Appendix C:
Preliminary Geotechnical Exploration

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PECTEN SITE

MILPITAS/SAN JOSE, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Scott Connelly
Valley Oak Partners, LLC
734 The Alameda
San Jose, CA 95126

PREPARED BY
ENGEO Incorporated

September 24, 2021

PROJECT NO.
19078.000.001

ENGEO
Expect Excellence

Project No.
19078.000.001

September 24, 2021

Mr. Scott Connelly
Valley Oak Partners, LLC
734 the Alameda
San Jose, CA 95126

Subject: Pecten Site
Milpitas/San Jose, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Connelly:

We are pleased to present this preliminary geotechnical exploration report for proposed developments at the "Pecten Site" located on the border of Milpitas and San Jose, California. The accompanying report presents our preliminary conclusions and recommendations regarding future developments at the site under consideration.

Our findings indicate that the site is suitable for the proposed residential development, provided the preliminary recommendations and guidelines provided in this report are implemented during project planning. The scope of this report is limited to an initial study, and we recommend a design-level geotechnical exploration should be conducted to further evaluate the geotechnical hazards identified in this report and to develop geotechnical parameters for grading plan preparation and foundation design.

We are pleased to have been of service on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions or comments regarding this preliminary report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

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TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical exploration report in support of your evaluation of developing the Pecten site, located in Milpitas and San Jose, California. As outlined in our executed agreement dated July 12, 2021, Valley Oak Partners, LLC authorized us to perform the following scope of services.

- Existing document review
- Limited subsurface exploration
- Data analysis and development of preliminary geotechnical recommendations
- Report preparation

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1.2 PROJECT LOCATION

The approximately 3.6-acre site is located southwest of the intersection of Interstate 680 (I-680) freeway and Montague Expressway and located in a mixed residential and industrial area spanning the border of San Jose and Milpitas, as shown in the Vicinity Map, Figure 1. Based on discussions with you and review of the Preliminary Title Report prepared by First American Title Insurance Company, we understand the site incorporates two parcels, consisting of two Assessor's Parcel Numbers (APNs), as shown in Exhibit 1.2-1.

The site is bounded generally by residential developments along the southern perimeter, industrial development along the eastern and northern perimeters, and the I-680 freeway to the east. The parcel numbers, property addresses, and APNs are summarized in Table 1.2-1.

EXHIBIT 1.2-1 Parcel Plan

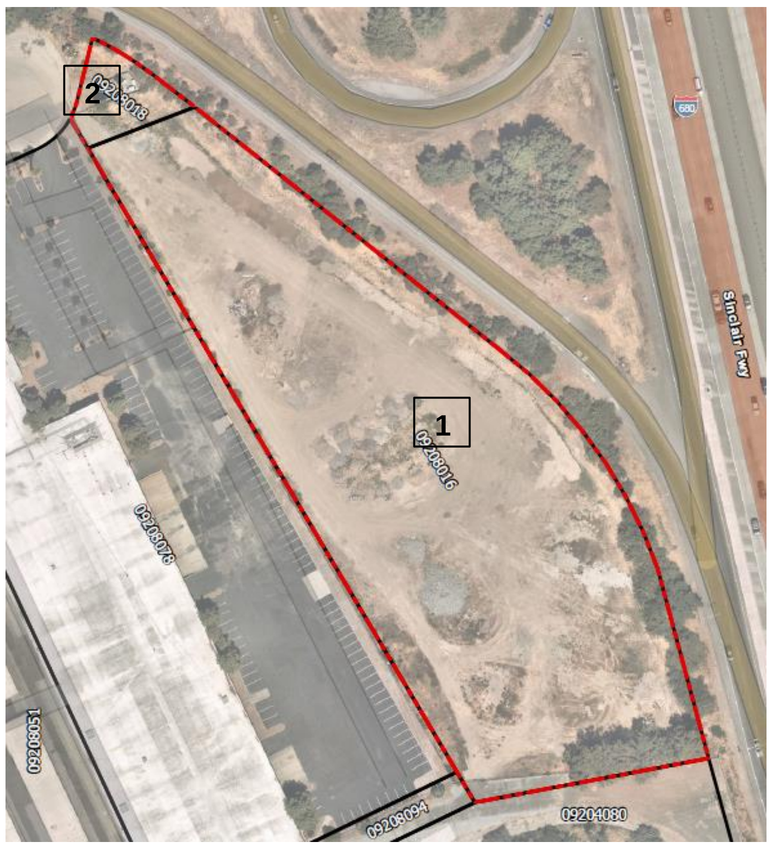

TABLE 1.2-1. Parcel Description

PARCEL	LOCATED IN	ASSESSOR'S PARCEL NUMBER
1	San Jose	092-08-016
2	Milpitas	092-08-018

1.3 PROPOSED DEVELOPMENT

At the time of our report, no conceptual level development plans were available. Based on our discussions with you, we understand the proposed development may include the construction of a warehouse building or a moderate- to high-density multi-family residential or commercial structure consisting of up to five stories. We understand the proposed structures may be supported at-grade, and basement levels are not being considered at this time. Site improvements may also include paved parking areas and associated underground utilities.

2.0 FINDINGS

2.1 HISTORICAL TOPOGRAPHIC MAP/AERIAL PHOTOGRAPH REVIEW

We reviewed readily available historical topographic maps and aerial photographs provided by the Environmental Data Resources report generated for this project site. In 1899, the site is shown as undeveloped land. In the 1939 aerial photograph, the site appears to be used for agricultural purposes, specifically row crops. No structures are visible on the site.

In 1953, the site is shown as undeveloped land (dry farmed). In the 1965 photograph, a large circular water tank is visible in the southern portion of the site, along with other equipment used for the San Jose Water well site. In 1974, the southern portion of the site appears to be paved and used as a parking lot and a storage area for farm-related tanks and equipment. In a 2006 photograph, the well site infrastructure has been removed from the southern portion of the site, and the site appears as a vacant dirt lot.

During our site visit on August 11, 2021, we observed that the site conditions have remained relatively unchanged compared to the conditions shown in the 2006 aerial photograph.

2.2 REGIONAL GEOLOGY AND SEISMICITY

2.2.1 Regional Geologic Setting

The site is mapped as Pleistocene alluvial fan and fluvial deposits (Qpaf) by Helley (1997). These deposits are described as: (1) brown dense gravely and clayey sand, or (2) clayey gravel that transitions to sandy clay as fines increase upward toward the surface. These deposits display various sorting and are located along most stream channels. According to the California Geological Survey (CGS) Seismic Hazard Zones map for Milpitas, the site is mapped within a liquefaction hazard zone (CGS, 2001). According to Witter 2006, the site is mapped as moderately susceptible to liquefaction (USGS, 2006).

2.2.2 Faulting and Seismicity

The project site is not located within a currently designated State of California Earthquake Fault Hazard Zone (1982) or a Santa Clara County Hazard Zone Fault Rupture Hazard Zone (2012), and no known active faults across the site, as shown in Figure 5, Regional Faulting and Seismicity. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (approximately the last 11,000 years) (Hart, 1997).

Other active faults near the site are summarized in Table 2.2.2-1 and include the Hayward-Rodgers Creek fault, Monte Vista-Shannon fault, San Andreas fault, and Calaveras fault. Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

TABLE 2.2.2-1: Known active Faults Capable of Producing Significant Ground Shaking at the Site
Latitude: 37.412451 **Longitude:** -121.881052

FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
Hayward-Rodgers Creek	3.3	6.8
Calaveras	4.6	6.5
Monte Vista-Shannon	12.1	6.5
North San Andreas	16.0	7.5

The Working Group on California Earthquake Probabilities (WGCEP, 2017) 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, including the Northern San Andreas fault, in UCERF3. UCERF3 estimated an overall probability of 72 percent for the Bay Area as a whole, a probability of 14.3 percent for the Hayward fault, 7.4 percent for the Calaveras fault, 6.4 percent for the Northern San Andreas fault, and 3.5 percent for the Concord-Green Valley fault.

Based on the historical seismicity, the proximity of known active faults, and the estimated earthquake probabilities for the Bay Area as a whole, it should be expected that the site will experience strong seismic ground shaking during the lifetime of the proposed improvements.

2.3 FIELD EXPLORATION

2.3.1 Cone Penetration Testing

On August 27, 2021, we retained Conetec to advance three cone penetration tests (CPTs) at the locations shown on Figure 2, Site Plan. CPTs 1-CPT1 and 1-CPT3 were advanced to a depth of about 50 feet below ground surface (bgs). We planned to advance 1-CPT4 to a depth of 100 feet bgs; however, we encountered refusal conditions at a depth of approximately 97 feet bgs.

CPT measurements include tip resistance to penetration of the cone, the resistance of the surface sleeve, and pore pressure (Robertson and Campanella, 1988); CPTs were performed in accordance with ASTM D-5778. Pore pressure dissipation testing was performed to interpret hydrostatic ground water levels. We present the CPT logs in Appendix A.

2.3.2 Soil Sampling and Laboratory Testing

As part of this study, we also collected a representative near-surface soil sample within the project site, which was tested at an ENGEO laboratory for Plasticity Index (PI). Laboratory results are included in Appendix B.

2.4 SURFACE CONDITIONS

Based on our review of available Google imagery, the current topography of the site can generally be characterized as relatively level to gently sloping and ranges from Elevation 76 feet to Elevation 78 feet (WGS-84). The site is currently a vacant dirt lot with moderate amount of surface vegetation. We observed a fill berm along the western perimeter of the site. We also observed an approximately 15-foot-wide by maximum 6-foot-deep excavated trench that was partially infilled with soil generated from spoils at the site and some ponding water.

2.5 SUBSURFACE CONDITIONS

The CPT data provides interpretations of soil behavior type (SBT) based on empirical correlations. The data generally indicates predominantly medium stiff clays and silts in the upper 15 feet, underlain by approximately 10 feet of soft clay. At 25 feet bgs, the CPTs encountered 10 to 15 feet of medium-stiff to stiff clay overlaying about 6 feet of very dense sand.

Below 50 feet bgs, 1-CPT3 shows medium-stiff to stiff clays and silts up to 82 feet bgs, overlaying approximately 6 feet of very dense sand transitioning into stiff clay to the terminal depth of the exploration (approximately 97 feet bgs). The clays and silts may exhibit expansion potential. Logs of our CPT explorations are presented in Appendix A.

Although CPT data interpretations may not identify undocumented fill, our review of historical aerial imagery suggests the presence of existing fill in the southern and western portions of the site. Since historical records of fill placement were not available at the time of our report, we assume the existing fill is non-engineered.

2.6 GROUNDWATER CONDITIONS

As discussed in Section 2.3, we performed pore pressure dissipation tests in the CPTs to interpret static groundwater conditions (based on piezometric levels indicated by the pore pressure tests). We summarize our data in Table 2.6-1.

TABLE 2.6-1: Pore Pressure Dissipation Test Results

EXPLORATION ID	INTERPRETED GROUNDWATER DEPTH (FEET, BGS)*
1-CPT1	17.1
1-CPT2	19.2
1-CPT3	18.0

* Based on piezometric levels indicated by the measured pore pressures at the CPT dissipation test depth.

Plate 1.2 of the Seismic Hazard Zone Report for the Milpitas Quadrangle (CGS, 2000) indicates that the historical high groundwater depths in the vicinity of the site may be expected up to 10 feet bgs. Our review of groundwater data also included publicly available resources, including well data compiled by the California State Resources Control Board – Geotracker. We reviewed available groundwater level data for wells located within a 1-mile radius of the site. The findings from the publicly available resources indicated that the seasonal high groundwater levels generally range between 8 feet to 12 feet below ground surface.

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

2.7 PRELIMINARY CPT-BASED LIQUEFACTION ANALYSES

As previously described, the site is located within a mapped State of California Seismic Hazard Zone (CGS, 2000) for areas that may be susceptible to liquefaction.

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

We analyzed potential liquefaction based on the CPT data and the computer software CLiq (Version 3.3.2.9) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). For our analysis, we considered a Maximum Considered Earthquake peak ground acceleration adjusted for site class effects (PGA_M) of 0.94g with an expected earthquake magnitude of 6.5, based on Figure 3.4 presented in the Seismic Hazard Report (CGS, 2000), which estimates the most probable moment magnitude earthquake within the site vicinity. To evaluate saturated soil zones at the site that could potentially undergo liquefaction, we used a high groundwater depth of 10 feet bgs. This groundwater depth is based on mapped historical high groundwater levels (CGS, 2000) in the vicinity of the site.

Based on the interpreted subsurface conditions from the CPTs, including fine-grained and coarse-grained soil beneath the groundwater table, we evaluated the susceptibility of “clay-like” and “sand-like” soil with the CLiq software using multiple analysis procedures, including methodology presented by Roberston (2009) and methodology described by Boulanger & Idriss (2014). To assess seismically induced settlements, we considered the methodology presented by Zhang et al. (2002). We summarize the results of our analysis in Table 2.7-1, and discuss the findings in Section 3.1.3.

TABLE 2.7-1: Total Estimated Liquefaction-Induced Settlement

EXPLORATION ID	SETTLEMENT (INCHES)	
	ROBERTSON (NCEER, 2009)	BOULANGER & IDRISS (2014)
1-CPT1	¾	½
1-CPT2	1¼	½
1-CPT3	1½	1

3.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based on our findings and experience, it is our opinion that the types of developments under consideration for the Pecten site are feasible, provided the anticipated geologic and geotechnical constraints are appropriately mitigated. The primary geological and geotechnical concerns that could affect development are liquefaction, compressible soil, potentially expansive soil, and existing fill. We summarize our preliminary conclusions below.

3.1 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 and liquefaction. Common secondary seismic hazards include ground shaking, liquefaction, and lateral spreading. The following sections present a discussion of these and other hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lurching, or landslides, is considered low to negligible at the site, and are not discussed further.

3.1.1 Ground Rupture

As discussed in Section 2.2.2, the site is not located in the Santa Clara County Fault Rupture Hazard Zone or an Alquist-Priolo Hazard Zone. It is our opinion that the risk for potential surface rupture of faults is considered low.

3.1.2 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 (refer to Figure 5). To mitigate the shaking effects, all 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. Structures designed to mitigate the effects of ground shaking should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some nonstructural damage, and (3) resist major earthquakes without collapse, but with some structural as well as nonstructural damage.

Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.1.3 Liquefaction / Cyclic Softening

Based on the findings discussed in Section 2.7, our preliminary liquefaction analysis indicates that the total seismic-induced ranges between ½ an inch to 1½ inches across the site.

The obtained CPT data were used to perform a preliminary liquefaction analysis; however, a more rigorous liquefaction hazard evaluation and settlement analysis should be performed during the design-level exploration. The design-level assessment should focus on further evaluating the susceptibility of fine-grained soil to cyclic softening by performing additional laboratory testing and screening using criteria presented by Bray and Sancio (2006) or similar. The design-level assessment should also consider groundwater depths based on proposed site grades, when they are available.

3.1.4 Lateral Spreading

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

Based on our evaluation of liquefaction potential, and the relatively flat topography in the vicinity of the site with no open face, it is our opinion that the risk of lateral spreading will not significantly impact land development.

3.2 COMPRESSIBLE SOIL

Soil may be subject to settlement when loaded with a new structure or additional fill. This settlement may occur as elastic or consolidation settlement. Elastic settlement is a function of soil stiffness while consolidation settlement is highly dependent on the amount of water-filled voids within the soil. The rate of settlement is highly dependent on the permeability of the soil and the presence of water. Consequently, sandy soil will typically settle almost immediately, whereas clayey soil below the water table will settle much more slowly.

Based on the obtained subsurface data, the soft to stiff fine-grained soil discussed in Section 2.5.2 may be potentially compressible when new building loads or fills are introduced. To evaluate the potential for compressible soil, we used the CPT data to perform a preliminary analysis (using CPeT-IT analysis software) to estimate load-induced settlement. We assumed that structural loads would be moderate for the five-story building. Our analysis assumed a mat foundation system, an average bearing pressure of 1,000 psf, and that future site grades at the time of construction will be similar to existing grades.

Based on our preliminary assessment, we estimate up to 2½ inches of static (load-induced) settlement. This estimated settlement value is intended to provide an understanding of soil response considering building loads, and potential load-induced static settlement risk; it should not be considered a design estimate. Load-induced settlement does not include the settlement from liquefaction discussed in Section 2.7.

Based on the results from our preliminary analysis, the estimated static settlement may exceed the typical tolerance for some of the structures under consideration (e.g. multi-story residential buildings). We recommend additional laboratory testing and analysis for consolidation settlement be performed during the design-level study. The estimate for total load-induced settlement may be refined with further laboratory testing and analysis. The design-level analysis should incorporate planned building geometries and loading conditions when they are provided by the project Structural Engineer.

3.3 EXISTING FILL

Based on a review of available subsurface data and historical aerial images, we anticipate existing fill may be encountered at the site. Since historical records of fill placement were not available at the time of our report, we assume the existing fill is non-engineered. Non-engineered fill may undergo excessive settlement, especially under new fill or building loads. The presence of existing subsurface obstructions may also lead to damaging differential settlement. The depth and extent of existing fill should be evaluated during the design-level exploration.

For preliminary planning purposes, existing fill should be overexcavated to a depth of competent material, as determined by a representative of our firm, and reconstructed with engineered fill. We provide preliminary earthwork recommendations in Section 4.3.

3.4 EXPANSIVE SOIL

Laboratory testing on the near-surface sample yielded a PI value of 21, which generally corresponds to moderate shrink/swell potential.

Expansive soil changes 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. Our CPTs encountered predominantly fine-grained soil at depths anticipated to underlie structure foundations, floor slabs, etc., that could be moderately to highly expansive. For this reason, additional exploration and laboratory testing should be performed during the design-level study to further evaluate expansive soil.

3.5 2019 CALIFORNIA BUILDING CODE PARAMETERS

The 2019 CBC utilizes design criteria set forth in the 2016 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 3.5-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.5-1: 2019 CBC Seismic Design Parameters, Latitude: 37.412476 **Longitude: -**121.881051

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.022
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.777
Site Coefficient, F_A	1.0
Site Coefficient, F_V	See ASCE Section 11.4.8

PARAMETER	VALUE
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.022
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	See ASCE Section 11.4.8
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.348
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	See ASCE Section 11.4.8
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.85
Site Coefficient, F_{PGA}	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.935

Considering the mid-rise multi-family residential or commercial development, we estimate the fundamental periods of the proposed structures to be less than 1.5*Ts*. Therefore, the structural engineer may consider exception of Section 11.4.8 of ASCE 7-16 as follows.

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with *S1* greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with Eq. (12.8-3) of ASCE 7-16 for $1.5T_S < T \leq T_L$."

If the noted exception is not used, a ground motion hazard analysis can be provided upon request in a separate letter.

3.6 SOIL CORROSION POTENTIAL

Our scope did not include sampling and testing for corrosion potential as part of this preliminary study. Representative samples of the foundation grade soil should be obtained during the design-level study to determine the potential for corrosion on buried metal and the potential for sulfate attack on foundation concrete and buried pipelines. If necessary, a corrosion consultant can determine if special design considerations are necessary in accordance with the 2019 California Building Code.

4.0 PRELIMINARY RECOMMENDATIONS

Based on the findings, it is our opinion that site conditions are generally suitable for the proposed development, provided the preliminary recommendations in this report are incorporated in planning and design.

4.1 GROUND IMPROVEMENT

If anticipated static settlements exceed design tolerances for the proposed structures, ground improvement may be considered to reduce the settlement and improve the bearing capacity for support of shallow foundation loads. Based on our experience, ground improvement methods such as deep soil mixing, drill displacement columns, or rammed aggregate piers may be effective in mitigation of liquefaction- and load-induced settlement. Additionally, if ground improvements are incorporated, the reinforced earth mat may be omitted depending on the ground improvement method and design.

Soil improvement is typically procured as a design-build element of a project after the design-level geotechnical exploration has been completed. The selection and design of the ground improvement system is commonly determined with input from an experienced general contractor. This allows consideration of contractors' proprietary means and methods in selecting a cost-effective approach that meets specific project performance and quality objectives. Conceptual ground improvement plans would typically show the extent of the work, coordination with other elements, including foundations, utilities, and project phasing requirements.

4.2 FOUNDATION RECOMMENDATIONS

For planning purposes, we recommend the foundation consist of a rigid post-tensioned mat, or spread footings in combination with ground improvement for the warehouse or 5-story residential structure under consideration. The foundation system should be designed to resist the potential shrink/swell of the soil (or other measures be implemented to mitigate expansive soils). The preliminary design of the foundation should tolerate 2½ inches of total post-construction load-induced static settlement and 1½ inches of total seismically induced settlement. The differential for static and seismic settlement can each be taken as one-half the total estimate over the width of the building, or a lateral distance of 30 feet, whichever is less.

For the residential building under consideration, a total static load-induced settlement of 2½ inches is typically beyond the tolerance of the structure and for improvements that are connected to the building. For planning purposes, we recommend the apartment buildings be supported on either a rigid mat foundation or spread footings overlying ground improvement. It should be anticipated that slabs-on-grade should be underlain by layers of non-expansive material such as sand or aggregate base, 18 to 24 inches thick. For preliminary design, an allowable dead-plus-live bearing pressure of 1,000 psf may be used for mat foundations. As an alternative, the structure may be supported on a rigid mat foundation at a lower grade elevation to reduce the net loading on the foundation subgrade soils by balancing structural loads with removed overburden soils.

4.3 EARTHWORK

Site development should commence with the removal of any existing buried structures, including utilities and foundations. Areas where non-engineered fill are identified should be overexcavated to a competent subgrade, as evaluated by a representative of our firm, and reconstructed with engineered fill. All debris should be removed from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow.

After removing existing fill, the contractor should scarify to a depth of at least 12 inches, then moisture condition and compact the subgrade in accordance with the table below. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

For planning purposes, we summarize preliminary fill compaction requirements in Table 4.3-1.

TABLE 4.3-1: Preliminary Fill Placement Requirements

FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM)
General Fill	90	3
Pavement Areas	95	1

We recommend additional laboratory testing be performed during the design-level study to determine index properties for the near-surface soil. These properties will be used to support earthwork operations and can be used by the project-grading contractor to help in project bidding.

5.0 FUTURE STUDIES

This report presents preliminary geotechnical findings, conclusions, and recommendations intended for preliminary planning purposes only. A design-level geotechnical exploration and assessment should be performed when development plans are available. The design-level geotechnical report should further discuss topics presented in this report and address the following items.

- Additional field exploration and laboratory testing to support design-level recommendations
- Design-level analyses related to geologic and geotechnical hazards should further evaluate the following
  - Refinement of liquefaction analysis and cyclic softening potential; consolidation settlement of potentially compressible layers
  - Static-load-induced elastic and consolidation settlement based on structure foundation loading
  - Groundwater conditions and variations in static water level in relation to proposed site grades
  - Expansive potential of near-surface soil
- Design recommendations for suitable foundations alternatives
- Design-level earthwork, ground improvement design (if needed), and construction recommendations

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, either 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.

