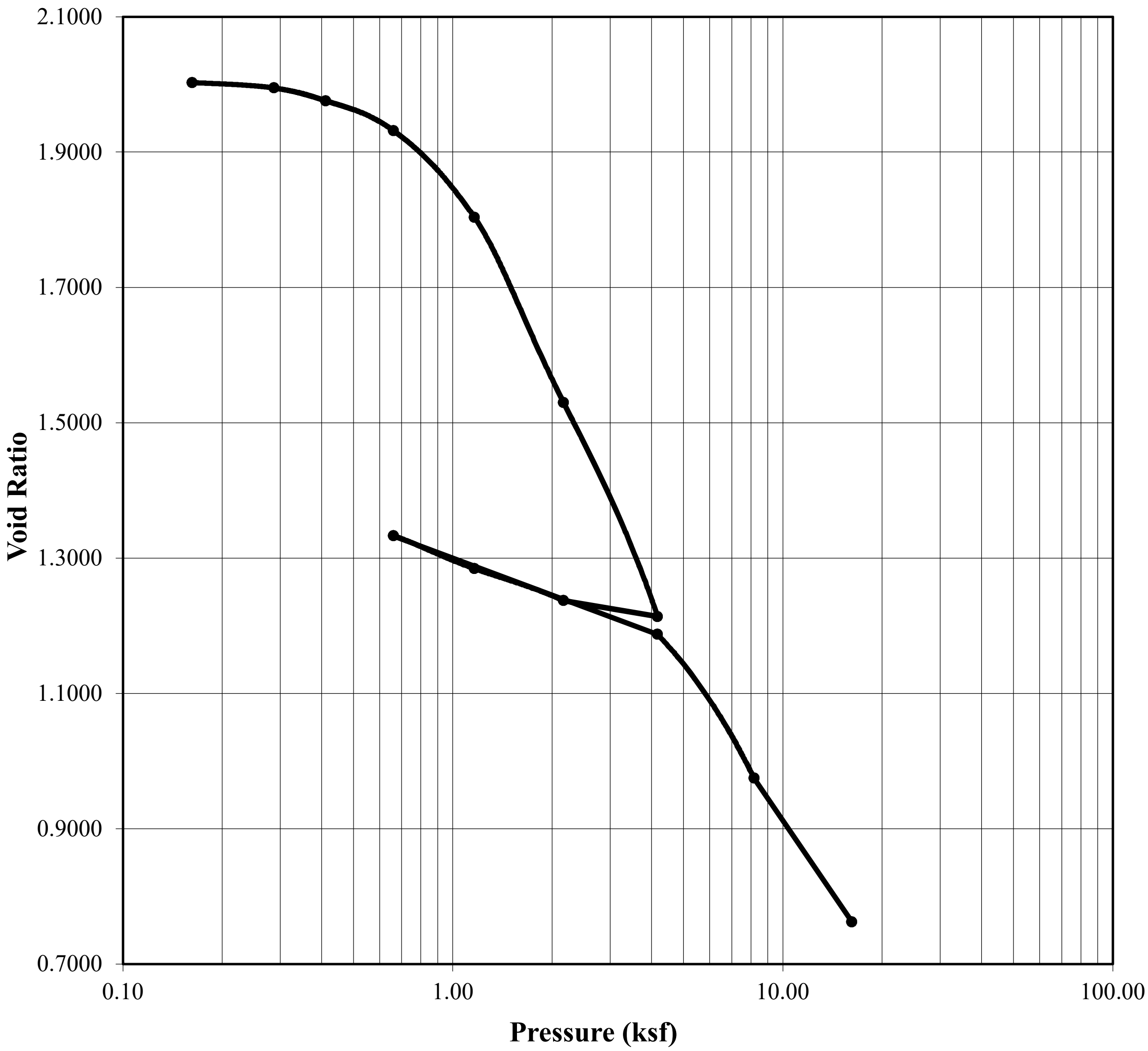
LOGGED / REVIEWED BY: K. McFadden / TB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Direct Push
HAMMER TYPE: N/A

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
			SILT (ML), olive gray, moist, trace fine grain sand												
			Increasing sand content				36	28	8		25.7				
			CLAYEY SAND (SC), light gray, moist, fine-grain sand												
30	-15		LEAN CLAY (CL), olive gray, moist, medium plasticity												
			Pale olive												
35	-20														
			SANDY SILT (ML), olive, moist, iron oxide staining, fine-grain sand				36	32	4		30.3				
			POORLY GRADED SAND WITH SILT (SP-SM), pale olive, moist, fine-grain sand												
			LEAN CLAY (CL), pale olive, moist												
40	-25		Boring terminated at 40 feet beneath ground surface. Groundwater not observed due to drilling method.												

