OAKRIDGE GEOSCIENCE, INC.

PRELIMINARY GEOTECHNICAL REPORT
PROPOSED MOORPARK LIBRARY
MOORPARK, CALIFORNIA

Prepared for:
City of Moorpark

June 17, 2017
Job No. 030.003

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June 17, 2017
Project No. 030.003

City of Moorpark
799 Moorpark Avenue
Moorpark, California 93021

Attention: Mr. Chris Ball

Subject: Preliminary Geotechnical Report, Proposed Moorpark Library, Moorpark, California

Dear Mr. Ball:

Oakridge Geoscience, Inc. (OGI) is pleased to provide this preliminary geotechnical report for the proposed library project in Moorpark, California. The purpose of the preliminary geotechnical study was to evaluate if seismic related geohazards including liquefaction, dry seismic settlement and lateral spreading, and hydroconsolidation (collapse) potential are present at the site and the need for ground improvement to mitigate potential settlements that may occur as a result of earthquake-induced ground shaking.

This report summarizes the geotechnical data review, field exploration, geotechnical laboratory testing, our evaluations, and our opinions of the site conditions based on the work performed. A supplemental geotechnical design report will be required as part of project design once the building type and location are selected.

Closure

Thank you for the opportunity to provide geotechnical services to the City of Moorpark for this project. Please contact us if you have any questions on the information presented herein or if we can be of further assistance on this project.

SINCERELY,
OAKRIDGE GEOSCIENCE, INC.

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1.0 INTRODUCTION

1.1 PROPOSED PROJECT AND PURPOSE

The City of Moorpark (City) is planning to build a new library building northwest of High Street and Moorpark Avenue near the location shown on Plate 1. As described in the staff report dated November 30, 2016, the library facility has not been designed but is anticipated to consist of an 18,000-square-foot, one-story building of standard wood frame construction.

A recent geotechnical study for the nearby Area Housing Authority (AHA) site development south of Everett Street (Plate 1) recommended ground improvement to reduce potential foundation settlement associated with liquefaction and dry seismic settlement from earthquake-induced ground shaking due to subsurface conditions at that site (Geotechnologies, Inc., 2016). The City retained Oakridge Geoscience, Inc. (OGI) to perform a preliminary geotechnical evaluation of the proposed library site to evaluate whether the conditions onsite will require subsurface ground improvement similar to the AHA site, prior to hiring an architect or engineer to design the proposed structure.

1.2 WORK PERFORMED AND AUTHORIZATION

The work performed for this study consisted of data review, project coordination, field exploration, laboratory testing, and geotechnical evaluation and reporting. The work was performed in general accordance with our revised proposal dated April 3, 2017 and was authorized by receipt of a Professional Services Agreement from the City, dated April 13, 2017.

1.2.1 Data Review and Project Coordination

We reviewed readily available published data and existing geotechnical reports provided by the City for the nearby AHA site to the east (Geotechnologies, Inc., 2016) and the Moorpark Apartments site (Gorian and Associates, 2013a) to the west. The approximate locations of the AHA and Moorpark Apartments sites are shown on Plate 1. Prior to field exploration, we performed a site reconnaissance to locate and mark the exploration locations for coordination with Underground Service Alert.

1.2.2 Field Exploration

Subsurface geologic conditions at the proposed library site were explored using a combination of cone penetrometer tests (CPTs) and drill holes near the locations shown on Plate 2. The CPT and drill hole logs are included in Appendix A.

CPTs. Five CPTs were advanced to depths of about 75 feet each on April 27 by Kehoe Testing & Engineering. The CPT is mounted on a 30-ton 3-axle truck and consists of an about 1.4-inch-diameter rod fitted with a cone at the base. The cone is sequentially connected to 1-meter-long rods and pushed into the subsurface at a constant rate by hydraulic rams using the weight of the truck as resistance. Additional rods are added to the rod length as the depth increases. The cone is equipped with electronic load cells which measure point (tip) resistance to the penetration and frictional resistance between the soils and the cylinder side (sleeve) of the cone. The subsurface stratigraphy and engineering parameters of the penetrated materials are inferred based on correlations of the recorded tip and sleeve properties. The CPT collects nearly continuous data (2-centimeter intervals) and allows for efficient evaluation of seismic-related hazards, engineering properties, and stratigraphy.