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

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SELECTED REFERENCES

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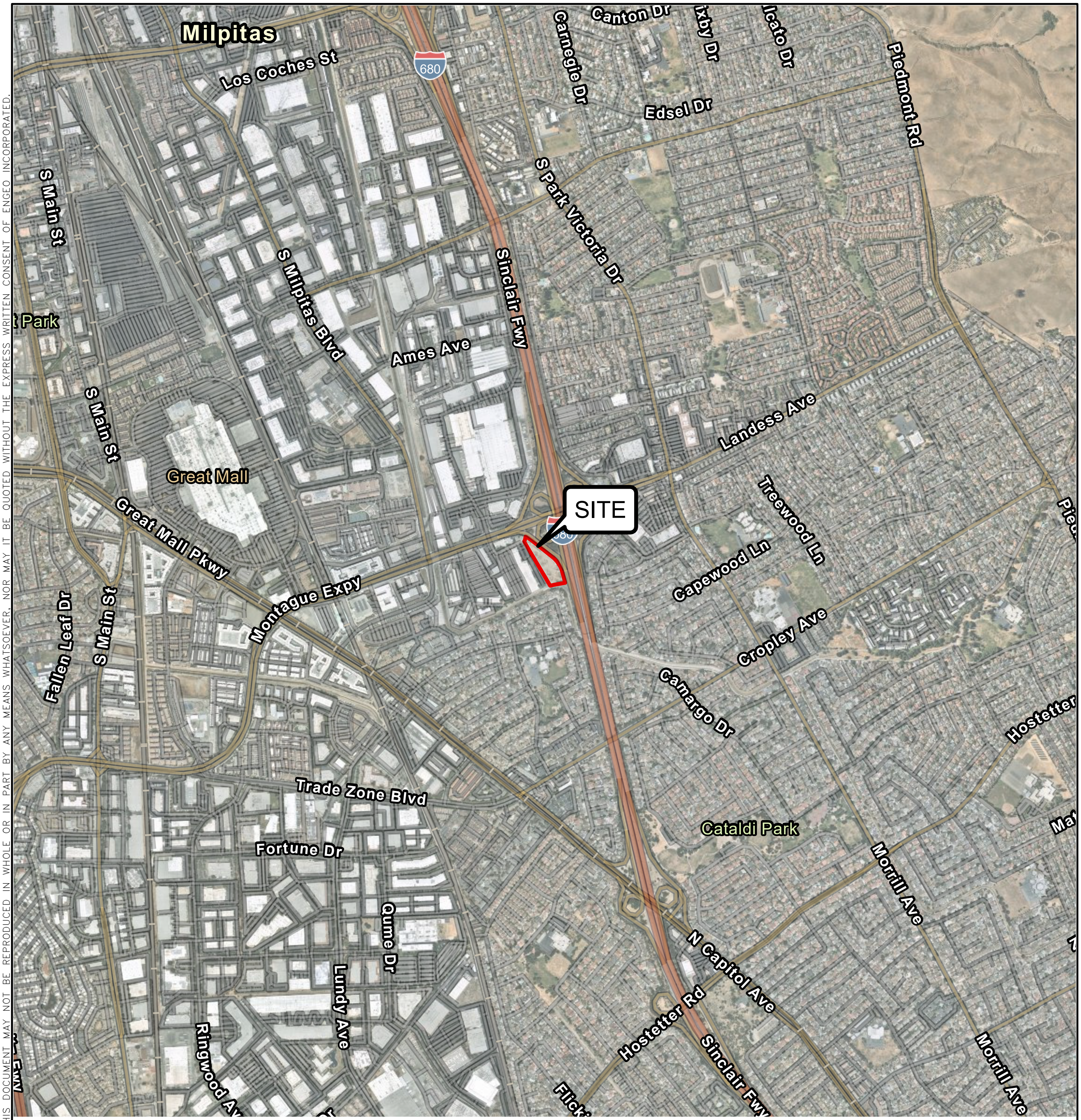
FIGURES



N

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FEET

BASEMAP SOURCE: NEARMAP MAPPING SERVICE 5/22/2021

ENGEO Expect Excellence	VICINITY MAP PECTEN SITE MILPITAS/SAN JOSE, CALIFORNIA	PROJECT NO. : 19078.000.001 SCALE: AS SHOWN DRAWN BY: QRL / CHECKED BY: MV	FIGURE NO. 1

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19078\19078000001\GEOTECH\GEX\PECTEN SITE.APRX
LAYOUT: VICINITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT3 CONE PENETRATION TEST (ENGEO, 2021)

N

0 50 100

FEET

BASEMAP SOURCE: NEARMAP MAPPING SERVICE 5/22/2021

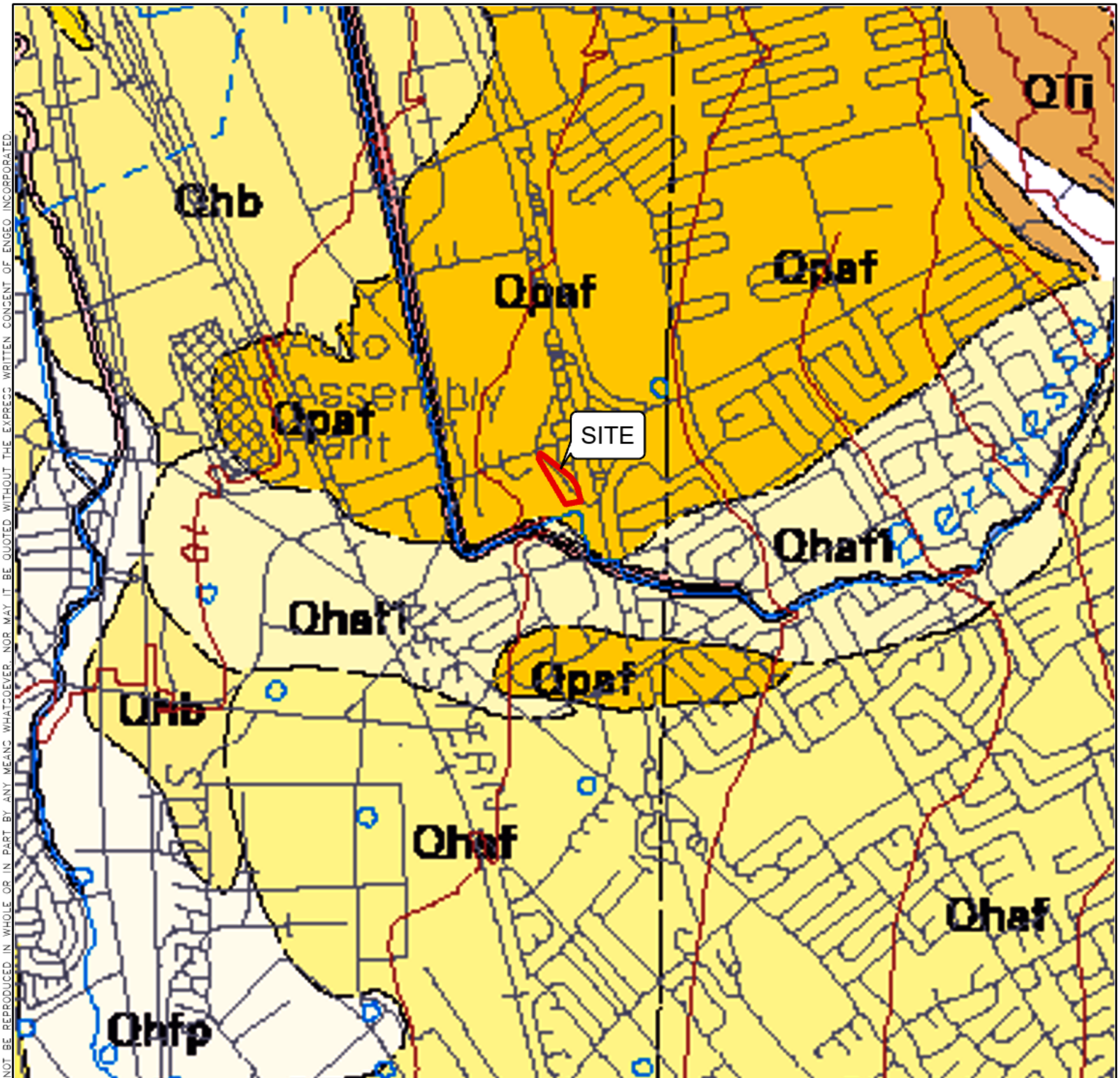
ENGEO
Expect Excellence

SITE PLAN
PECTEN SITE
MILPITAS/SAN JOSE, CALIFORNIA

PROJECT NO. : 19078.000.001	FIGURE NO. 2
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: MV

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19078\19078000001\GEOTECH\GEX\PECTEN SITE.APRX
LAYOUT: SITE PLAN USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qhaf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhaf1	ALLUVIAL TERRACE DEPOSITS (HOLOCENE)
Qhb	BASIN DEPOSITS (HOLOCENE)
Qhfp	FLOODPLAIN DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN DEPOSITS (PLEISTOCENE)
QTi	IRVINGTON GRAVELS (PLEISTOCENE AND/OR PLIOCENE)

N

0 1,000 2,000

FEET

GRAYMER, 1997

ENGEO Expect Excellence

REGIONAL GEOLOGIC MAP

PECTEN SITE

MILPITAS/SAN JOSE, CALIFORNIA

PROJECT NO.: 19078.000.001

SCALE: AS SHOWN

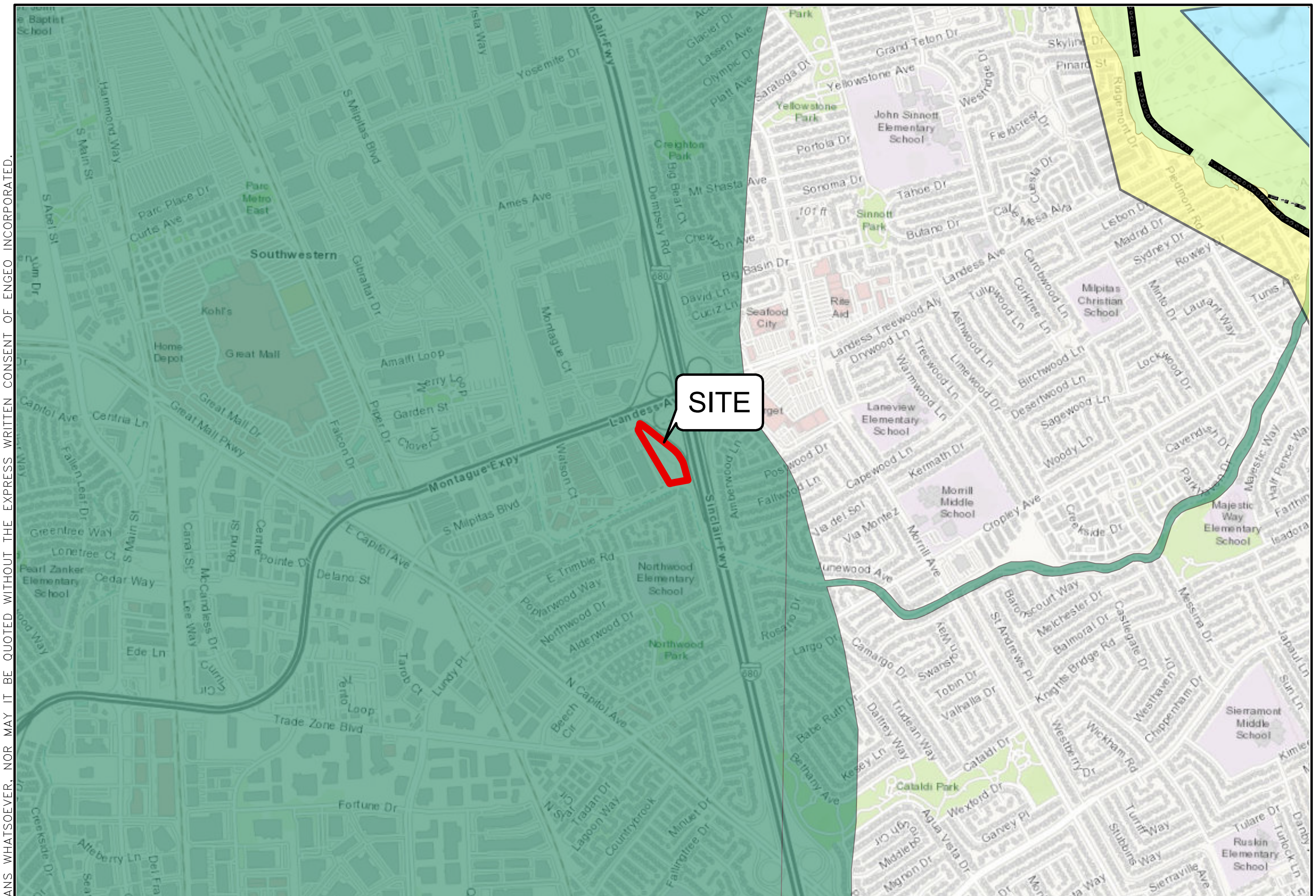
DRAWN BY: QRL

CHECKED BY: MV

FIGURE NO. 3

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19078\19078000001\GEOTECH\GEX\PECTEN SITE.APRX
LAYOUT: GEOLOGIC USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

FAULT TRACES

INFERRED

CONCEALED

EARTHQUAKE FAULT ZONE
ZONE BOUNDARIES ARE DELINEATED BY STRAIGHT-LINE SEGMENTS; THE BOUNDARIES DEFINE THE ZONE ENCOMPASSING ACTIVE FAULTS THAT CONSTITUTE A POTENTIAL HAZARD TO STRUCTURES FROM SURFACE FAULTING OR CREEP SUCH THAT AVOIDANCE AS DESCRIBED IN PUBLIC RESOURCES CODE SECTION 2621.5(A) WOULD BE REQUIRED

EARTHQUAKE-INDUCED LANDSLIDE ZONES
AREAS WHERE THE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED.

LIQUEFACTION ZONE
AREAS WHERE THE HISTORICAL OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTEHNICAL AND GROUND WATER CONDITIONS INDICATE A POTENTIAL L FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

N

0 1,000 2,000
FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

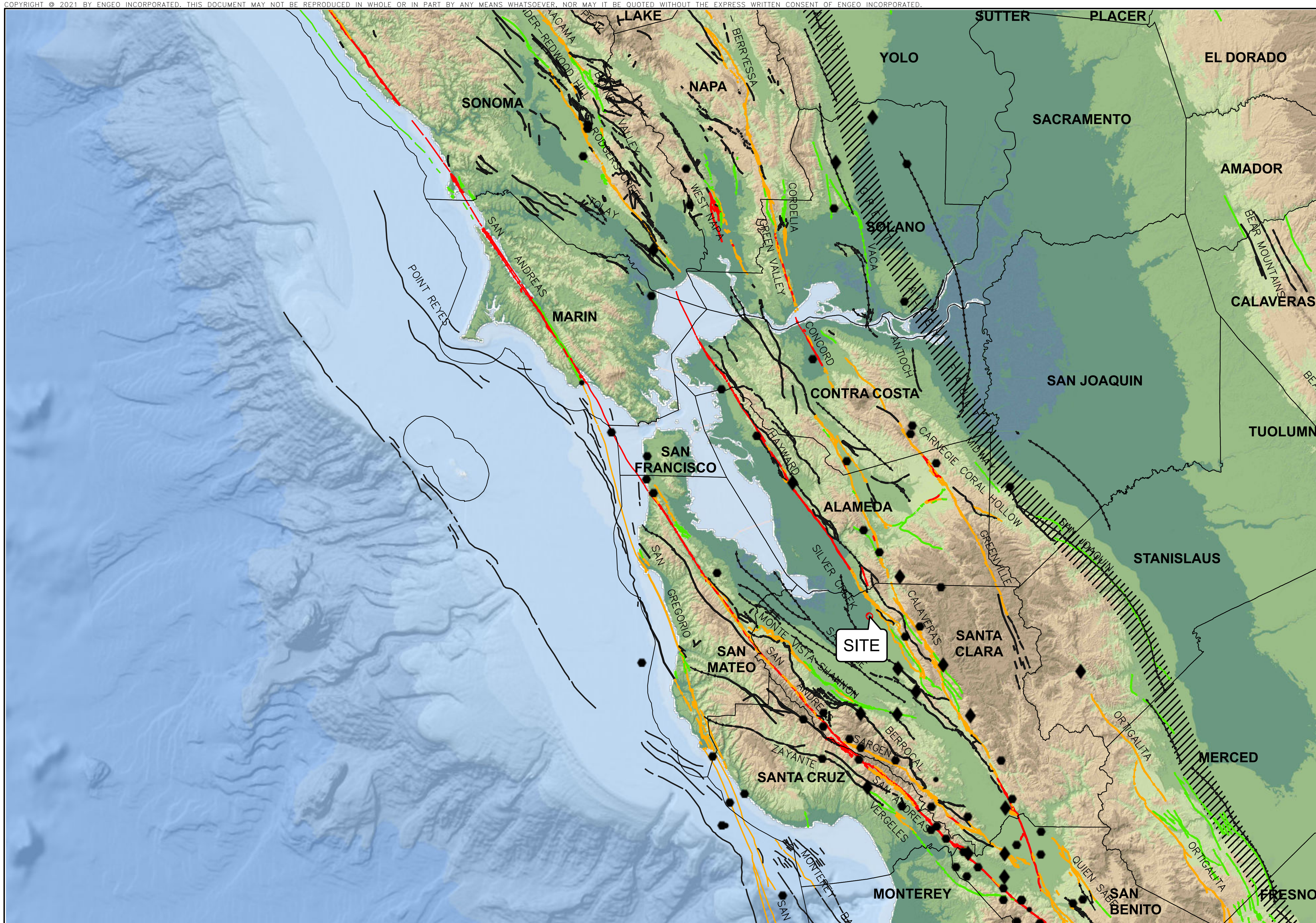
ENGEO
Expect Excellence

SEISMIC HAZARDS ZONE MAP
PECTEN SITE
MILPITAS/SAN JOSE, CALIFORNIA

PROJECT NO. : 19078.000.001	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: MV

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19078\19078000001\GEOTECH\GEX\PECTEN SITE.APRX
LAYOUT: SEISMIC HAZARD USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



N

0 7.5 15
Miles

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL (<150 YEARS)
- LATEST QUATERNARY (<15,000 YEARS)
- LATE QUATERNARY (<130,000 YEARS)
- UNDIFFERENTIATED QUATERNARY (<1.6 MILLION YEARS)
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY MAP
PECTEN SITE
MILPITAS/SAN JOSE, CALIFORNIA

PROJECT NO.: 19078.000.001	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: MV

PATH: G:\DRAFTING\PROJECTS_18000 TO 19999\19078\19078000001\GEOTECH\GEX\PECTEN SITE.APRX
LAYOUT: FAULT&SEISMICITY USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

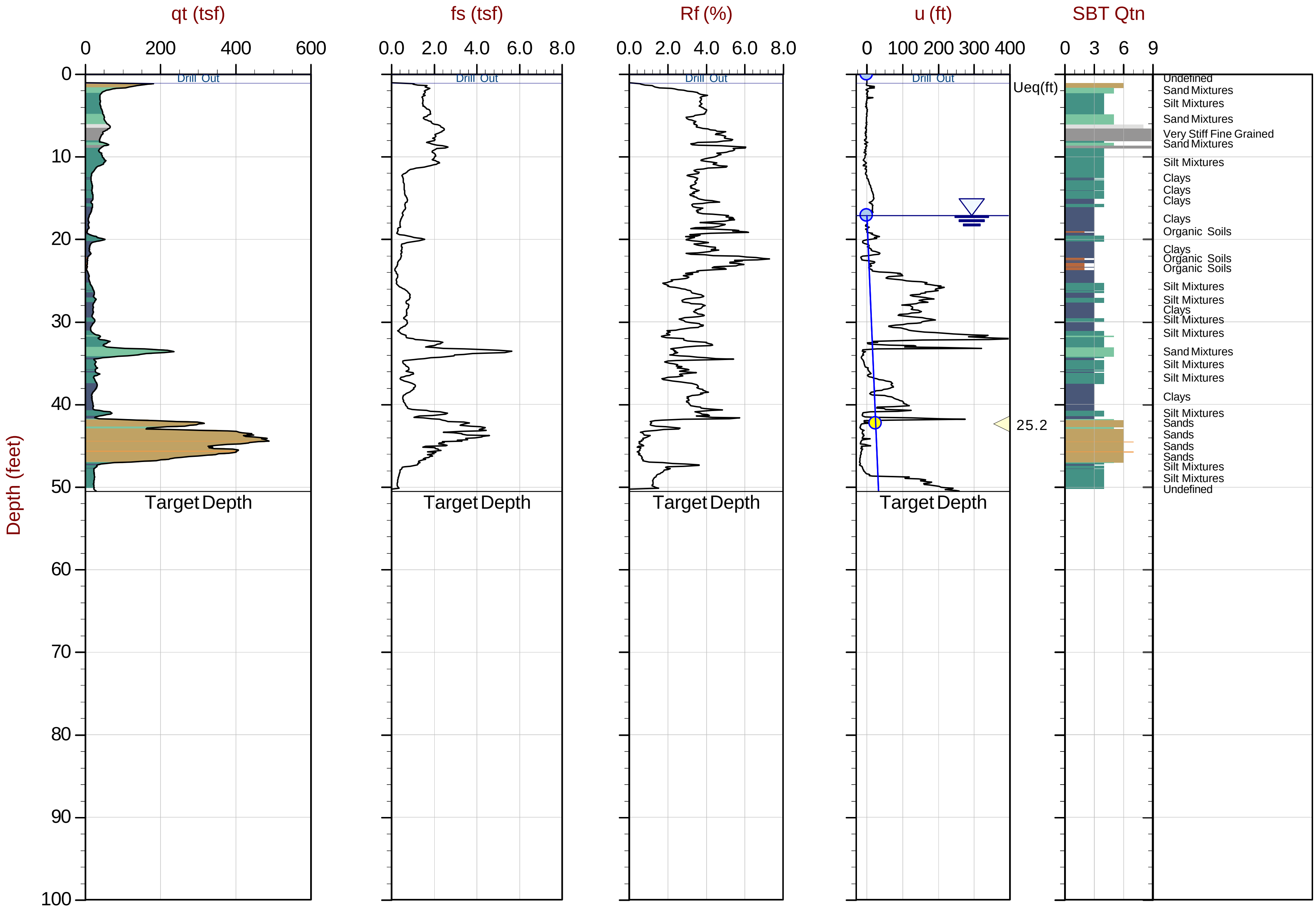
CONE PENETRATION TEST DATA

ConeTec

ENGEO

Job No: 21-56-22923
Date: 2021-08-27 18:00
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT1
Cone: 537:T1500F15U500



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

File: 21-56-22923_CP-1102-01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4141137m E: 599042m