LABORATORY TESTING

15571.001.000
July 19, 2021

Incremental Consolidation
ASTM D2435 - Method B



	Before	After	ASTM D4318 - Wet Method		Test Date: 06/25/21
Moisture (%):	63.14	40.27	Liquid Limit:	n/a	
Dry Density (pcf):	54.86	79.97	Plastic Limit:	n/a	
Saturation (%):	83.08	99.99	ASTM D854 - Measured		
Void Ratio:	2.0102	0.7653	Specific Gravity:	2.646	
Soil Description:	See exploration logs		Remarks:		

Project Number: 15571.002.001 PH001
Sample Number: 1-B1@13-15.5
Project Name: East D St
Client: KB Home North Bay
Location: Petaluma, California
Tested By: G. Criste **Checked By:** K. Lecce

Depth: 13.0-15.5 fee
Boring #: 1-B1

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LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	1-B1@15-16.5	N	16-16.25	3	738
2	1-B1A@11-13	N	12.50-12.75	4	1233

Testing remarks:

PROJECT NAME: East D St
PROJECT NUMBER: 15571.002.001 PH002
CLIENT: KB Home North Bay
ROJECT LOCATION: Petaluma, California

DATE: 06/17/21

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Tested by: G. Criste
Reviewed by: P. Galicia

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	1-DP1 @24.5	1-DP1@28	1-DP1@3					
DEPTH (ft.)	24.5	28	37					
METHOD A OR B	B	B	B					
MOISTURE CONTENT (%)	30.3	25.7	30.3					

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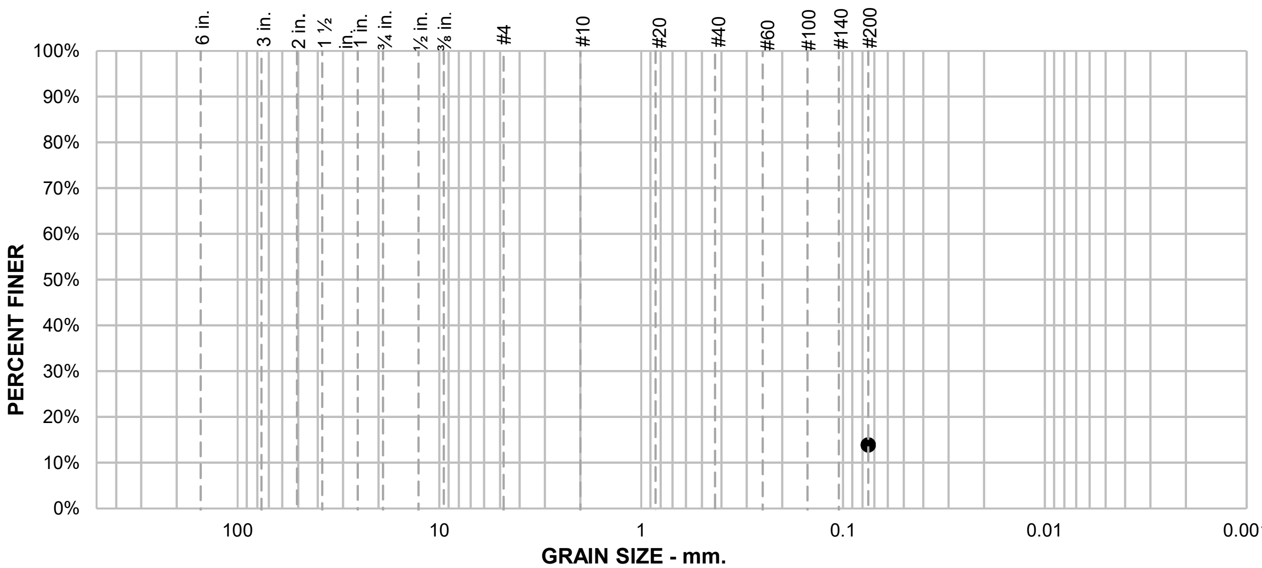
CLIENT: KB Home North Bay
PROJECT NAME: East D St
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, California
REPORT DATE: 6/17/2021
TESTED BY: A. Perez
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B1@21.5-23

DEPTH (ft): 21.5-23

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						13.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13.9		

* (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

Soak time = 180 min
Dry sample weight = 789.7 g

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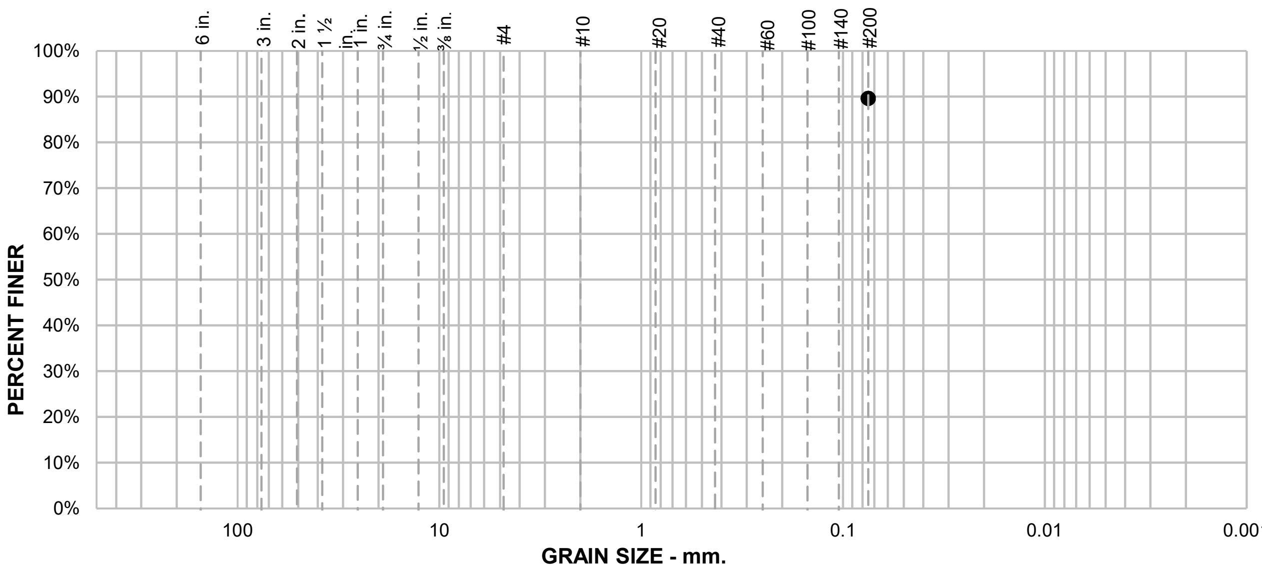
CLIENT: KB Home North Bay
PROJECT NAME: East D Street
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 6/21/2021
TESTED BY: G. Criste
REVIEWED BY: M. Quasem