Additionally, the CPT was equipped with a piezo-cone which measures excess pore pressure as a result of the penetration to further aid in evaluation of the depth to groundwater at the site. Pore-pressure dissipation tests were performed in CPT-3 and CPT-5.

Following the completion of each CPT, the rods were withdrawn, and the small-diameter holes were backfilled to the ground surface with fine bentonite chips.

Drill Holes. Two hollow-stem-auger drill holes, DH-1 and DH-2, were advanced near CPT-3 and CPT-4 by S/G Drilling on May 1, 2017 using a CME-85 drill rig equipped with 8-inch-diameter augers and a 140-pound automatic trip hammer. The drill holes were advanced to depths of 50 and 75 feet to help in evaluation of the subsurface conditions, to “ground truth” the CPT data, and to collect samples for laboratory testing and evaluation of liquefaction consistent with the guidelines published by the California Division of Mines and Geology (CDMG, now California Geologic Survey [CGS]), Special Publication 117A (CGS, 2008).

The drill holes were sampled at about 2.5-foot intervals to about 15 feet and at about 5-foot intervals to total depth using a combination of driven modified California and standard penetration test (SPT) samplers. In addition, bulk samples were collected from the near surface materials recovered from the auger flights. Our field geologist logged the recovered samples in general accordance with ASTM D2488 for visual soil classification. Groundwater depths encountered during drilling were measured and recorded on the drilling logs.

Following completion of drilling and sampling at each location, the drill holes were backfilled to the surface with the drill cuttings mixed with cement to create soil-cement and tamped.

1.2.3 Laboratory Testing

Geotechnical laboratory testing was performed on selected earth materials sampled in the drill holes to characterize the materials and estimate relevant preliminary engineering design parameters. The testing consisted of moisture/density relationships, grainsize, Atterberg limits (plasticity), hydroconsolidation (collapse) potential, R-value, and soil chemistry for corrosion (pH, resistivity, sulfates, and chlorides).

The laboratory test results are presented on the drill hole logs (Appendix A) and in Appendix B.

1.2.4 Geotechnical Evaluation and Reporting

We evaluated the field and laboratory geotechnical data, developed preliminary geotechnical engineering recommendations for the project, and prepared this report to summarize our findings, opinions and recommendations. Our report includes the following:

- Summary of soil and groundwater conditions encountered;
- Logs of CPT and drill hole explorations;
- Geologic cross sections depicting interpreted subsurface conditions;
- Laboratory test data;
- Evaluation of seismic-related hazards including fault rupture, liquefaction, dry seismic settlement and lateral spreading;
- Potential need for ground improvement;
- Preliminary design parameters for soil bearing and estimated settlement, and lateral earth pressures;
- Suitability of onsite soil for use as fill and select fill material;
- Anticipated excavation conditions; and
- Preliminary grading recommendations.

2.0 FINDINGS

2.1 BACKGROUND

Geotechnical studies for two nearby sites: 1) AHA site (Geotechnologies, Inc., 2016) and 2) Moorpark Apartments (Gorian and Associates, 2013a) have documented the potential for seismic-related geohazards (liquefaction, dry seismic settlement, lateral spreading) and hydroconsolidation (collapse) potential in the downtown Moorpark area. The approximate locations of the two sites relative to the proposed Moorpark Library site are shown on Plate 1.

AHA Site. At the AHA site, Geotechnologies, Inc. reported zones of medium dense granular soils ranging from less than one-foot to about 18-feet thick between depths of 15 to 75 feet. Their report indicated those soils could liquefy in response to the design earthquake event with settlements ranging from about two- to six-inches. On that basis, Geotechnologies recommended ground improvement to a depth of 30 feet to reduce total settlement to less than two inches and differential settlement to less than one inch. Their report indicated the structure could be supported on shallow spread footings following the recommended ground improvement. Alternatively, if the ground improvement could not reduce the total settlement to less than two inches the structure could be supported on a mat foundation. The report indicated the “most feasible ground improvement techniques could consist of a mixture of soil mixing, stone columns, aggregate piers or earthquake drains.” The final ground improvement design was to be performed by a specialized ground improvement contractor.