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

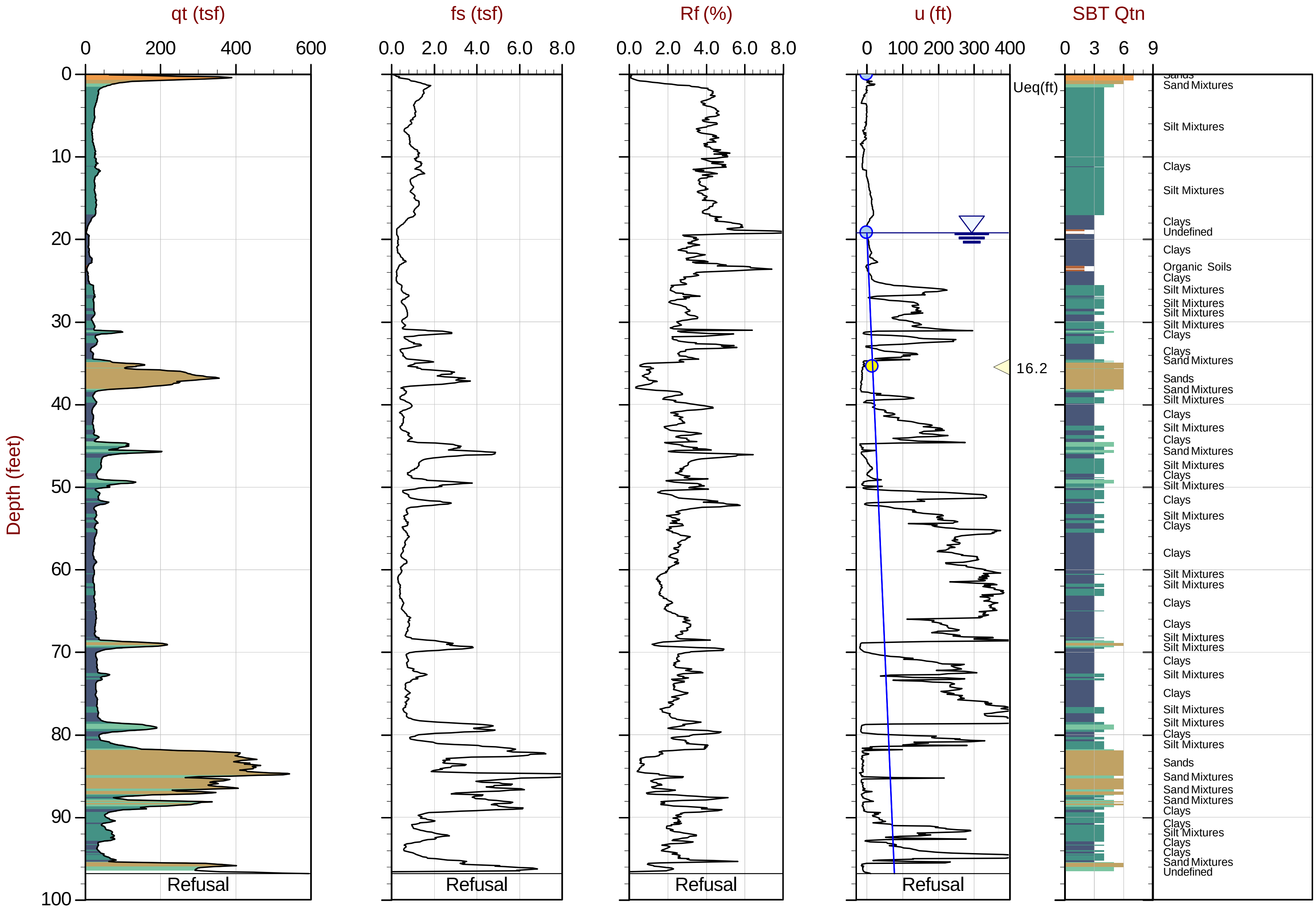
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-22923
Date: 2021-08-27 15:08
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT2
Cone: 537:T1500F15U500



Max Depth: 29.525 m / 96.87 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-22923_SP-1102-02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4141231m E: 599023m

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

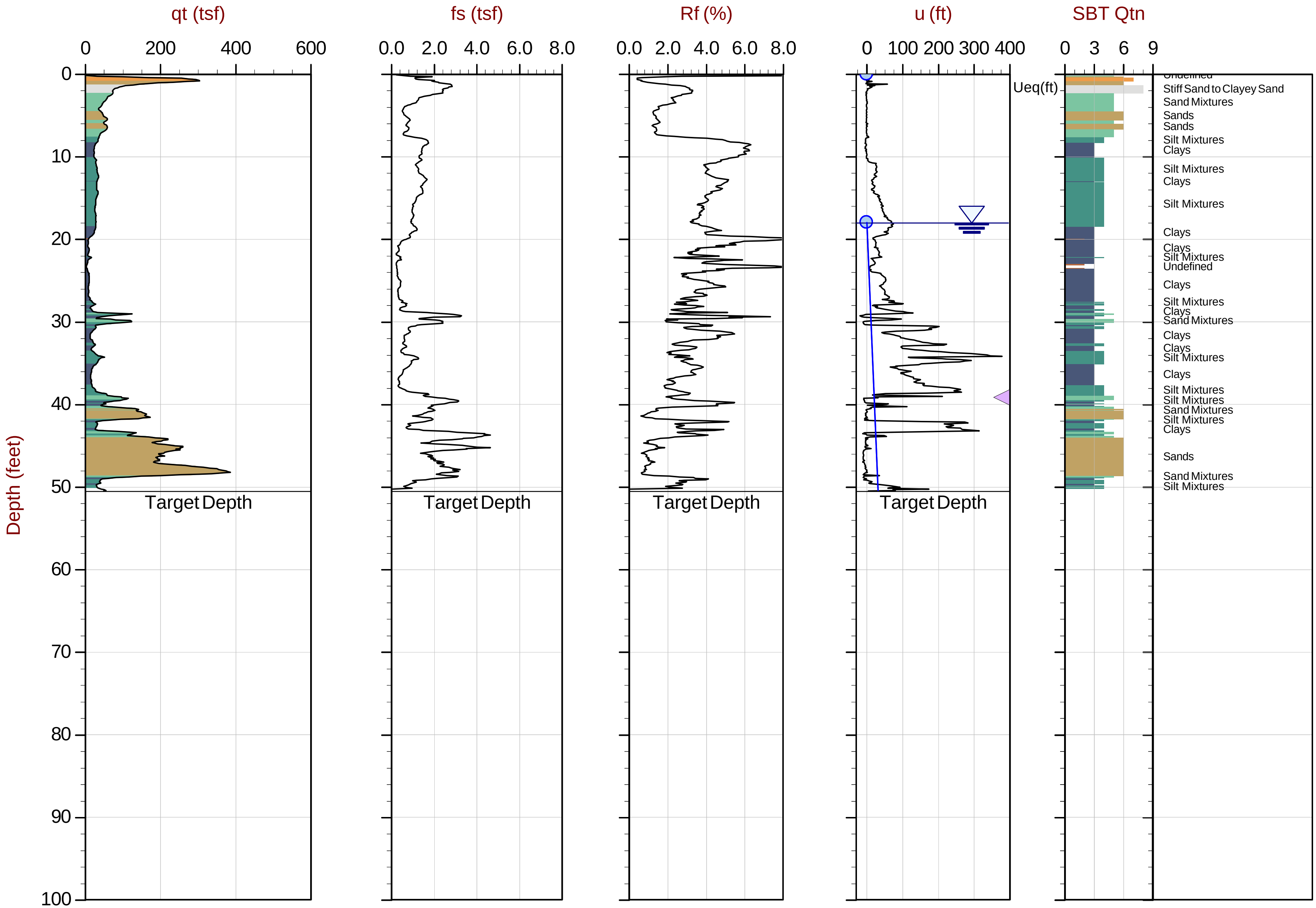
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 21-56-22923
Date: 2021-08-27 17:07
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT3
Cone: 537:T1500F15U500



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

File: 21-56-22923_CP-1102-03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4141270m E: 598967m

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.

Job No:	21-56-22923
Client:	ENGEO
Project:	1102 Pecten Ct Milpitas, CA
Sounding ID:	1-CPT2
Date:	08:27:21 15:08
Seismic Source:	Beam
Seismic Offset (ft):	2.10
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
2.56	1.90	2.83			
5.81	5.15	5.56	2.73	7.38	370
9.02	8.37	8.63	3.06	5.39	568
12.37	11.71	11.90	3.27	3.92	836
15.58	14.93	15.08	3.18	4.14	768
18.96	18.31	18.43	3.35	4.02	835
22.24	21.59	21.69	3.26	4.46	731
25.53	24.87	24.96	3.27	3.75	872
28.81	28.15	28.23	3.27	3.47	942
32.09	31.43	31.50	3.27	3.69	887
35.27	34.61	34.68	3.18	2.76	1151
38.65	37.99	38.05	3.37	2.97	1136
41.93	41.27	41.33	3.28	3.69	888
48.49	47.83	47.88	6.55	7.33	895
55.02	54.36	54.40	6.52	6.17	1057
58.30	57.64	57.68	3.28	3.47	945
61.58	60.93	60.96	3.28	3.75	875
64.86	64.21	64.24	3.28	3.80	863
68.14	67.49	67.52	3.28	3.69	890
71.42	70.77	70.80	3.28	3.48	944
74.71	74.05	74.08	3.28	3.78	867
77.99	77.33	77.36	3.28	3.63	905
81.27	80.61	80.64	3.28	2.91	1126
84.55	83.89	83.92	3.28	3.02	1086
87.83	87.17	87.20	3.28	2.20	1493
91.04	90.39	90.41	3.21	2.64	1219
94.39	93.73	93.76	3.35	2.75	1218
96.85	96.19	96.22	2.46	2.41	1020

CONETEC

Job No: 21-56-22923
Client: ENGEO Incorporated
Project: Valley Oak Partners Multiple Parcels, Santa Clara County
Start Date: 27-Aug-2021
End Date: 27-Aug-2021

CONE PENETRATION TEST SUMMARY

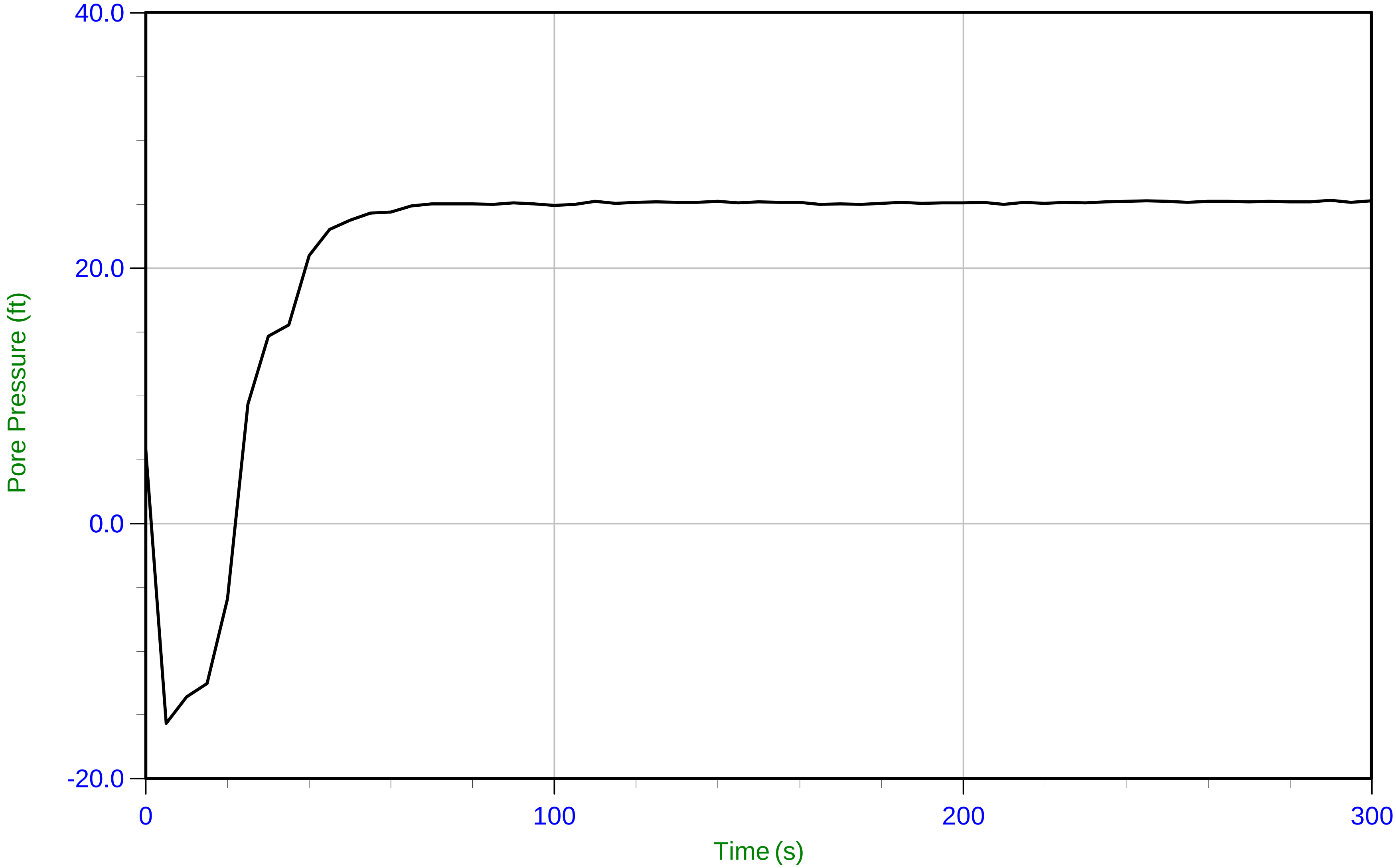
Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting[2] (m)	Elevation[3] (ft)	Refer to Notation Number
1-CPT1	21-56-22923_CP-1102-01	27-Aug-2021	537:T1500F15U500	17.1	50.52	4141137	599042	76	
1-CPT2	21-56-22923_SP-1102-02	27-Aug-2021	537:T1500F15U500	19.2	96.87	4141231	599023	77	
1-CPT3	21-56-22923_CP-1102-03	27-Aug-2021	537:T1500F15U500	18.0	50.52	4141270	598967	77	4
1-CPT1	21-56-22923_SP-320-01	27-Aug-2021	537:T1500F15U500	25.8	26.08	4126646	591984	205	
1-CPT2	21-56-22923_SP-320-02	27-Aug-2021	537:T1500F15U500	>22.6	22.56	4126737	592048	203	4
1-CPT3	21-56-22923_SP-320-03	27-Aug-2021	537:T1500F15U500	>16.4	16.32	4126693	592016	205	

1. The assumed phreatic surface was based on the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions are assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10S.
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface was based on the pore pressure dissipation tests at nearby soundings.

ENGEO

Job No: 21-56-22923
Date: 08/27/2021 18:00
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT1
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 21-56-22923_CP-1102-01.PPF
Depth: 12.900 m / 42.322 ft
Duration: 300.0 s

u Min: -15.7 ft
u Max: 25.3 ft
u Final: 25.3 ft

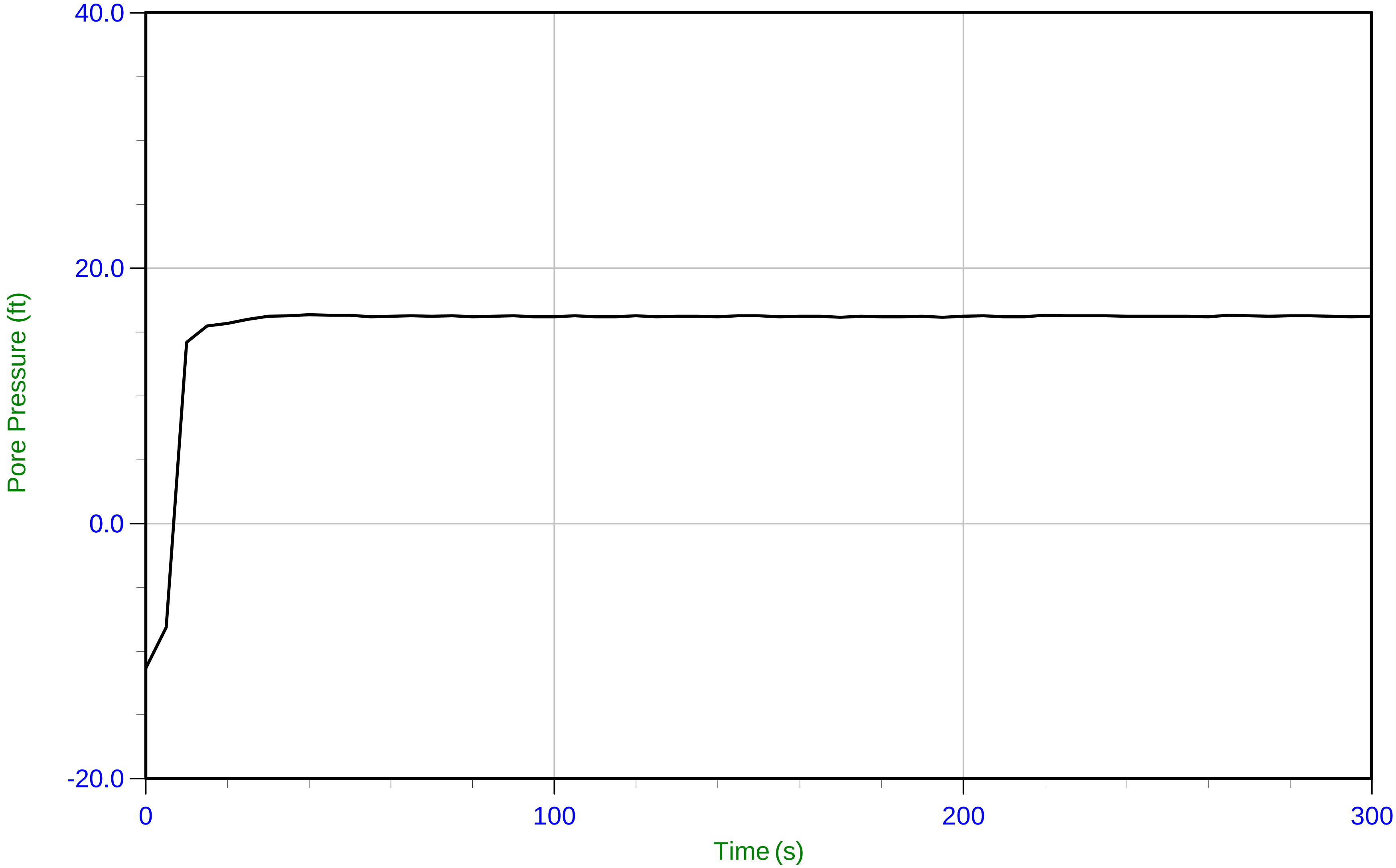
WT: 5.219 m / 17.122 ft
Ueq: 25.2 ft

ConeTec

ENGEO

Job No: 21-56-22923
Date: 08/27/2021 15:08
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT2
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 21-56-22923_SP-1102-02.PPF
Depth: 10.800 m / 35.433 ft
Duration: 300.0 s

u Min: -11.4 ft
u Max: 16.3 ft
u Final: 16.2 ft

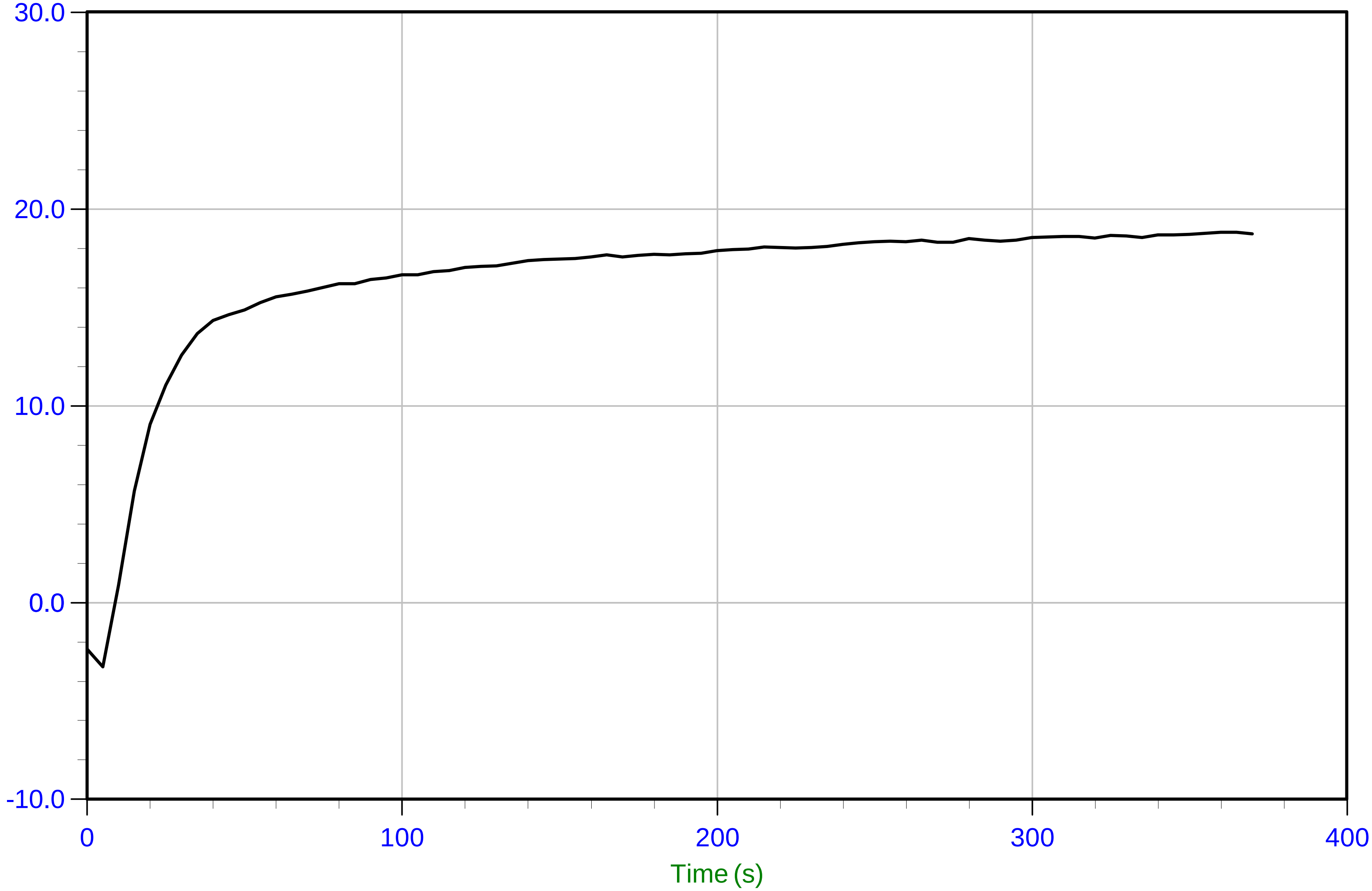
WT: 5.856 m / 19.212 ft
Ueq: 16.2 ft

CONETEC

ENGEO

Job No: 21-56-22923
Date: 08/27/2021 17:07
Site: 1102 Pecten Ct Milpitas, CA

Sounding: 1-CPT3
Cone: 537:T1500F15U500 Area=15 cm²



Trace Summary:

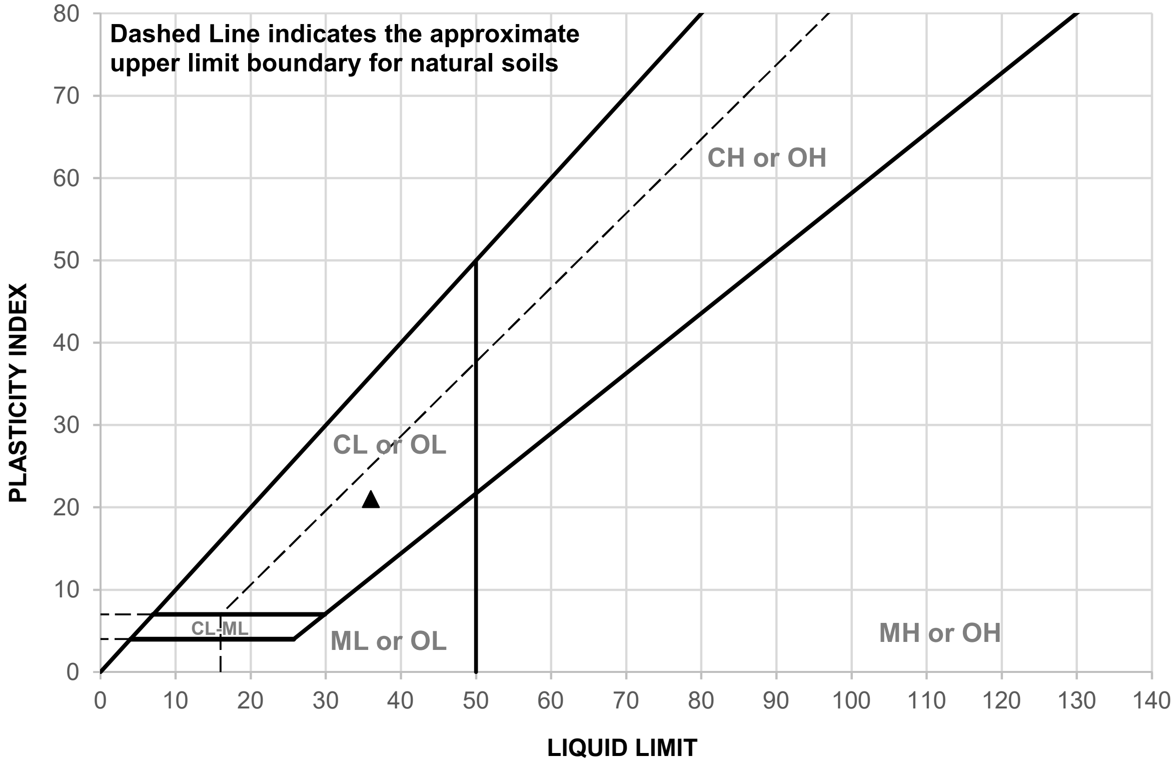
Filename: 21-56-22923_CP-1102-03.PPF
Depth: 11.925 m / 39.124 ft
Duration: 370.0 s

u Min: -3.3 ft
u Max: 18.8 ft
u Final: 18.7 ft

APPENDIX B

LABORATORY TEST DATA – PLASTICITY INDEX
ENGEO, 2021

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	Sample Point 1-S01	n/a	Dark brown sandy lean CLAY	36	15	21

	SAMPLE ID	TEST METHOD	REMARKS
▲	Sample Point 1-S01	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Valley Oaks Partners, LLC
PROJECT NAME: Valley Oak Partners Phase 1 ESAs
PROJECT NO: 19078.000.001 PH004
PROJECT LOCATION: Pecten Site
REPORT DATE: 9/9/2021
TESTED BY: G. Criste
REVIEWED BY: D. Seibold

APPENDIX C

LIQUEFACTION ANALYSIS

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

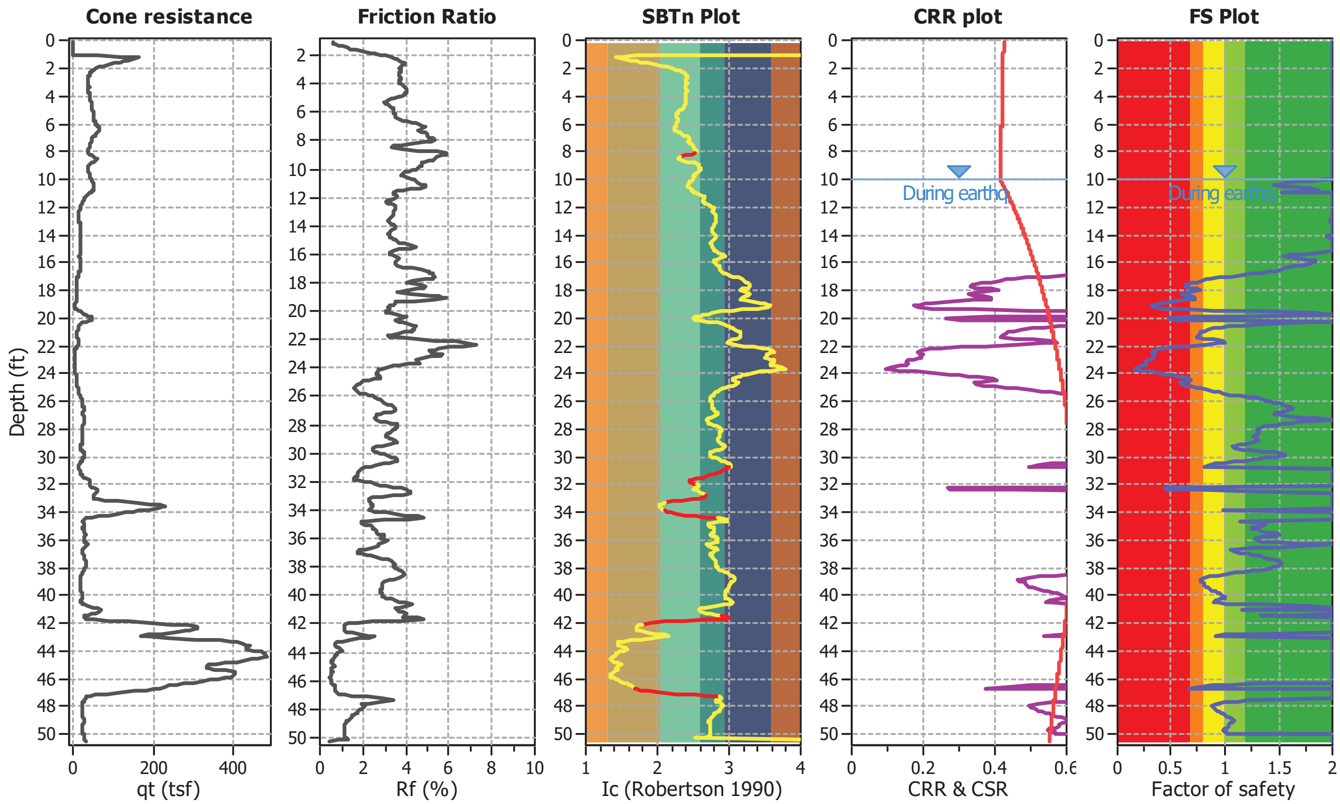
LIQUEFACTION ANALYSIS REPORT

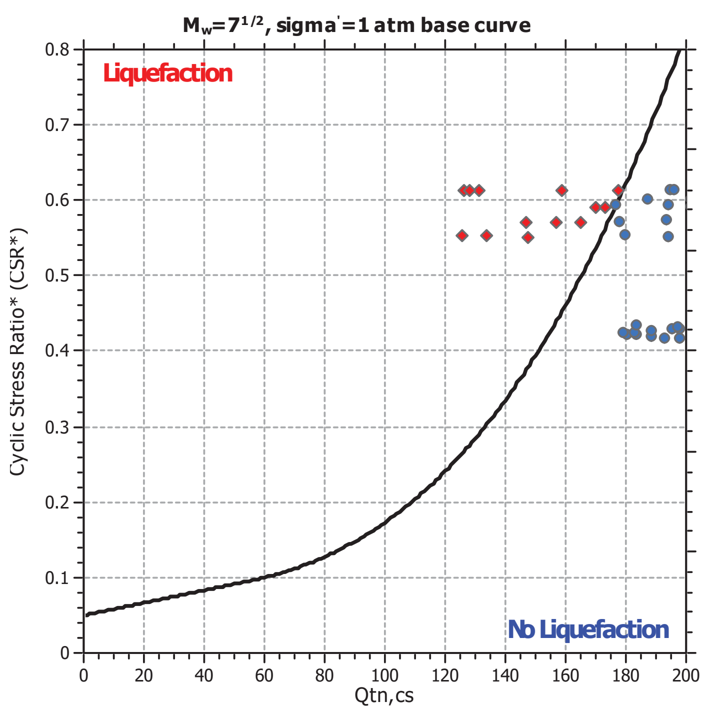
Project title : Pecten Site **Location : Milpitas/San Jose, CA**

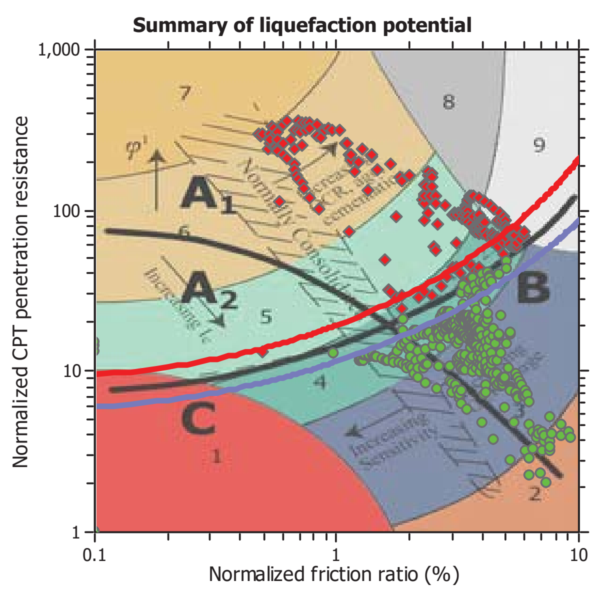
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	17.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.94	Unit weight calculation:	Based on SBT	K_σ applied:	No		

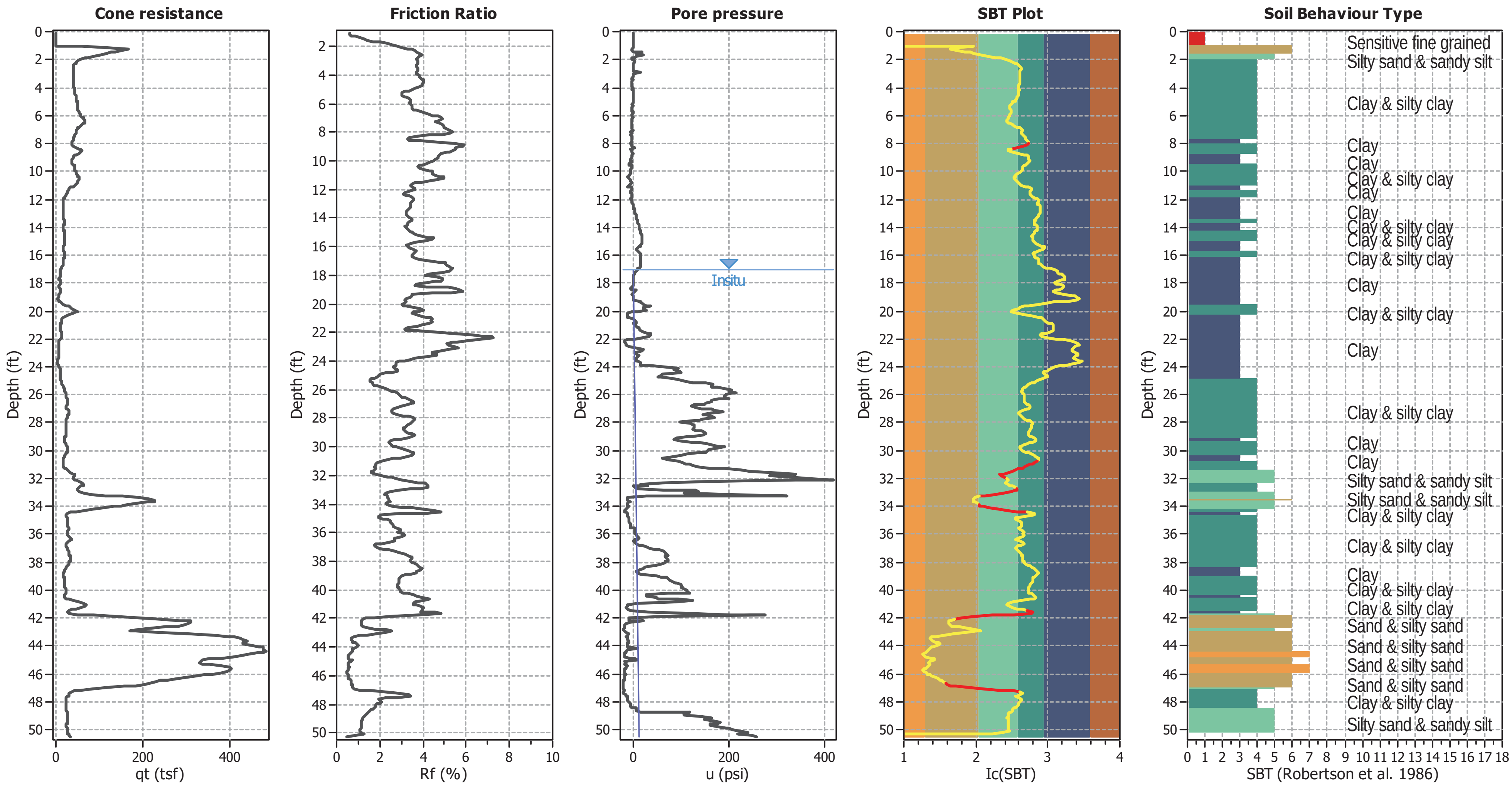






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



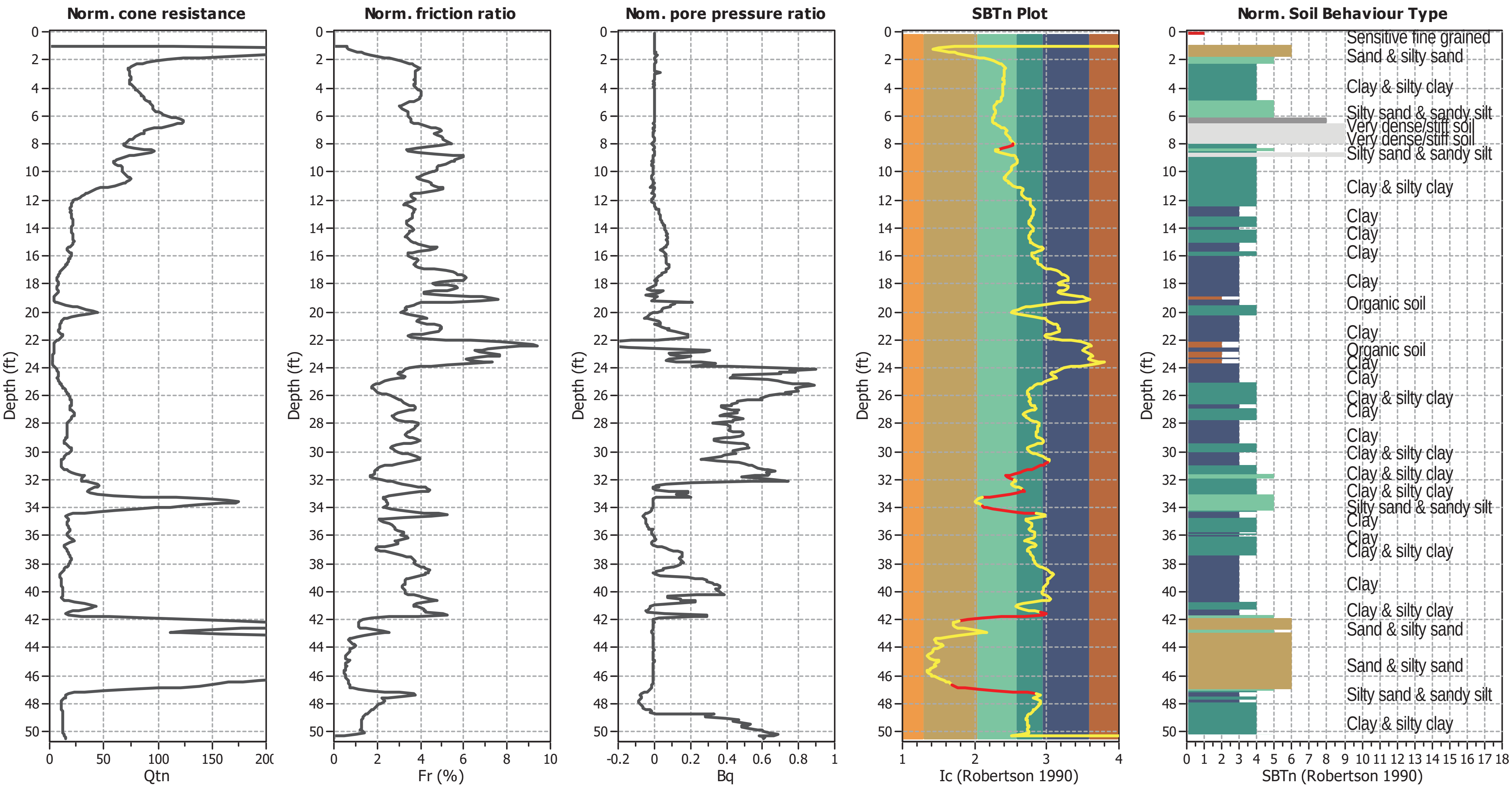
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 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)



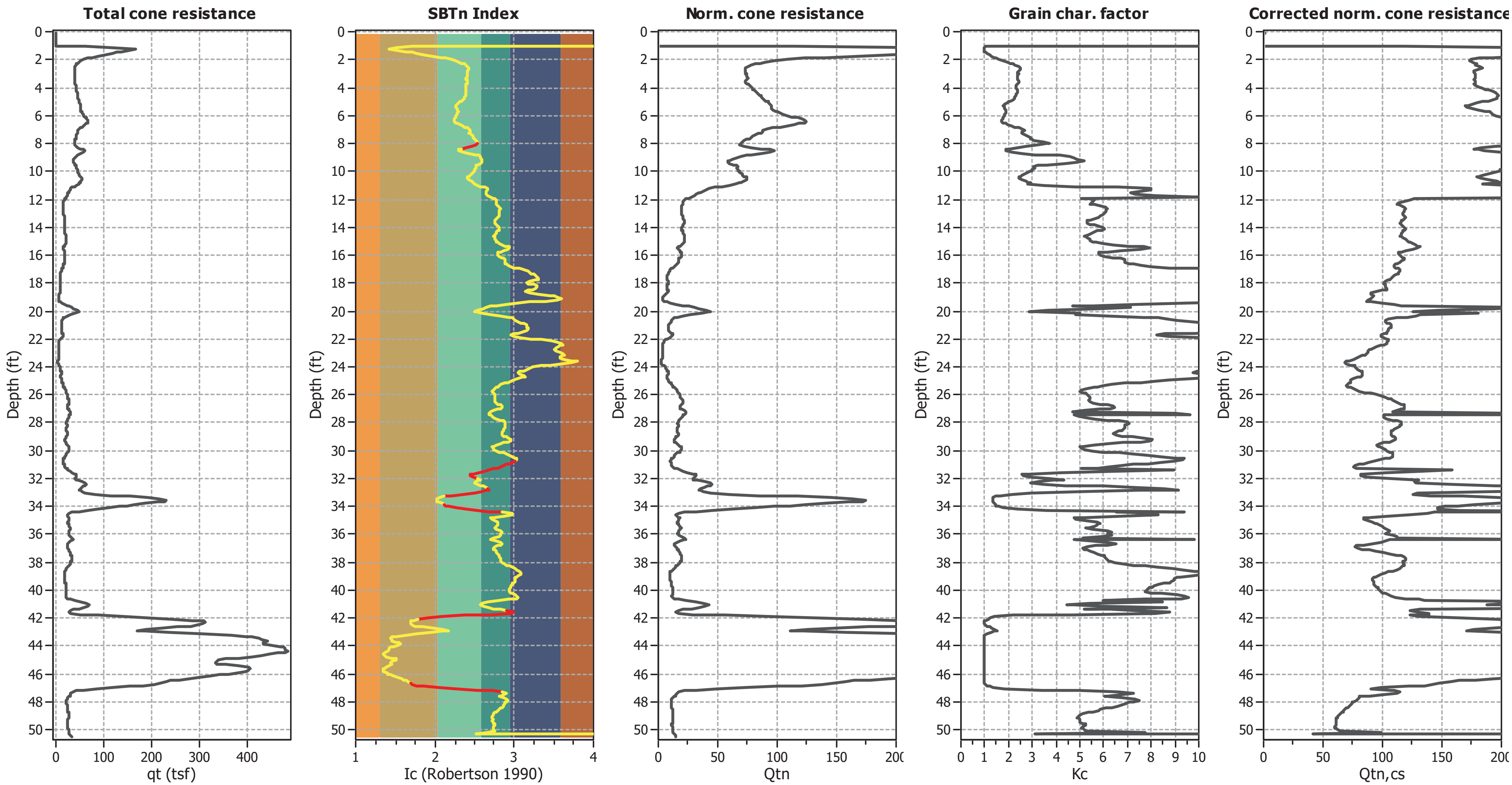
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 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

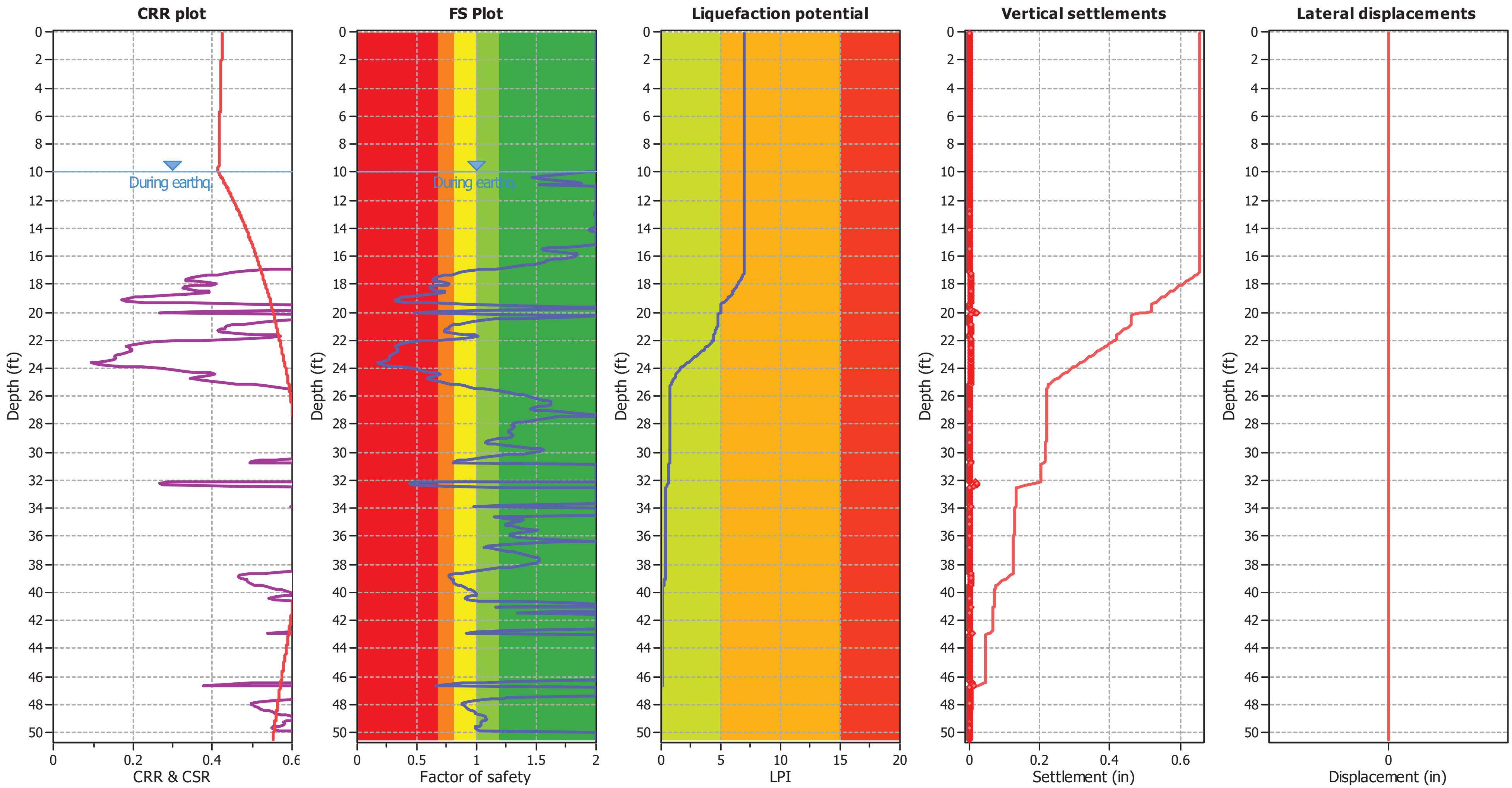
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