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-DP1@24.5

DEPTH (ft): 24.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						89.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	89.6		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 24 LL = 59 PI = 35

COEFFICIENTS

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

CLASSIFICATION

USCS = CH

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 33.27 g

CLIENT: KB Home North Bay

PROJECT NAME: East D Street

PROJECT NO: 15571.002.001 PH002

PROJECT LOCATION: Petaluma, CA

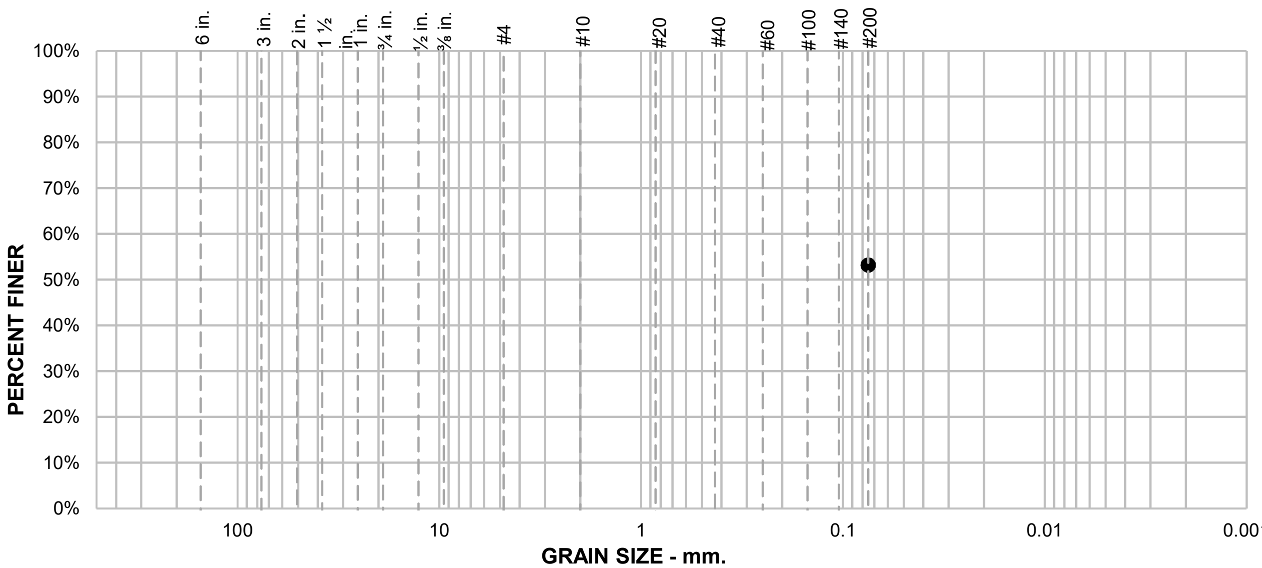
REPORT DATE: 6/21/2021

TESTED BY: G. Criste

REVIEWED BY: M. Quasem

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-DP1@28

DEPTH (ft): 28

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						53.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	53.2		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 28 LL = 36 PI = 8

COEFFICIENTS

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

CLASSIFICATION

USCS = ML

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 31.22 g

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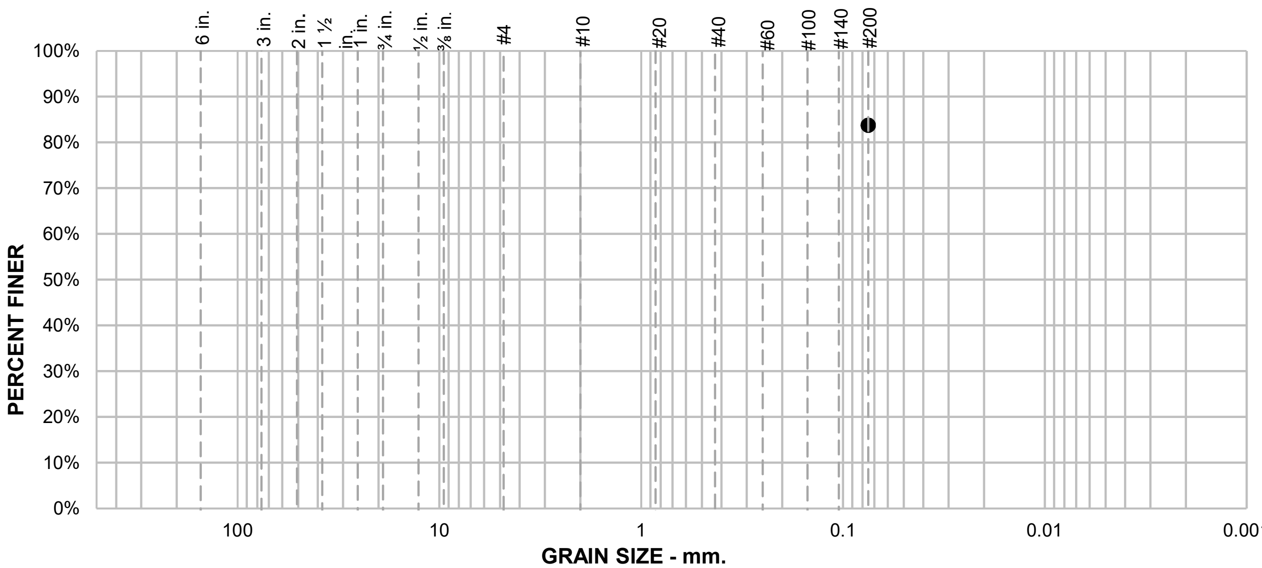
CLIENT: KB Home North Bay
PROJECT NAME: East D Street
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 6/21/2021
TESTED BY: G. Criste
REVIEWED BY: M. Quasem