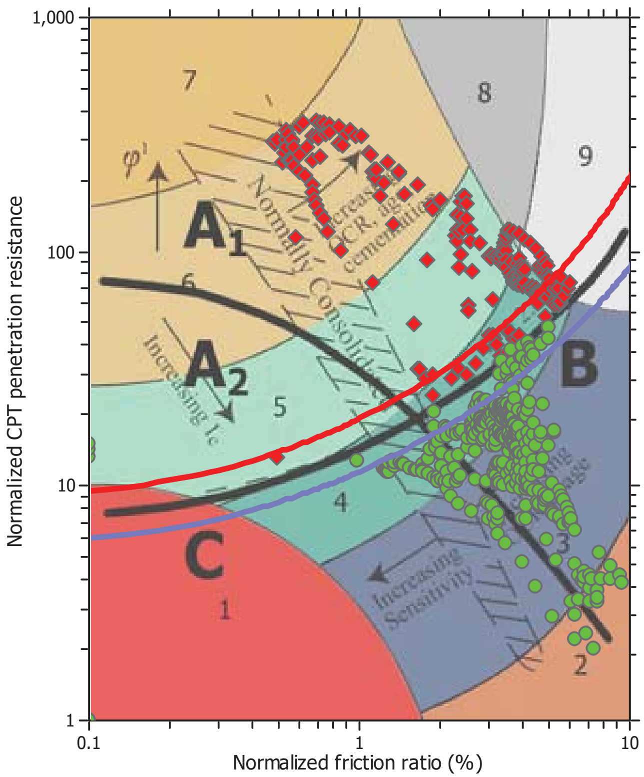
F.S. color scheme

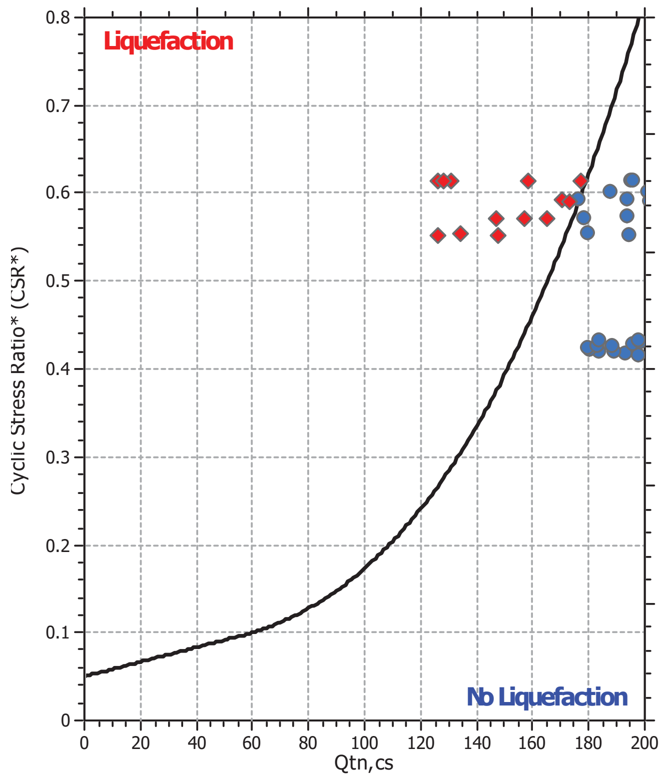
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

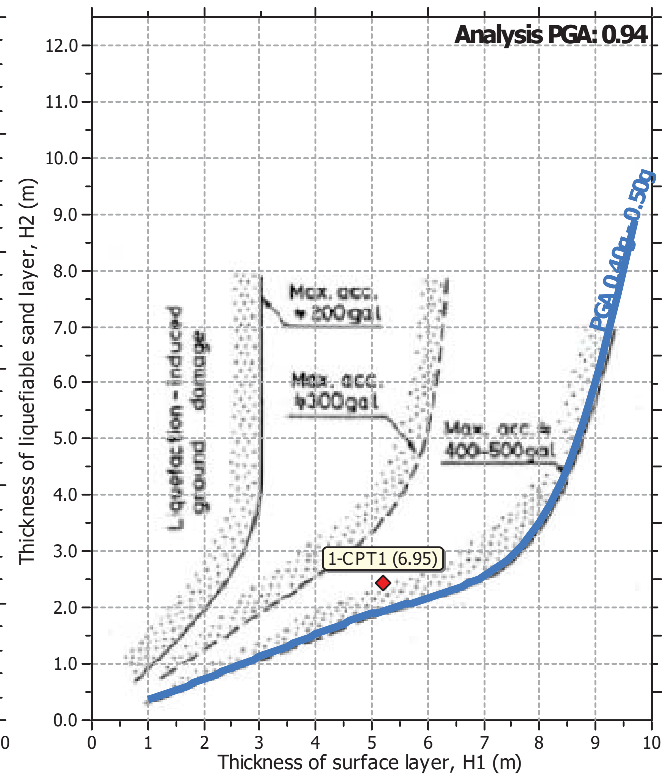
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



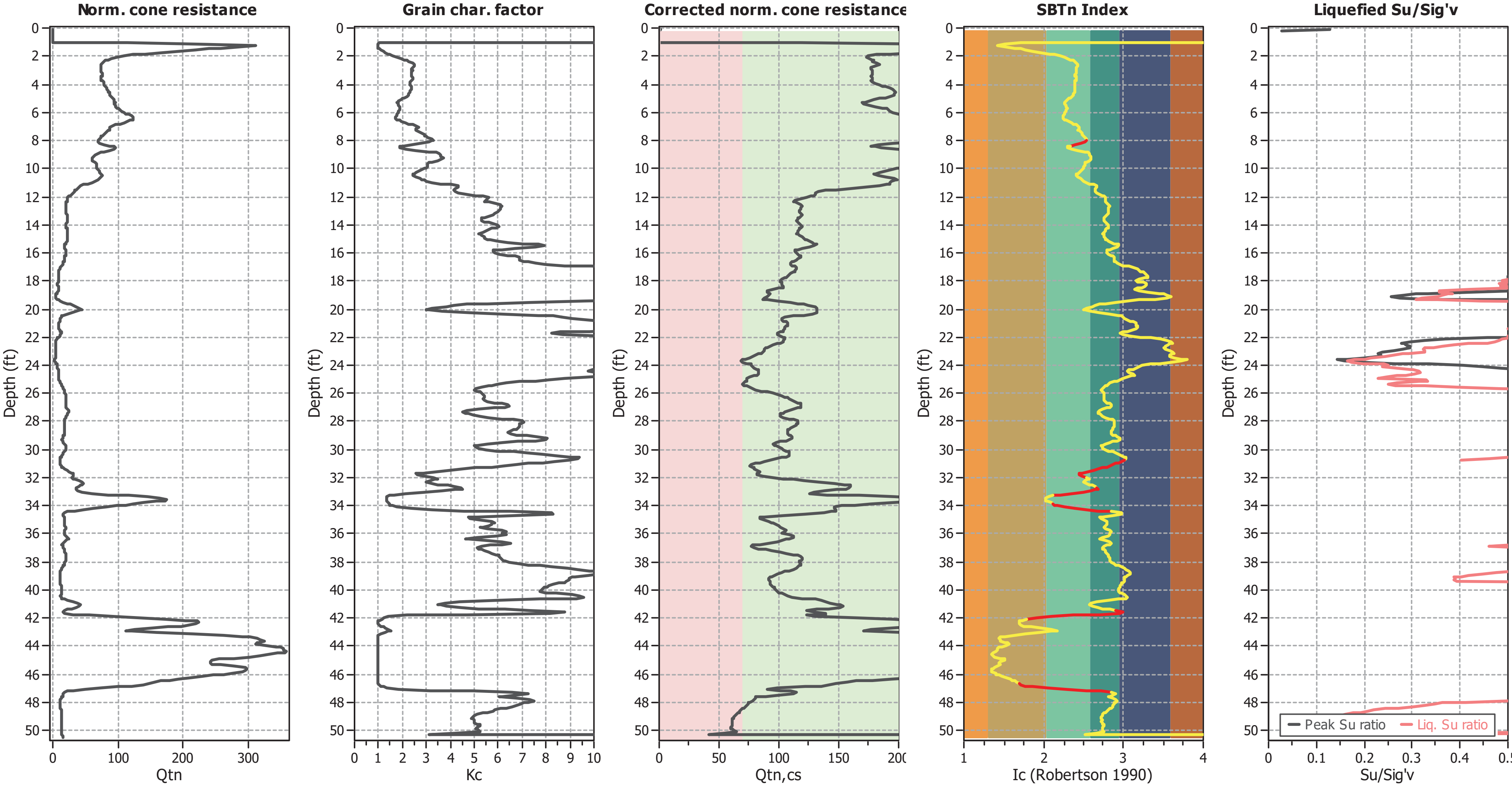




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

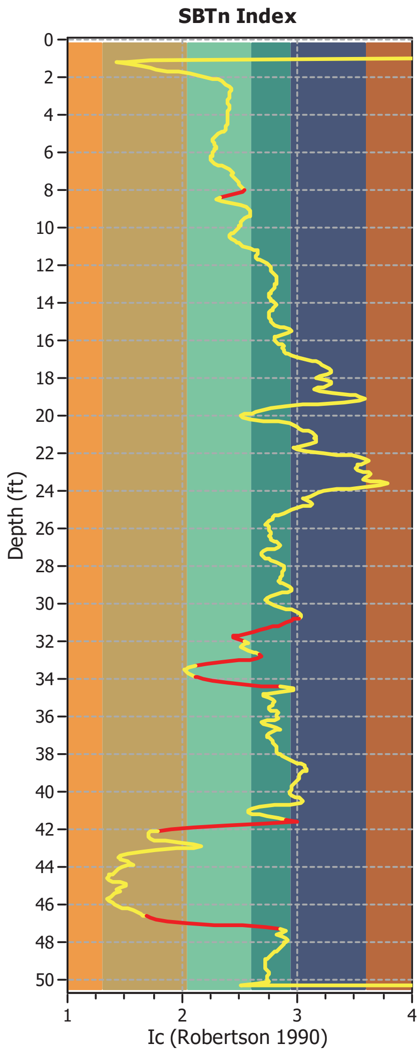
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

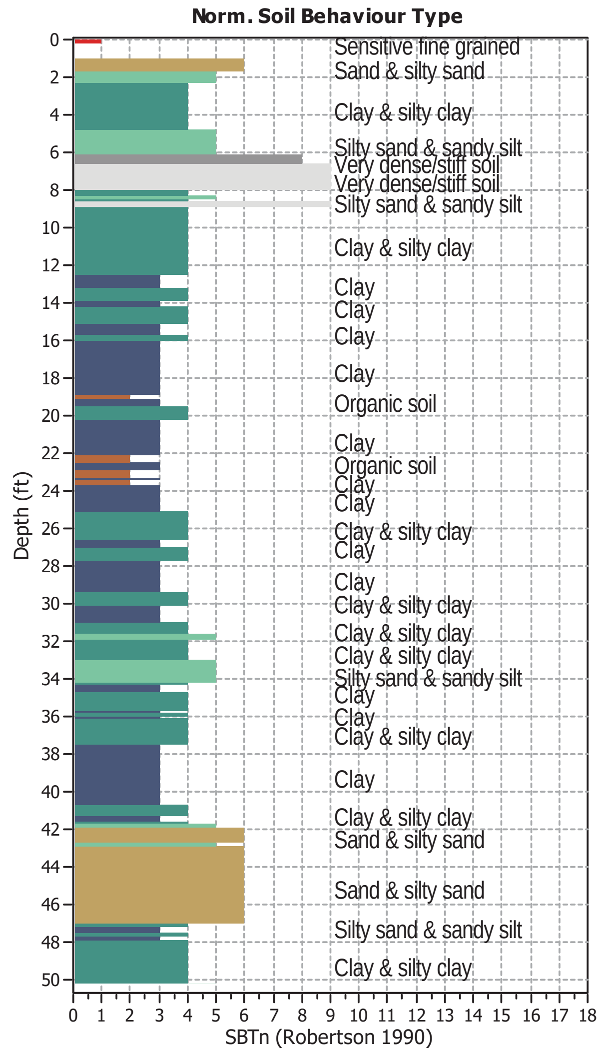
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

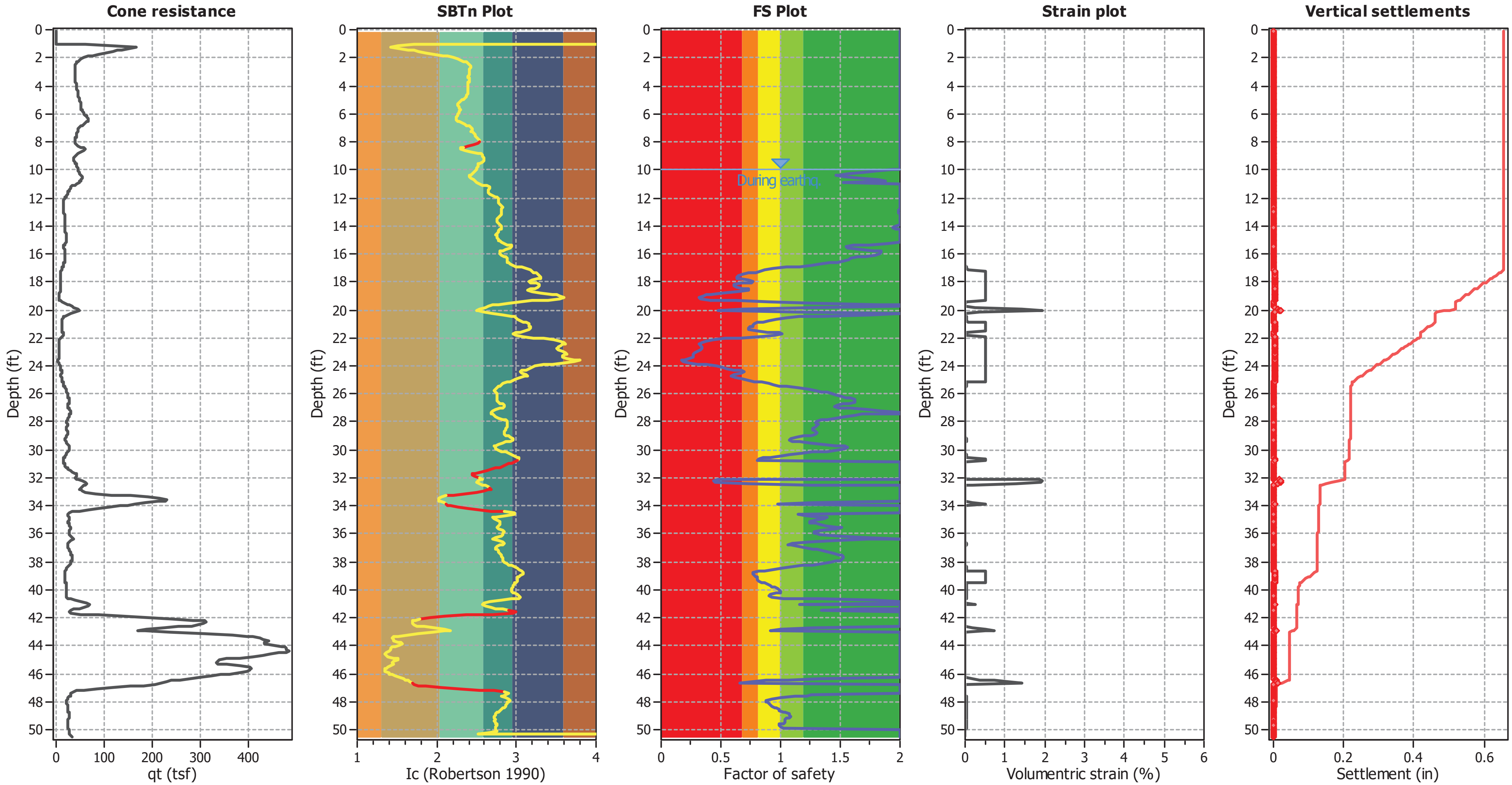
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	616
I_c maximum check value:	3.00	Total points excluded:	54
I_c change ratio value:	0.0250	Exclusion percentage:	8.77%
Minimum number of points in layer:	4	Number of layers detected:	7

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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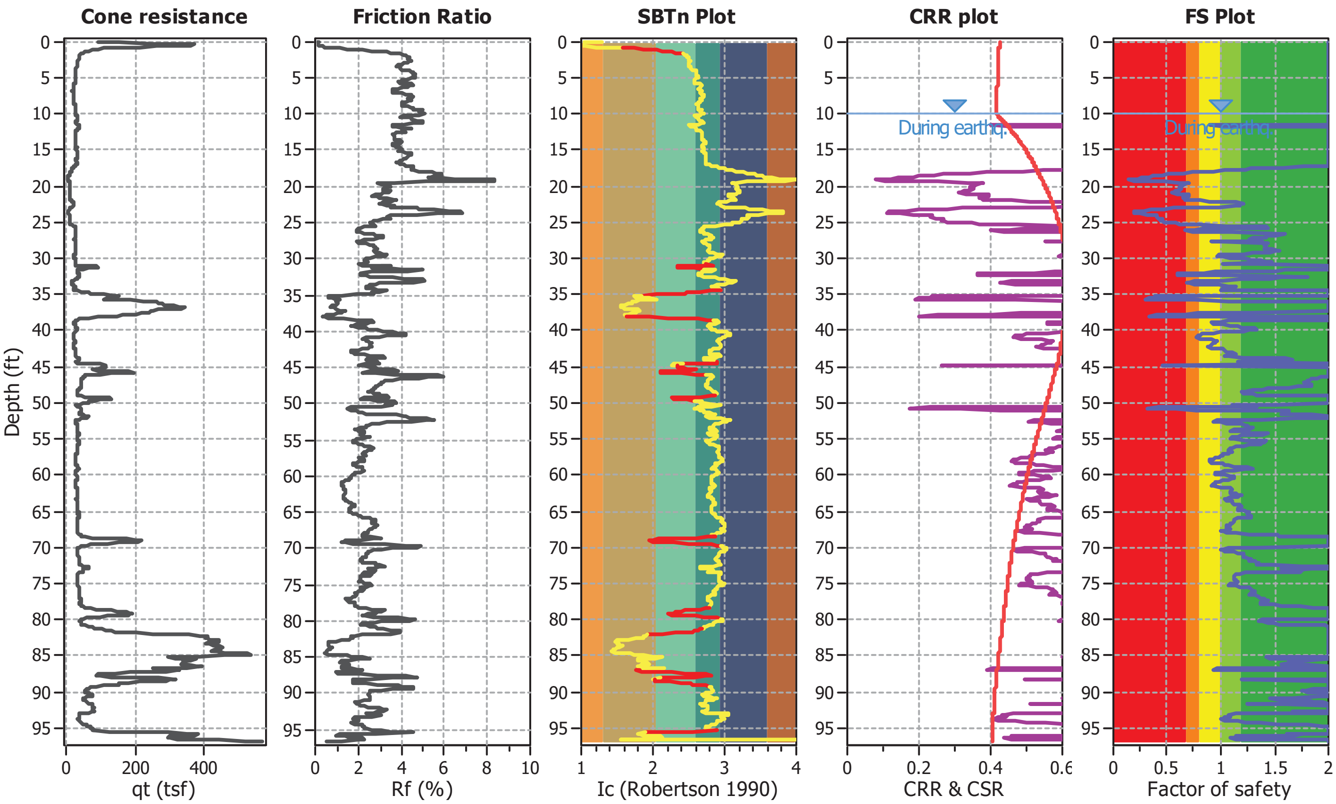
LIQUEFACTION ANALYSIS REPORT

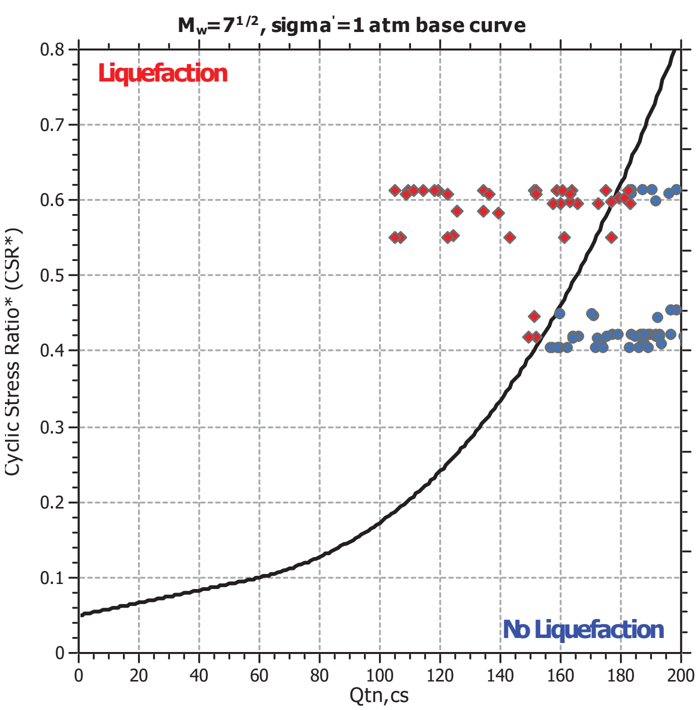
Project title : Pecten Site **Location : Milpitas/San Jose, CA**

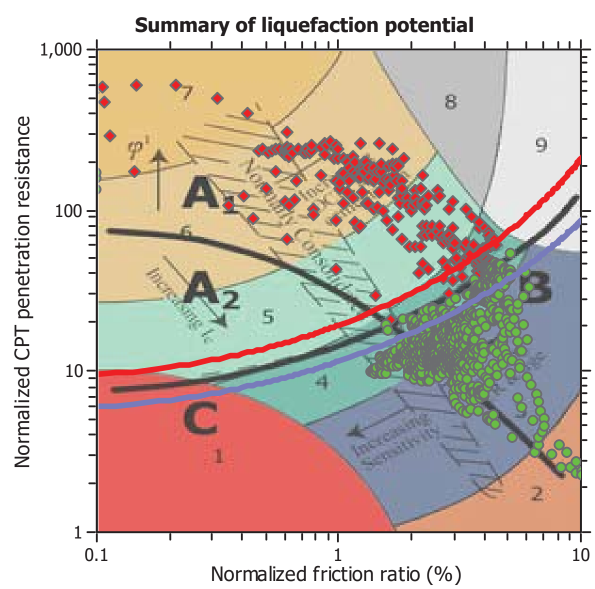
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	18.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 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:	100.00 ft
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.94	Unit weight calculation:	Based on SBT	K_σ applied:	No		

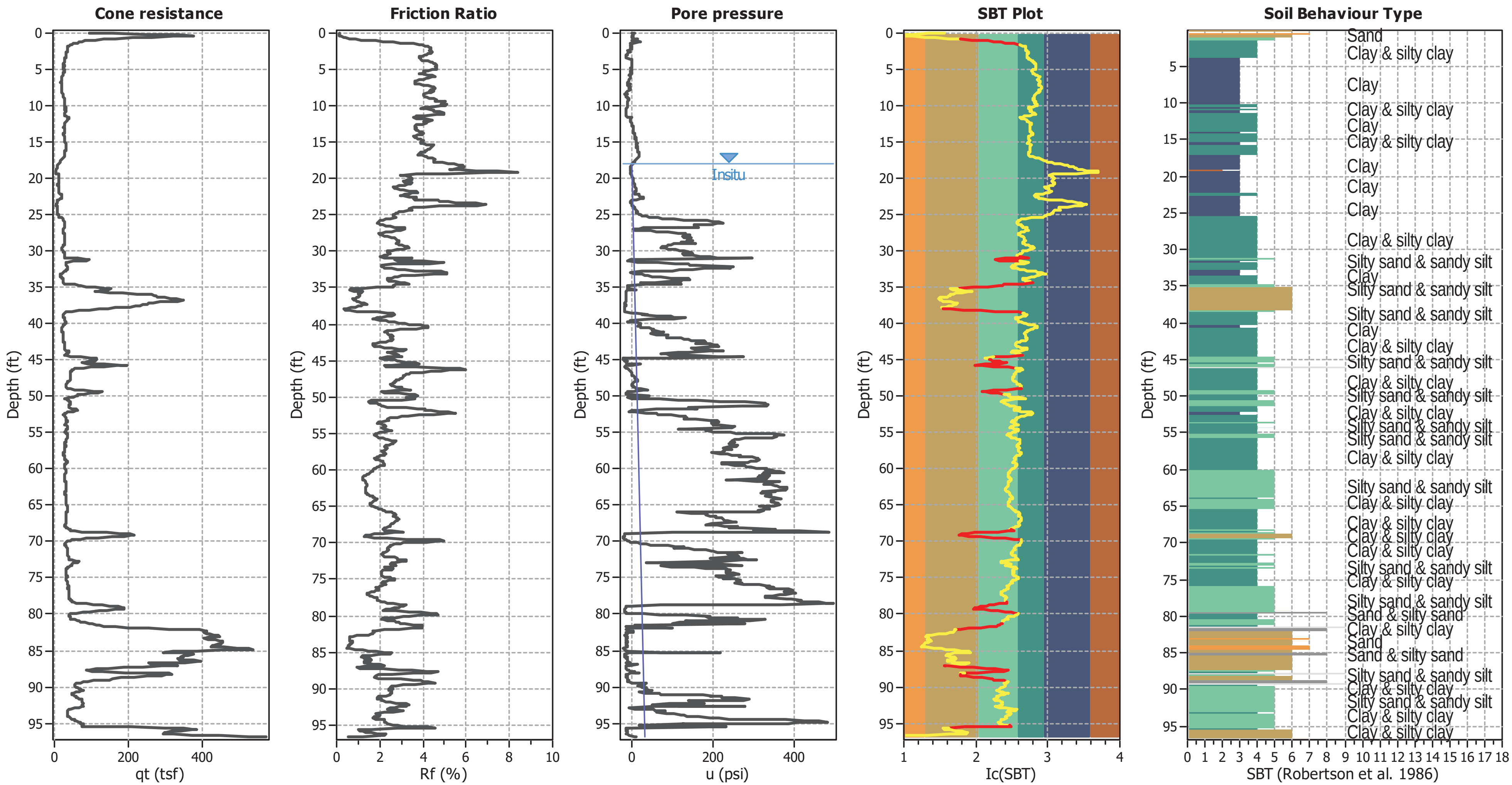






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



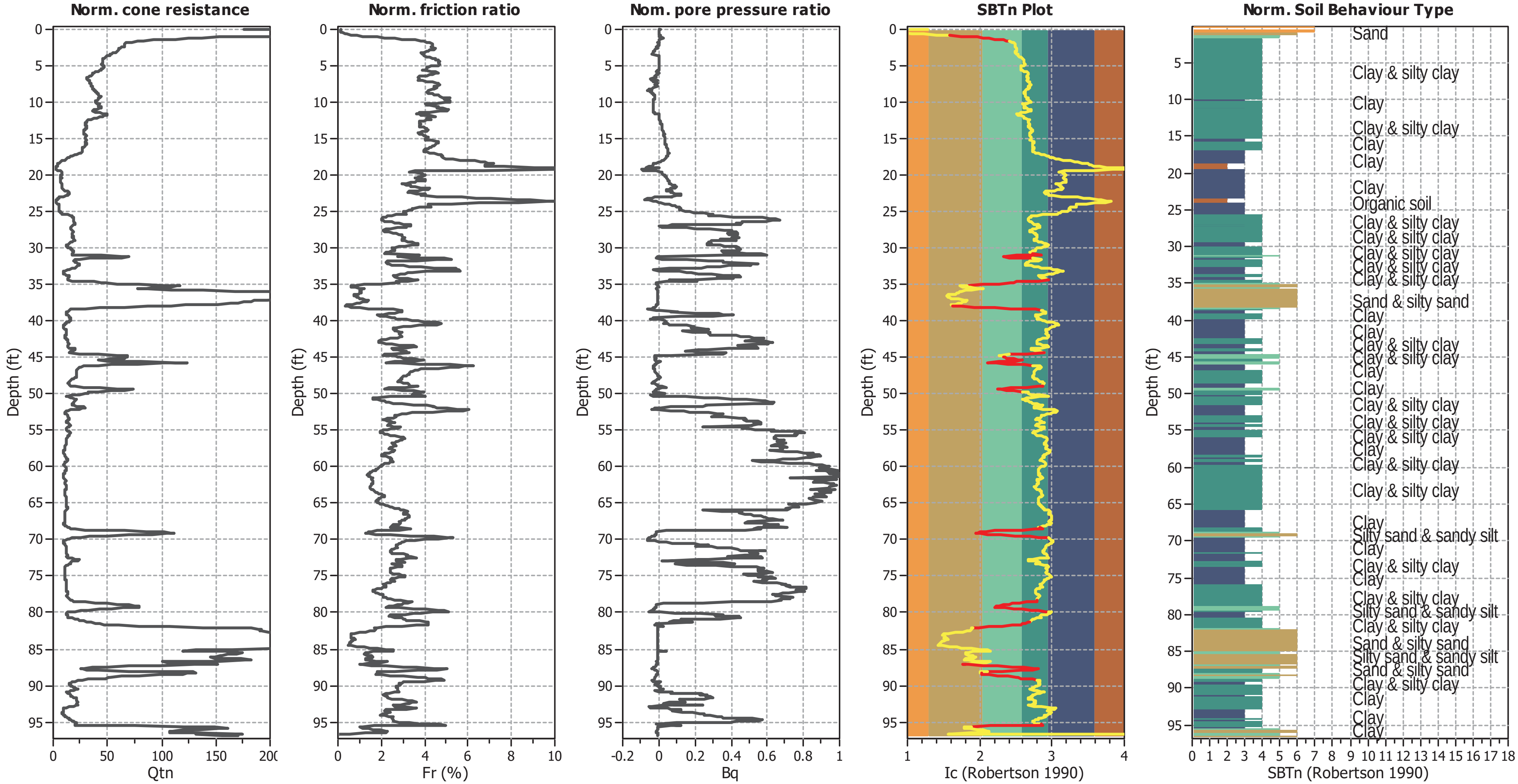
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 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)



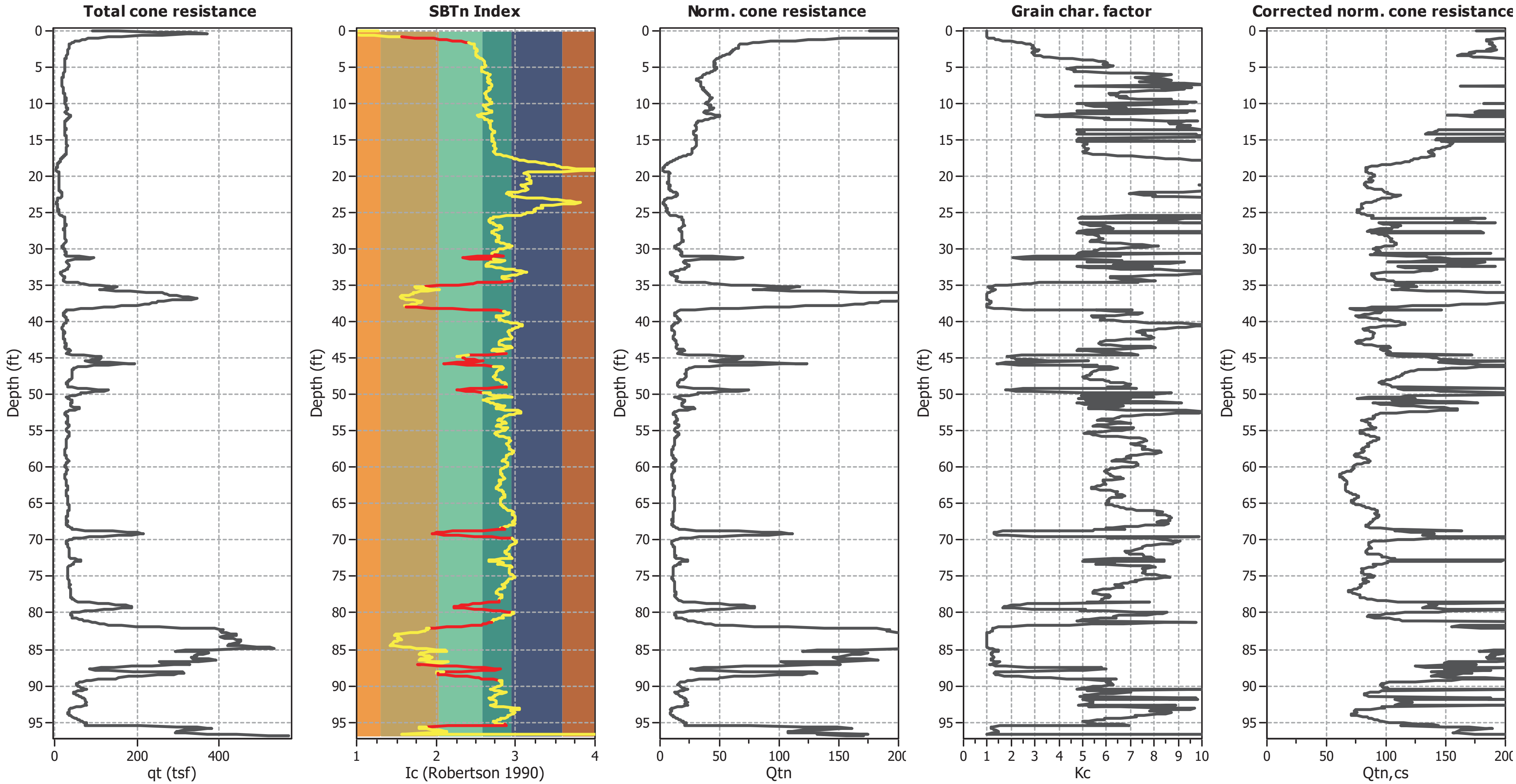
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 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

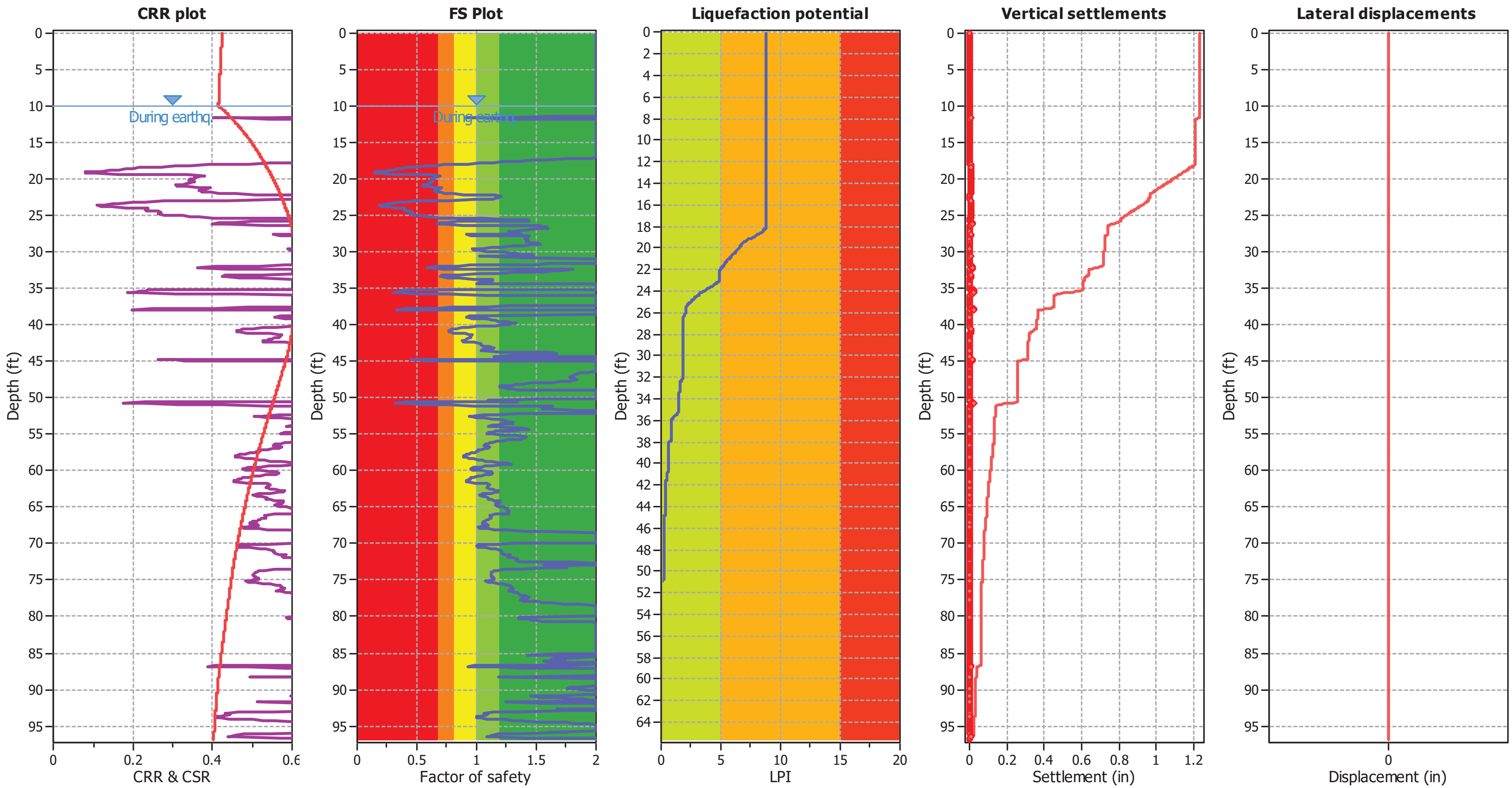
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 ft

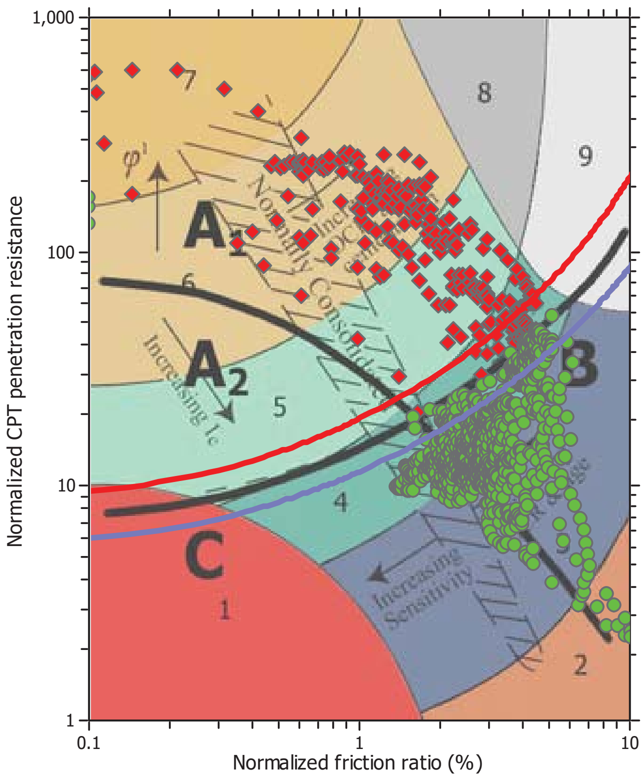
F.S. color scheme

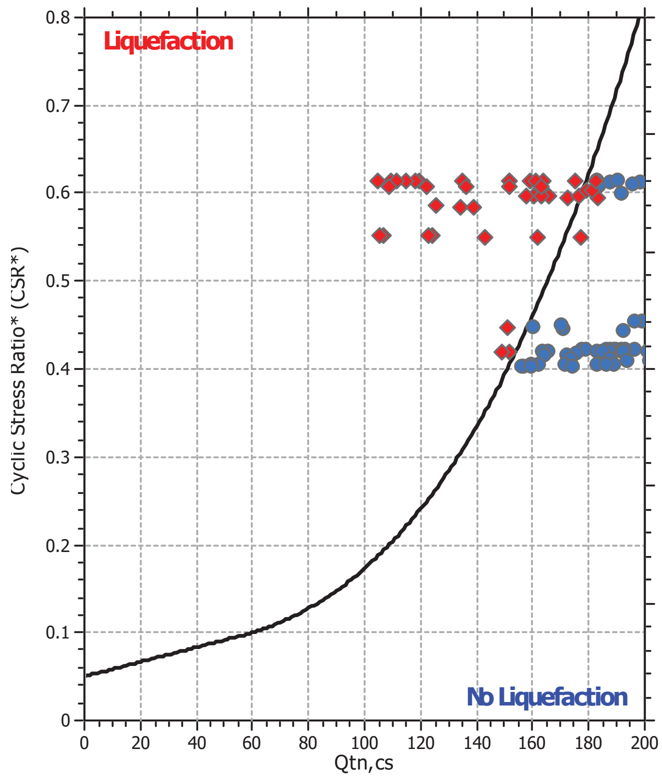
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

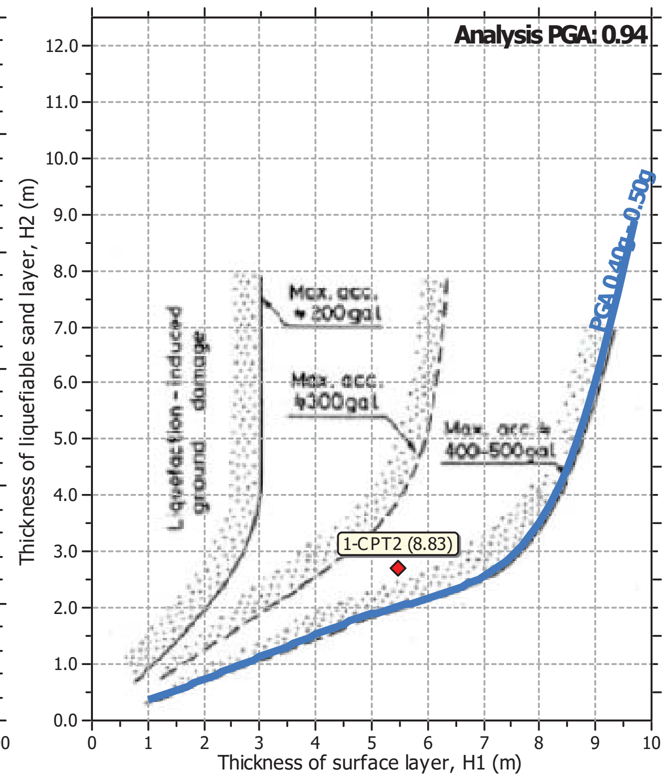
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