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-DP1@37

DEPTH (ft): 37

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						83.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	83.7		

SOIL DESCRIPTION
See exploration logs

ATTERBERG LIMITS

PL = 32	LL = 36	PI = 4

COEFFICIENTS

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

CLASSIFICATION
USCS = ML

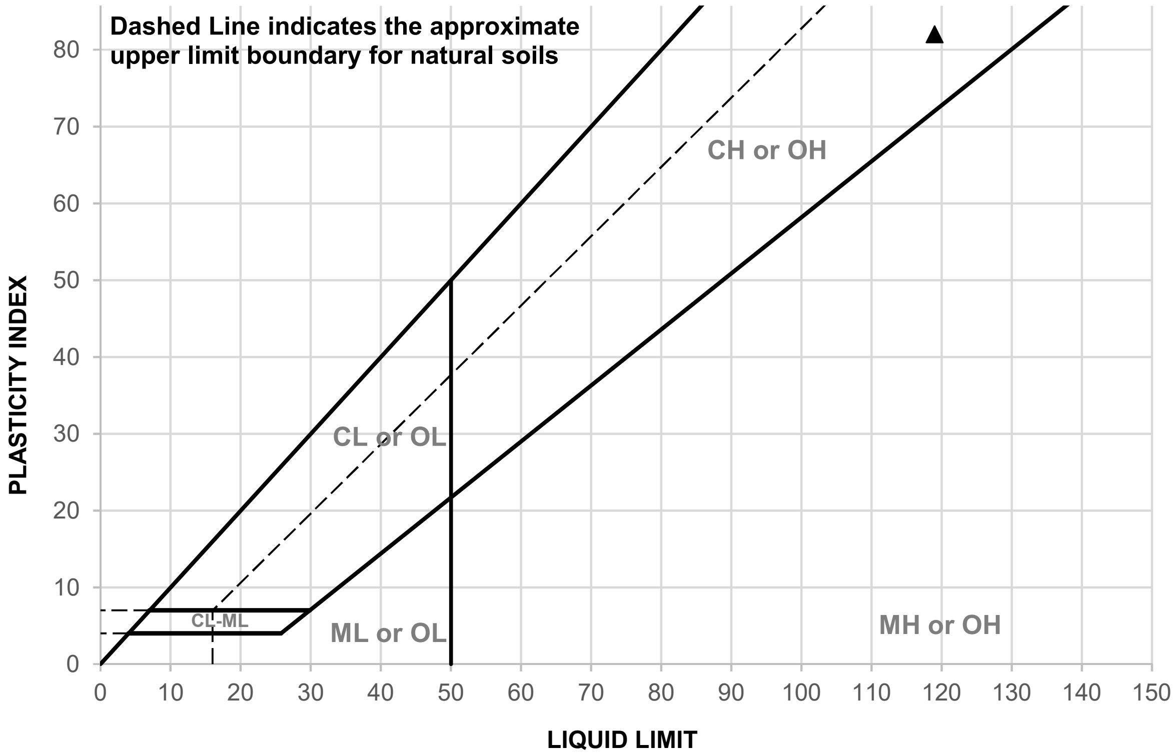
REMARKS
PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 64.23 g

* (no specification provided)

CLIENT: KB Home North Bay
PROJECT NAME: East D Street
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 6/21/2021
TESTED BY: G. Criste
REVIEWED BY: M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@15-16.5	15-16.5 feet	See exploration logs	119	37	82

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@15-16.5	PI: ASTM D4318, Wet Method	