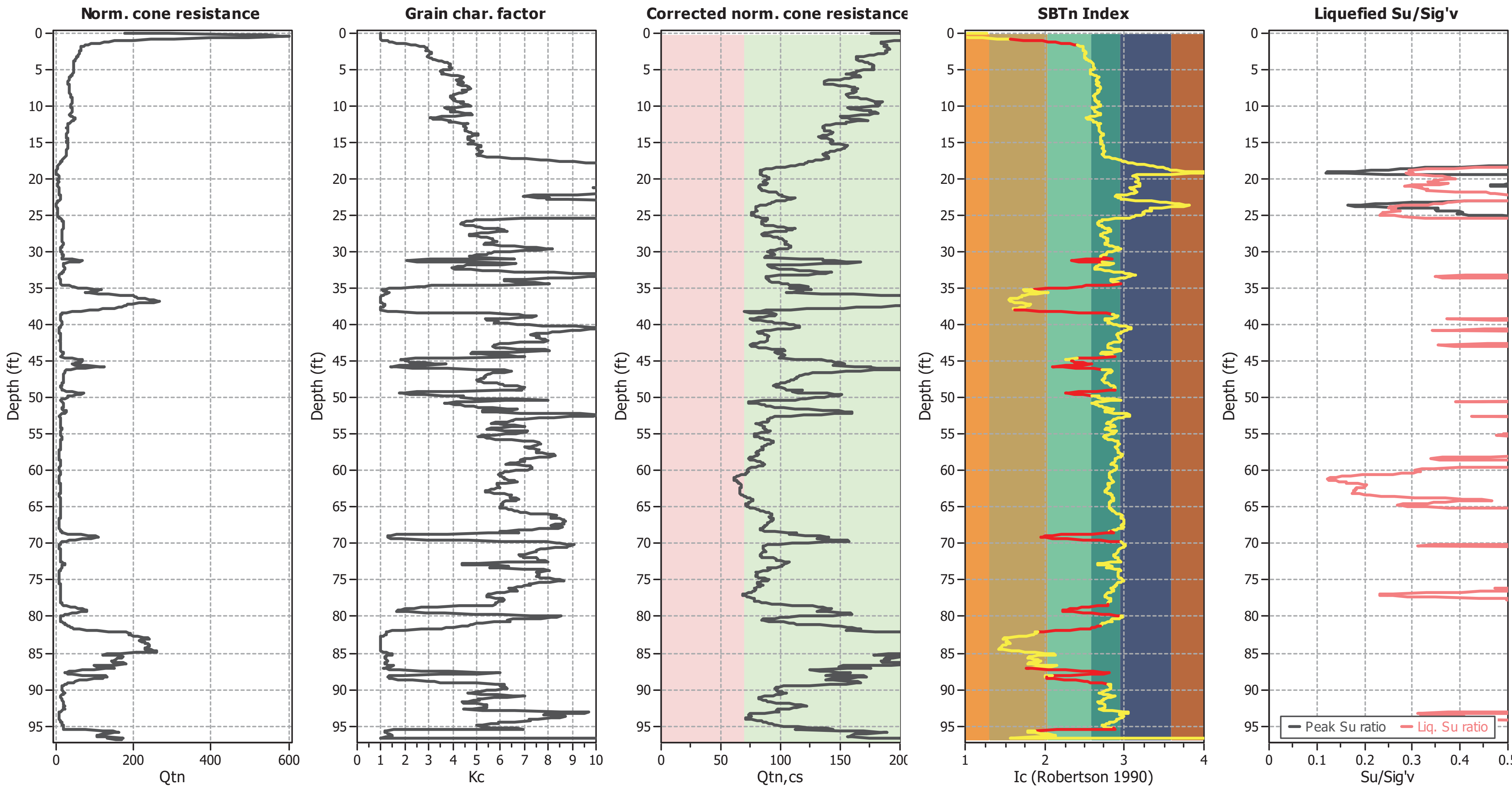




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

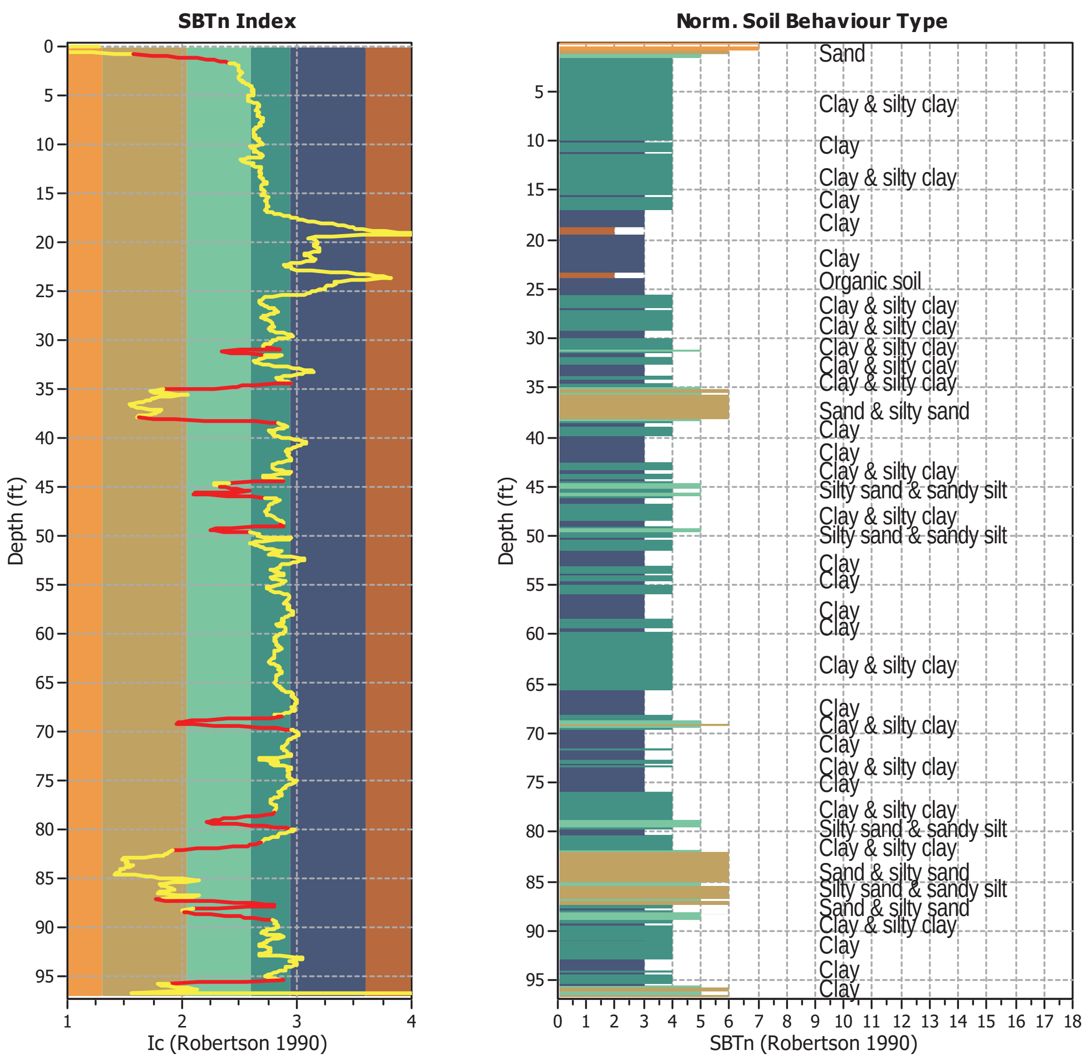
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	18.00 ft	Fill height:	N/A	Limit depth:	100.00 ft

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

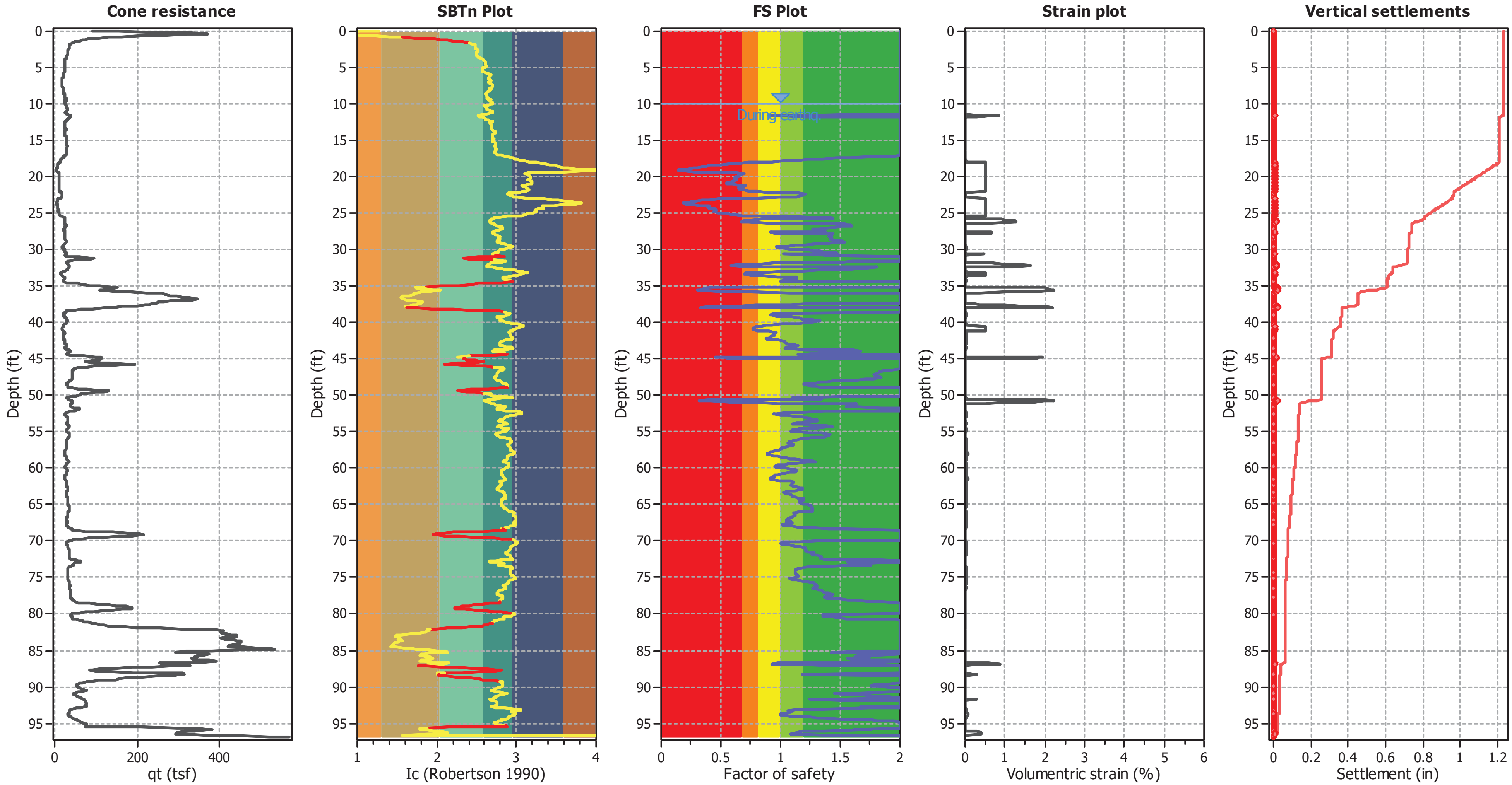
The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	1181
I_c maximum check value:	3.00	Total points excluded:	150
I_c change ratio value:	0.0250	Exclusion percentage:	12.70%
Minimum number of points in layer:	4	Number of layers detected:	20

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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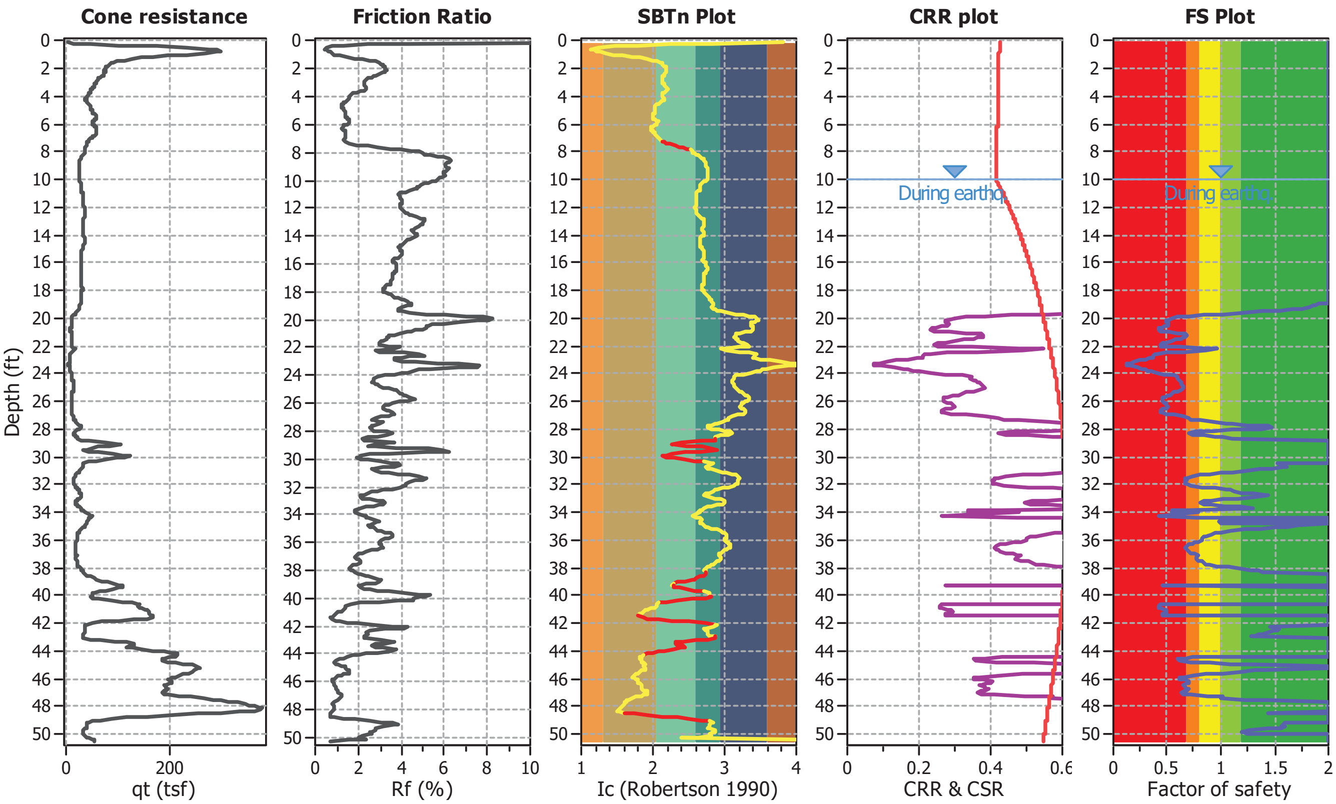
LIQUEFACTION ANALYSIS REPORT

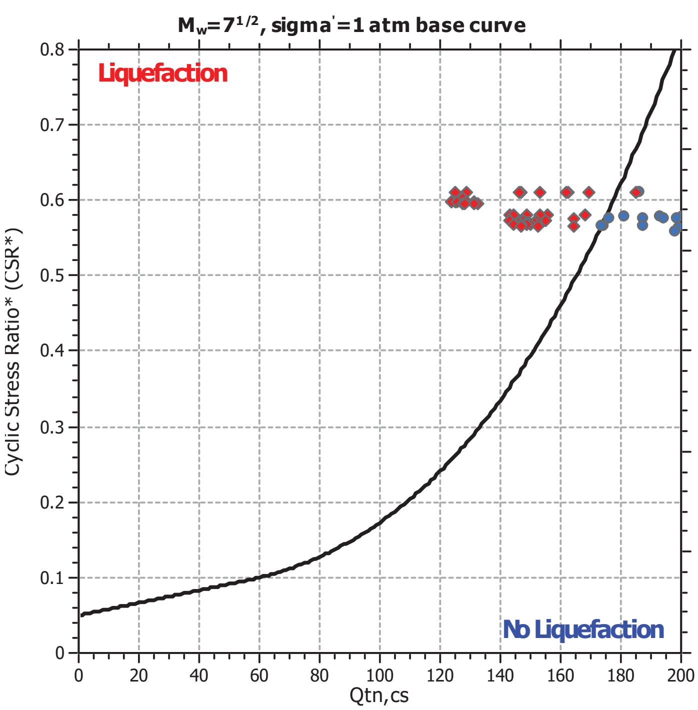
Project title : Pecten Site **Location : Milpitas/San Jose, CA**

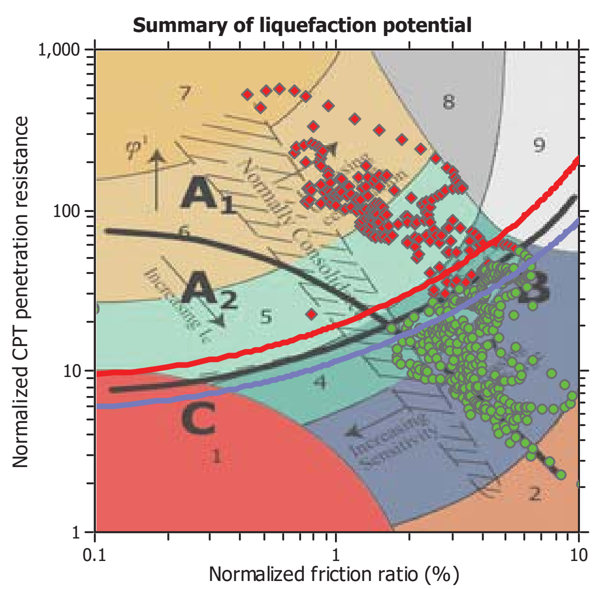
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	19.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.94	Unit weight calculation:	Based on SBT	K_σ applied:	No		

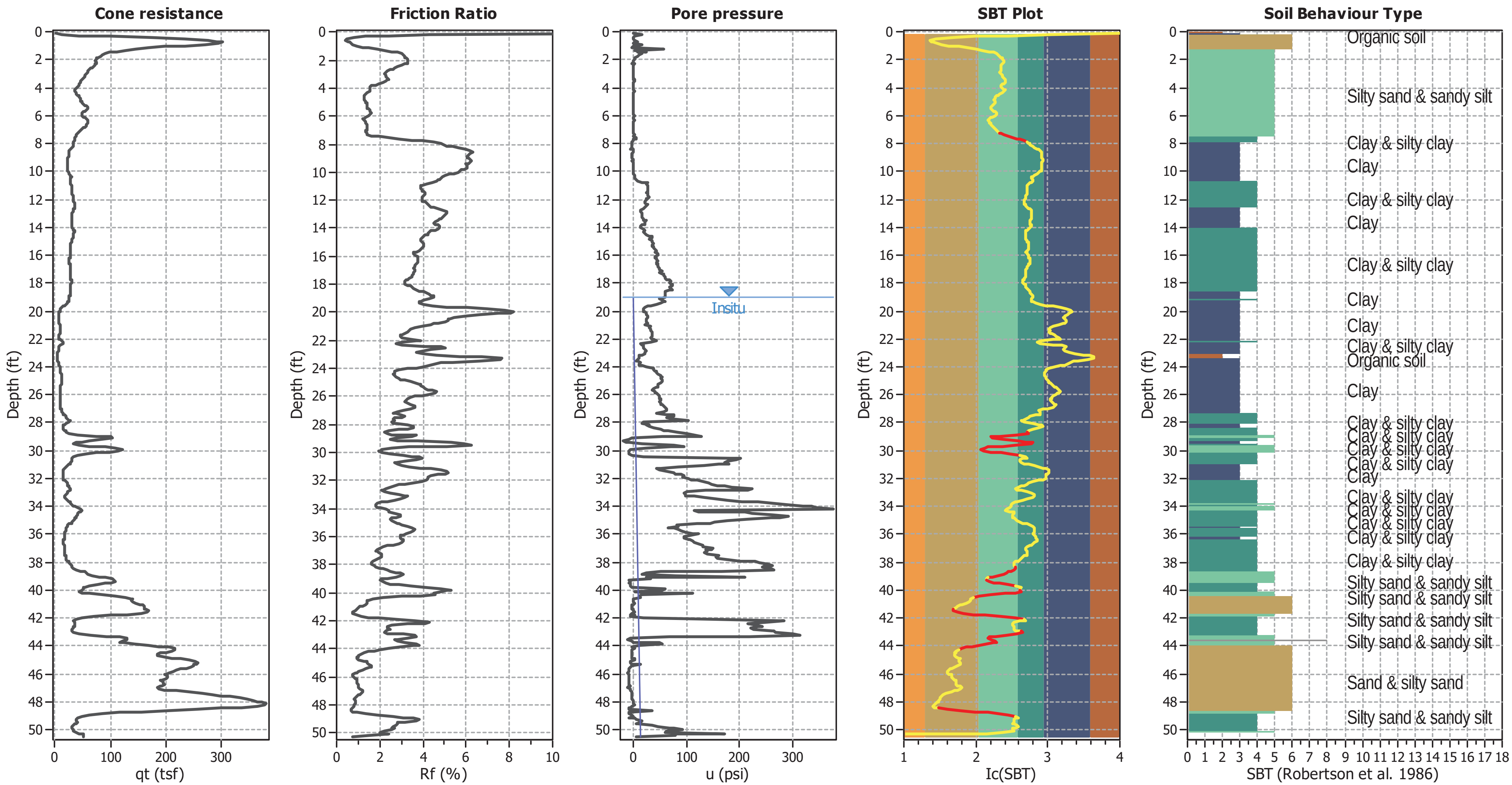






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



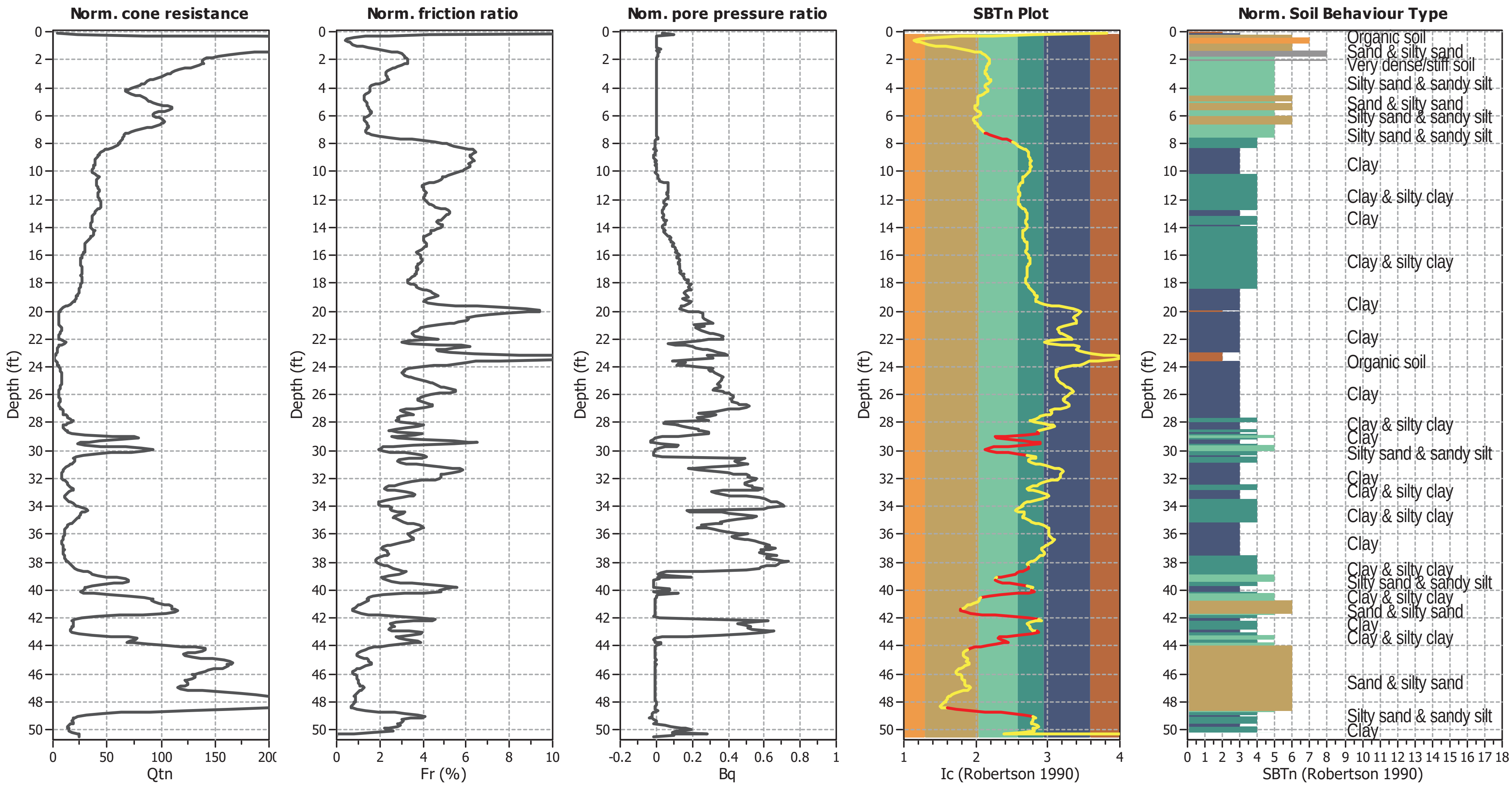
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 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)



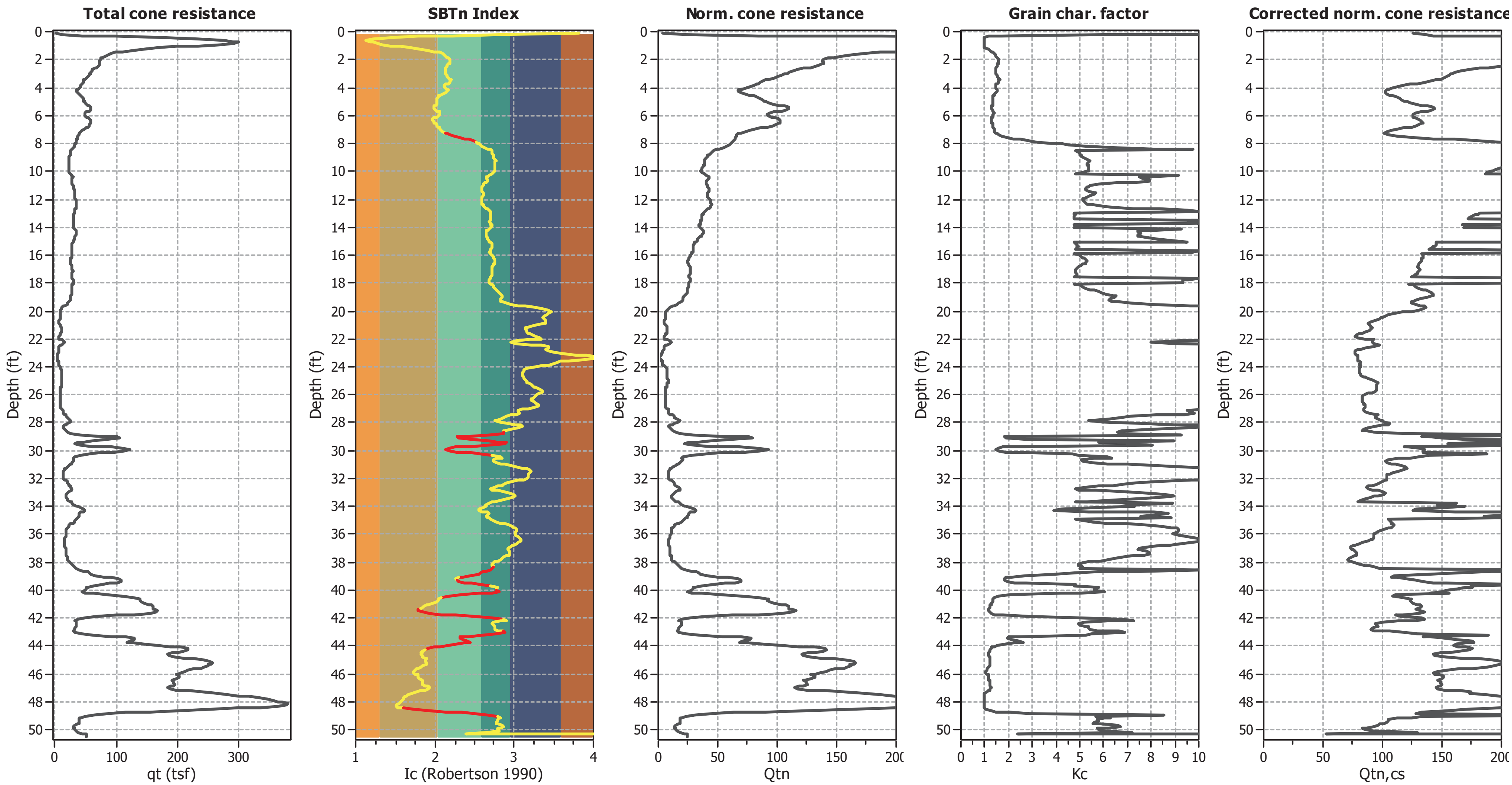
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 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

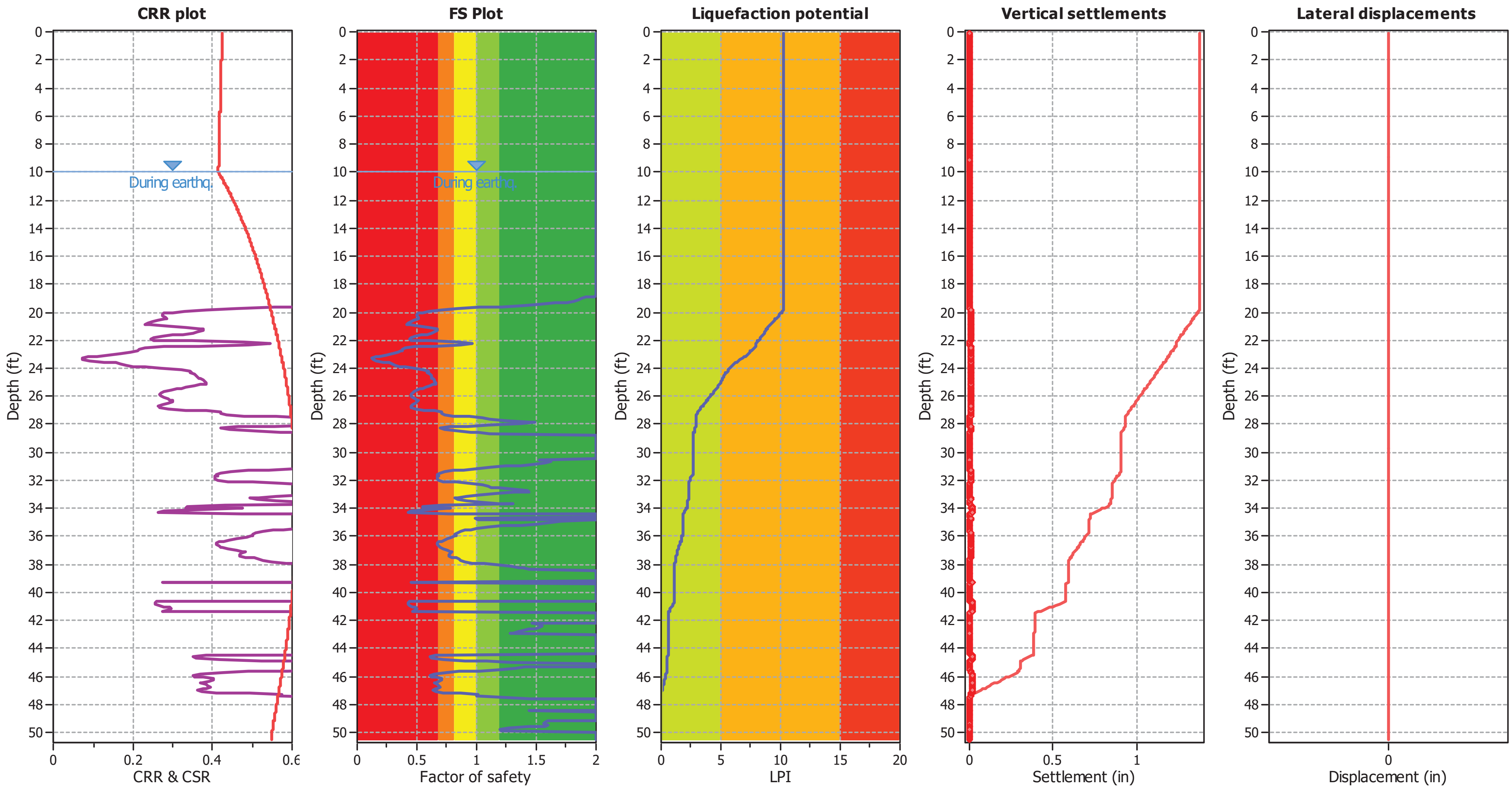
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

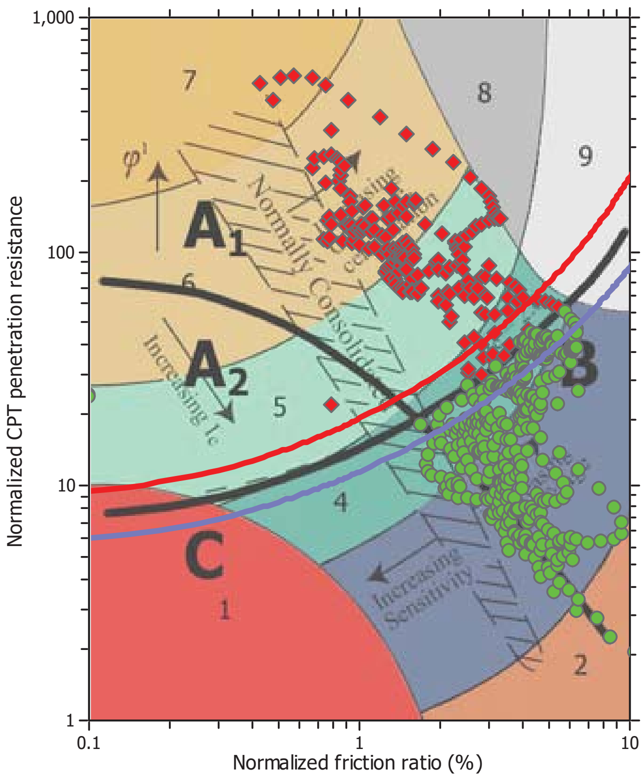
F.S. color scheme

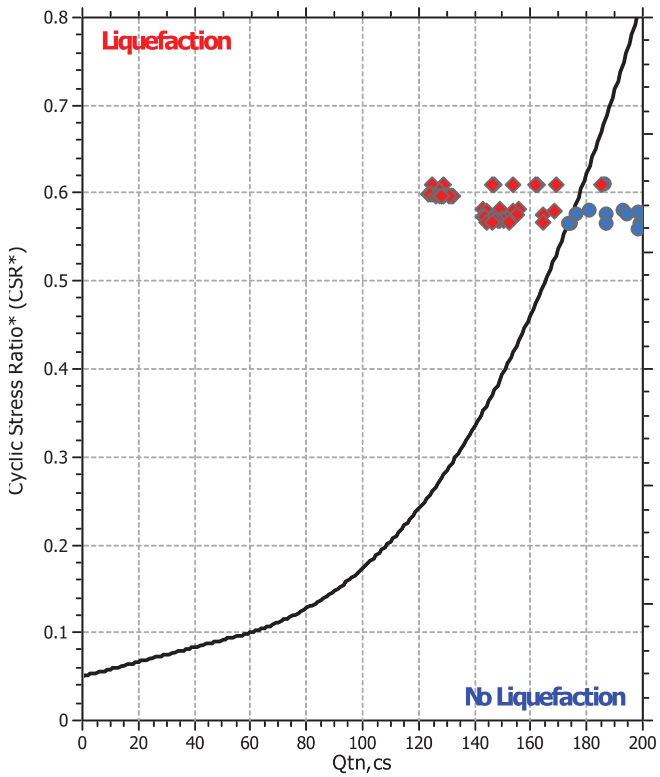
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

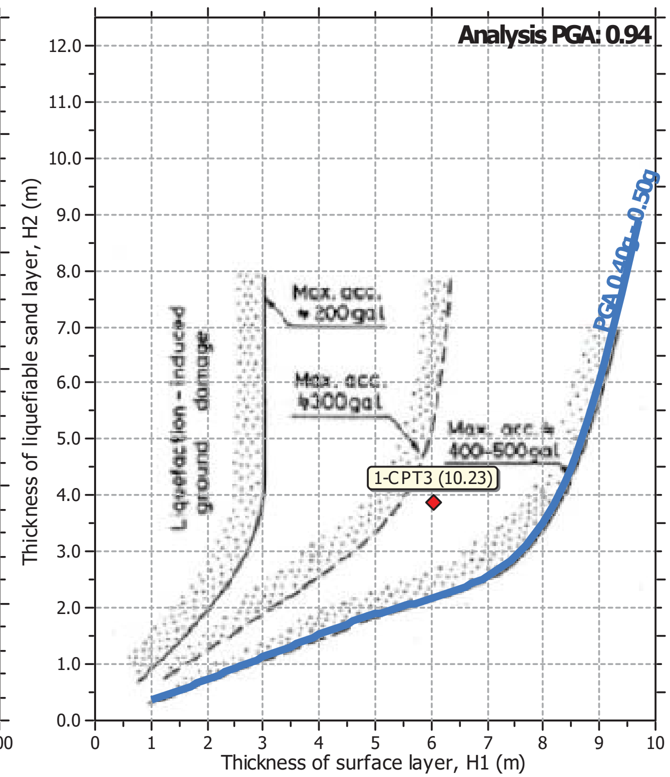
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



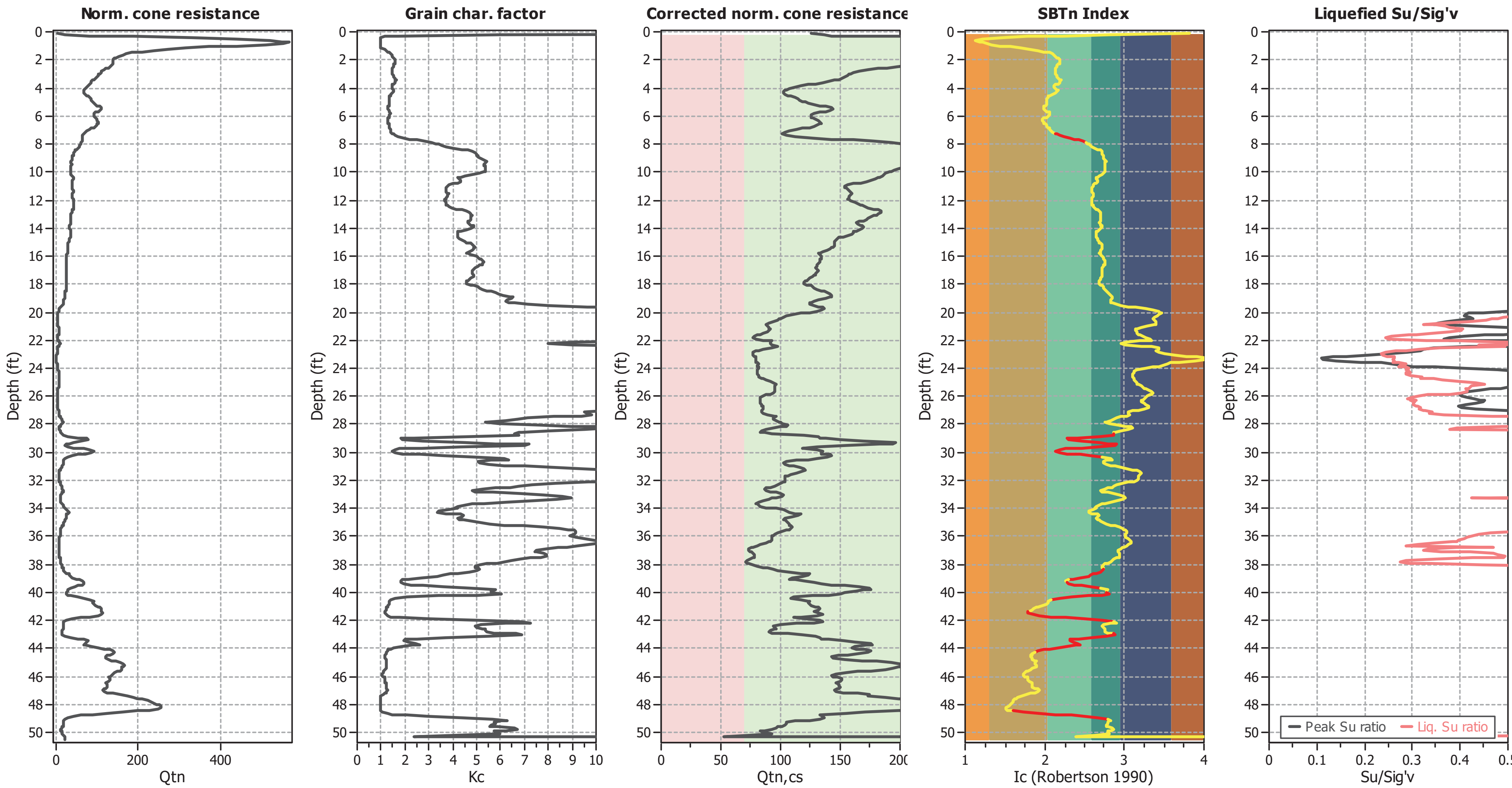




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 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:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

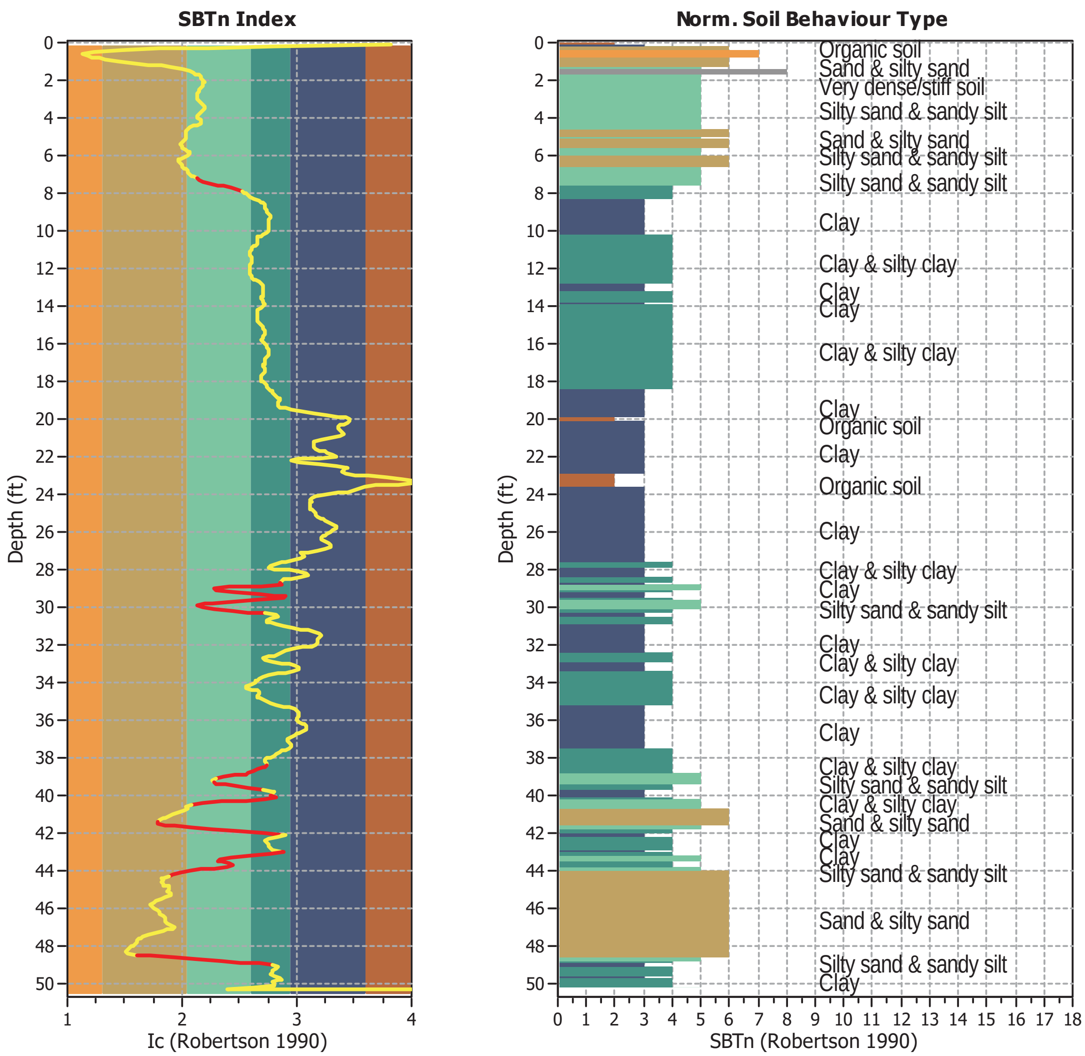
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.94	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

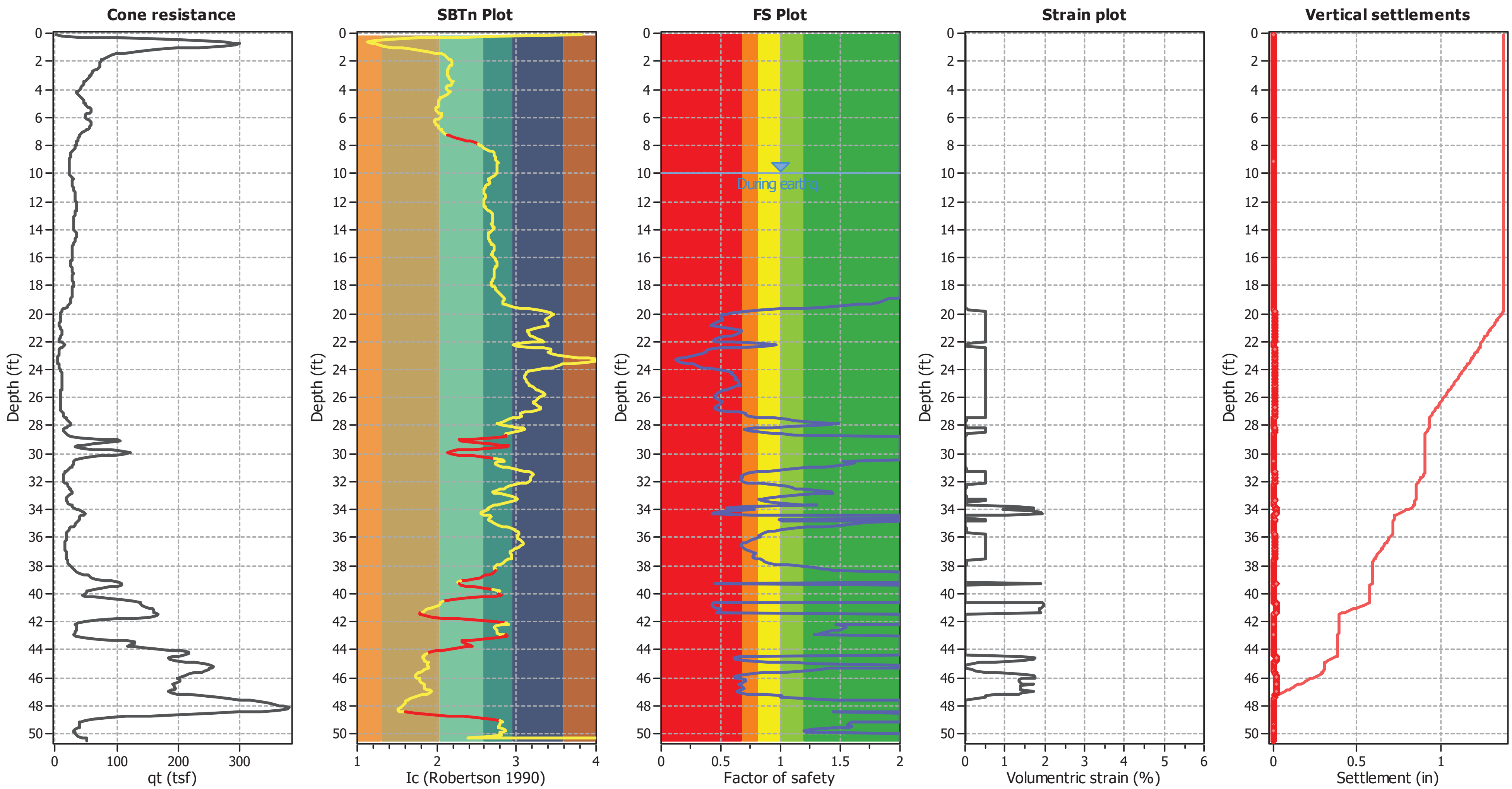
The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	616
I_c maximum check value:	3.00	Total points excluded:	89
I_c change ratio value:	0.0250	Exclusion percentage:	14.45%
Minimum number of points in layer:	4	Number of layers detected:	13

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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