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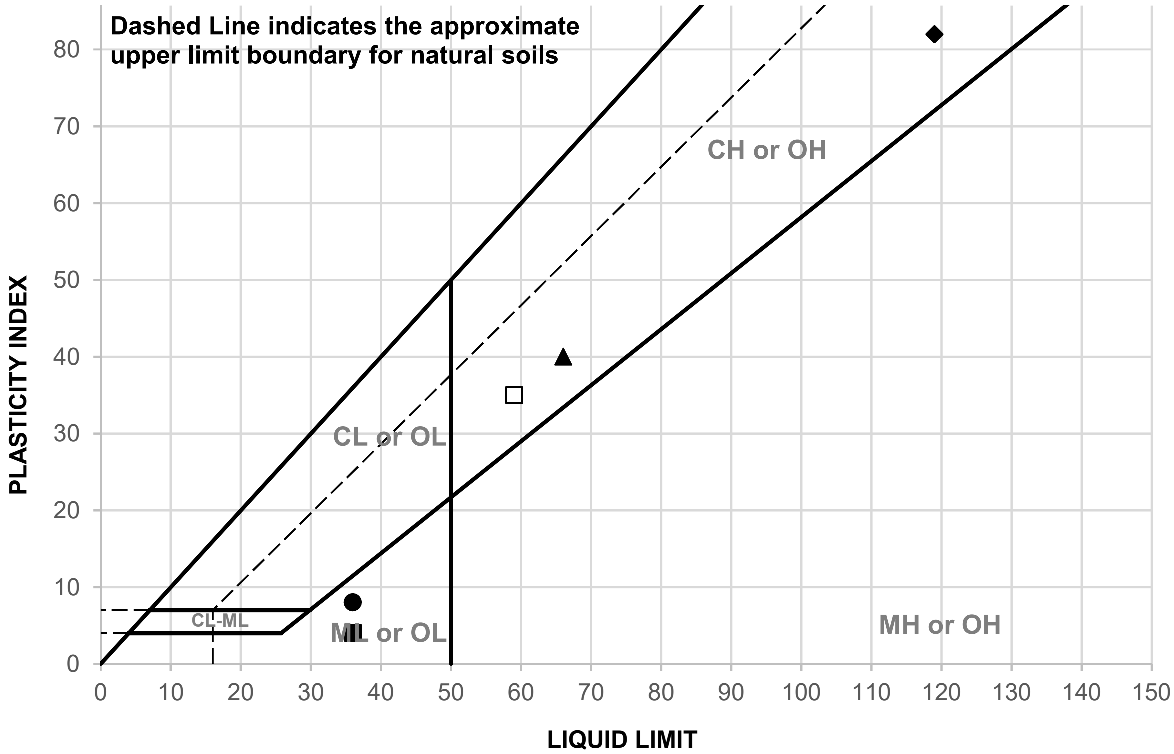
CLIENT: KB Home North Bay
PROJECT NAME: East D St
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, California
REPORT DATE: 6/14/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1A@11-13	11-13 feet	See exploration logs	66	26	40
◆	1-B1A@13.5-15.5	13.5-15.5 feet	See exploration logs	119	37	82
□	1-DP1@24.5	24.5 feet	See exploration logs	59	24	35
●	1-DP1@28	28 feet	See exploration log	36	28	8
■	1-DP1@37	37 feet	See exploration logs	36	32	4

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1A@11-13	PI: ASTM D4318, Wet Method	
◆	1-B1A@13.5-15.5	PI: ASTM D4318, Wet Method	
□	1-DP1@24.5	PI: ASTM D4318, Wet Method	
●	1-DP1@28	PI: ASTM D4318, Wet Method	
■	1-DP1@37	PI: ASTM D4318, Wet Method	

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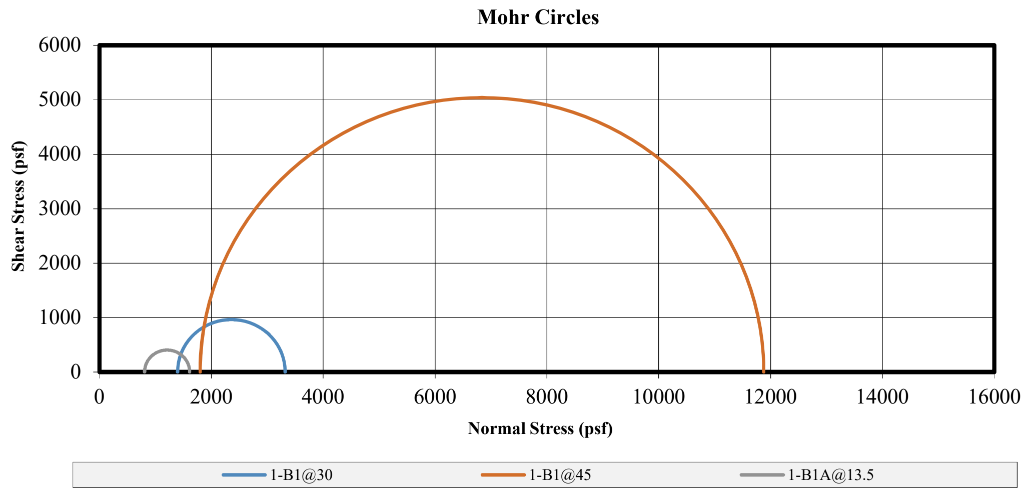
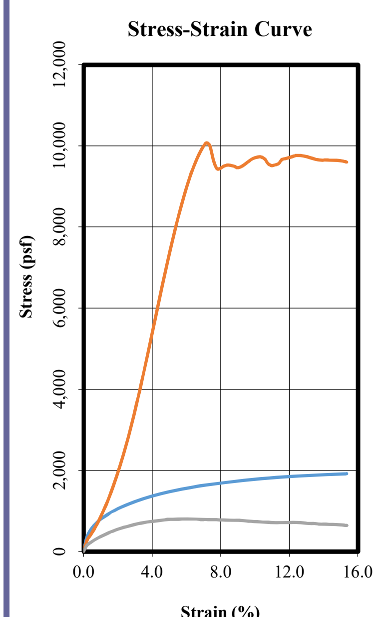
CLIENT: KB Home North Bay
PROJECT NAME: East D St
PROJECT NO: 15571.002.001 PH002
PROJECT LOCATION: Petaluma, CA
REPORT DATE: 6/21/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

Tested By: G. Criste   Date: 6/17/2021   Checked By: K. Lecce   Date: 06/18/21





Before Test	Specimen 1-B1@30	1-B1@45	1-B1A@13.5	
Water Content (%)	25.68	27.54	89.98	
Dry Density (pcf)	101.60	96.50	48.80	
Saturation (%)	99.94	98.71	98.70	
Void Ratio	0.72	0.76	2.48	
Diameter (in)	2.397	2.391	2.836	
Height (in)	4.990	5.019	5.939	
Height-to-Diameter Ratio	2.082	2.099	2.094	
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Assumed				
Specific Gravity	2.795	2.720	2.720	
After Test	**1-B1@30**	**1-B1@45**	**1-B1A@13.5**	
Water Content (%)	25.68	27.54	89.98	
Saturation (%)	99.94	98.70	98.70	
Strain Rate (%/min)	0.05	0.05	0.06	
Peak Deviator Stress (psf)	1923.2	10077.8	805.0	
Axial Strain @ Failure (%)	15.341	7.173	6.230	
	Cell Pressure			
Cell (psf)	1396.8	1800.0	806.4	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	3320.0	11877.8	1611.4	
σ3 (psf)	1396.8	1800.0	806.4	
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)	
Cohesion, c (psf)	n/a
Friction Angle Ø	n/a

	Cohesion at Failure with a Zero Friction Angle (Ø=0)			
	961.6	5038.9	402.5	
	n/a	n/a	n/a	

Project Information	
Project Name:	East D St
Project Number:	15571.002.001 PH002
Project Location:	Petaluma, California
Client:	KB Home North Bay
Description:	See exploration logs
Test Remarks:	

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Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

Tested By: G. Criste   Date: 6/17/2021   Checked By: K. Lecce   Date: 06/18/21

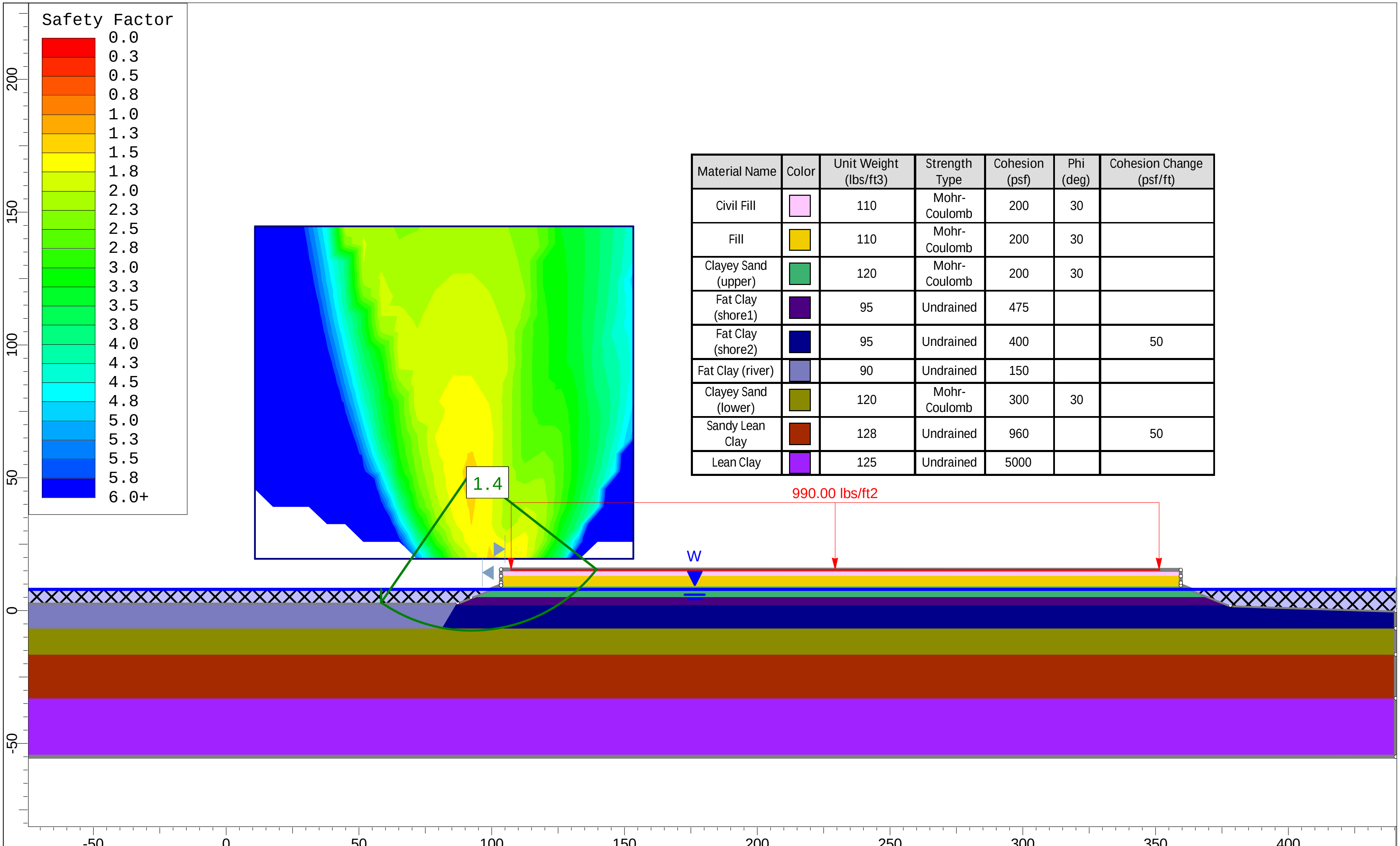
SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B1@30	SAMPLE NUMBER: 1-B1@45
	
SAMPLE NUMBER: 1-B1A@13.5	
	

Project Information		
Project Name:	East D St	
Project Number:	15571.002.001 PH002	ENGEO Expect Excellence
Project Location:	Petaluma, California	
Client:	KB Home North Bay	
Description:	See exploration logs	
Test Remarks:		

APPENDIX E

SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Civil Fill		110	Mohr-Coulomb	200	30	
Fill		110	Mohr-Coulomb	200	30	
Clayey Sand (upper)		120	Mohr-Coulomb	200	30	
Fat Clay (shore1)		95	Undrained	475		
Fat Clay (shore2)		95	Undrained	400		50
Fat Clay (river)		90	Undrained	150		
Clayey Sand (lower)		120	Mohr-Coulomb	300	30	
Sandy Lean Clay		128	Undrained	960		50
Lean Clay		125	Undrained	5000		

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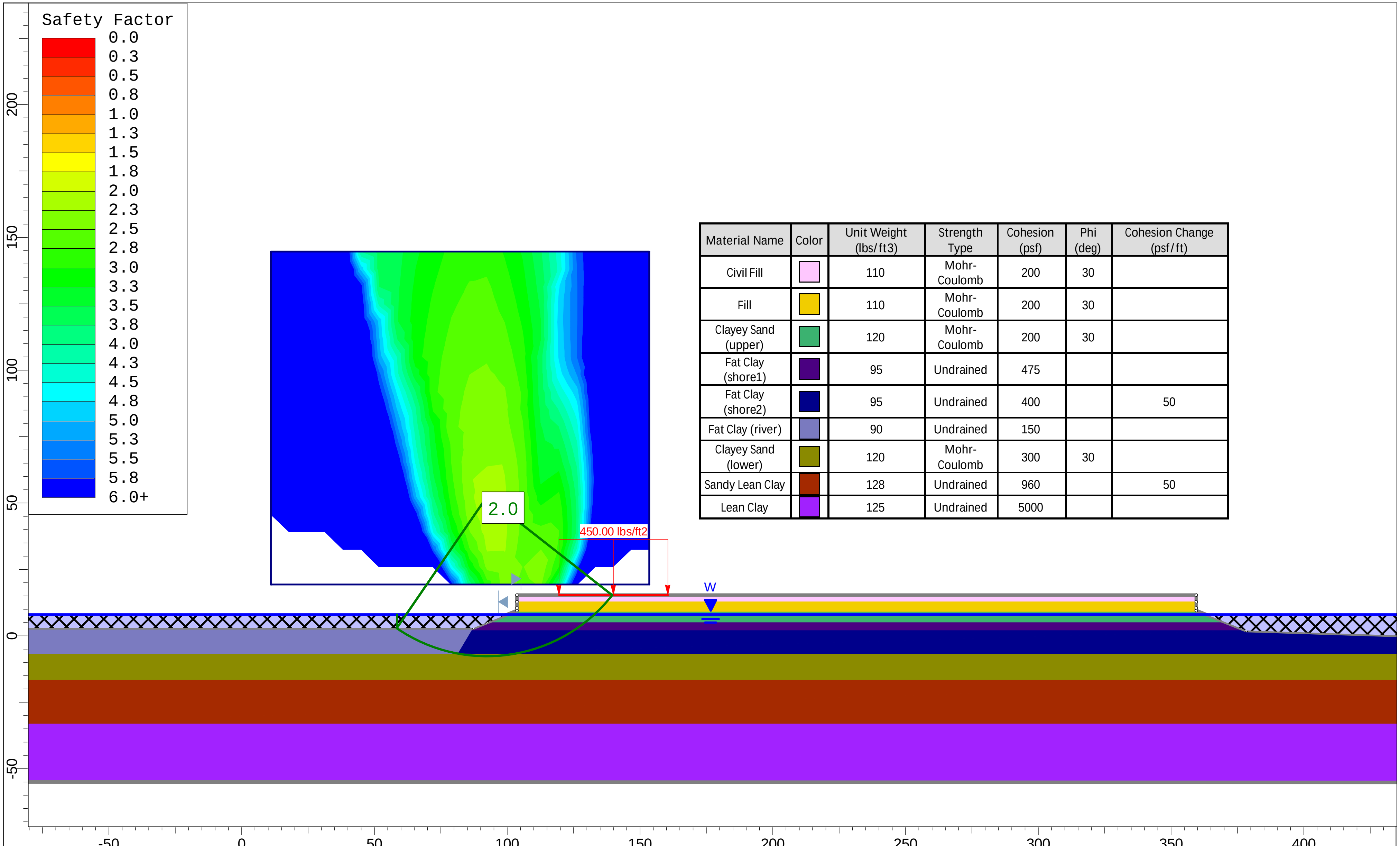
SLIDEINTERPRET 9.027

Project	15571.003.000 - Oyster Cove (1:600 Scale)
Analysis Description	Static - Temporary Surcharge Loading
Drawn By	KDK/TPB/JAF
Company	
Date	01/25/2024
File Name	15571003000_Slide_Section A-A'.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Civil Fill		110	Mohr-Coulomb	200	30	
Fill		110	Mohr-Coulomb	200	30	
Clayey Sand (upper)		120	Mohr-Coulomb	200	30	
Fat Clay (shore1)		95	Undrained	475		
Fat Clay (shore2)		95	Undrained	400		50
Fat Clay (river)		90	Undrained	150		
Clayey Sand (lower)		120	Mohr-Coulomb	300	30	
Sandy Lean Clay		128	Undrained	960		50
Lean Clay		125	Undrained	5000		

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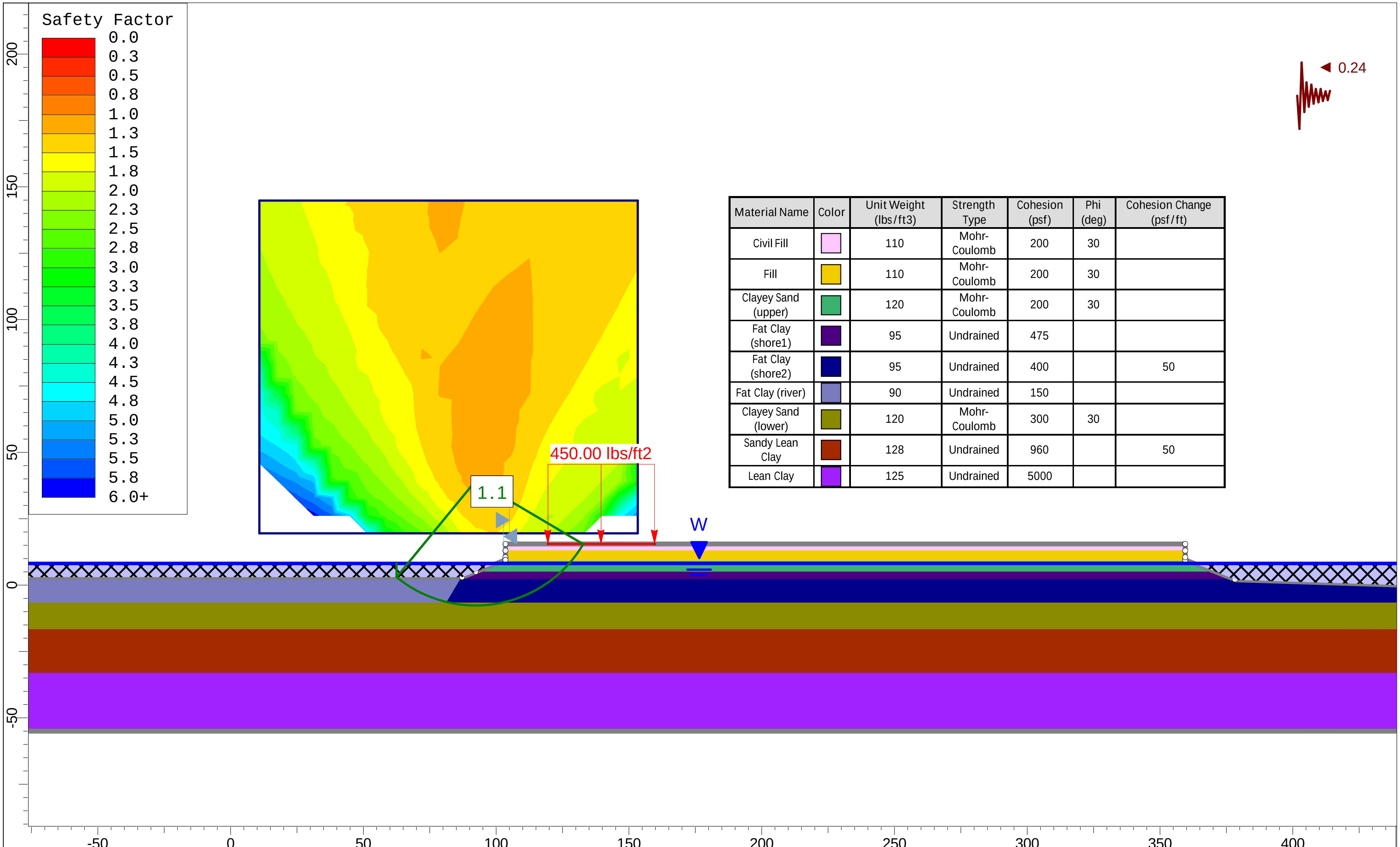
SLIDEINTERPRET 9.027

Project	15571.003.000 - Oyster Cove (1:600 Scale)
Analysis Description	Static - Long-Term Loading
Drawn By	KDK/TPB/JAF
Company	
Date	02/13/2024
File Name	15571003000_Slide_Section A-A'.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Civil Fill		110	Mohr-Coulomb	200	30	
Fill		110	Mohr-Coulomb	200	30	
Clayey Sand (upper)		120	Mohr-Coulomb	200	30	
Fat Clay (shore1)		95	Undrained	475		
Fat Clay (shore2)		95	Undrained	400		50
Fat Clay (river)		90	Undrained	150		
Clayey Sand (lower)		120	Mohr-Coulomb	300	30	
Sandy Lean Clay		128	Undrained	960		50
Lean Clay		125	Undrained	5000		

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SLIDEINTERPRET 9.027

Project 15571.003.000 - Oyster Cove (1:600 Scale)

Analysis Description Seismic (5 cm) - Long-Term Loading

Drawn By KDK/TPB/JAF

Company

Date 01/25/2024

File Name 15571003000_Slide_Section A-A'.slmd

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