As a follow-up to our initial review of the AHA geotechnical report, we spoke briefly with the City’s Geotechnical review consultant, RJR Engineering. Mr. Rob Anderson with RJR

Engineering indicated seismic-related settlement issues have been reported at other locations within the City the Moorpark in addition to the AHA site. Sites closer to the Arroyo Simi drainage channel along the southern portion of the City seem to have a higher amount of estimated seismic settlement. The estimated seismic settlement in other areas in the City is variable.

Moorpark Apartments Site. Gorian and Associates (Gorian, 2006; 2013a; 2013b) prepared a geotechnical study for the Moorpark Apartments site directly west and northwest of the proposed Library Site (Plate 1). Gorian's evaluation of the subsurface conditions indicated the potential for up to nine inches of seismic-related settlement (liquefaction and dry seismic settlement) based on a groundwater level of 15 to 25 feet below the ground surface and an earthquake ground acceleration of 0.68g. Exploration by Gorian was limited to a depth of 50 feet, therefore, subsurface data are not available to evaluate if liquefaction could also occur at deeper depths for that site. We note Gorian (2006) indicates up to 15 inches of dry seismic settlement were estimated from CPT-3A, but the calculated value was not considered accurate and the soils in the upper portion of the CPT would be mitigated as part of site grading. Gorian recommended ground improvement consisting of overexcavation and recompaction of soils to a depth of 13 to 22 feet below the existing grade to mitigate soils susceptible to seismic-related settlement; the proposed mitigation reduced the estimated vertical seismic settlement to about one-and-one-half to four inches. Gorian also recommended the proposed structures be supported on a "strong mat" type foundation to reduce the potential for differential settlement.

2.2 GEOLOGIC SETTING

The project site is located within the Transverse Ranges geologic/geomorphic province of California. That province is characterized by generally east-west-trending mountain ranges composed of sedimentary and volcanic rocks ranging in age from Cretaceous to Recent. Major east-trending folds, reverse faults, and left-lateral strike-slip faults reflect regional north-south compression and are characteristic of the Transverse Ranges. Several authors including Dibblee (1992), and Weber (1973) have mapped the Moorpark area.

The project site is located south of the confluence of two southerly draining tributaries (Walnut Canyon and an unnamed canyon) to the Arroyo Simi. As mapped by Dibblee (1992), the earth materials in the vicinity of the proposed library site consists of alluvial sediments of silt, sand, and gravel deposits.

2.3 REGIONAL GEOLOGIC HAZARDS

Mapping by the CDMG, (now CGS, 2000) indicates the proposed library site is located in a potential liquefaction area based on a regional evaluation of geologic and geotechnical conditions. Proposed habitable developments within this zone are required to have a site-specific liquefaction evaluation performed in accordance with CGS Special Publication 117A (CGS, 2008).

2.4 SITE CONDITIONS

The project site is roughly an "L"-shaped vacant lot located west of the intersection of Moorpark Avenue and West High Street, south of the existing City library and parking lot as indicated on Plate 2. Review of images on Google Earth and the USGS topographic map indicate the project site was formerly developed with small structures that were demolished after about 2003. Asphalt concrete pavement is located in the northwest portion of the "L"-shaped property; the remainder of the site is earthen. The site topography slopes gently to the south. Based on ground surface elevations from the USGS Moorpark Quadrangle, the ground surface at the project site slopes southward from about elevation (El.) +520 feet at the northern portion of the site to about El. +514 feet at the southern portion of the site (6 feet of elevation difference) over a distance of about 270 feet (approximately a 2.2 percent slope).

2.5 EARTH MATERIALS

Descriptions of soil conditions presented herein are based on visual classification of samples obtained from our field exploration combined with the results of laboratory testing.

As depicted on the attached Geologic Cross Sections A-A' and B-B' (Plates 3a and 3b), the earth materials encountered by the CPTs and drill holes for this study consist primarily of interbedded granular alluvial deposits of sand and silty sand to depths of about 40 feet and interbedded silty to clayey sand, sandy clay, and silt from about 40 to 75 feet (maximum depth explored). As shown on the CPT logs in Appendix A, the silt, clay, and sand layers below a depth of 40 feet are typically thinly bedded ranging from several inches to two feet in thickness, with occasional clay or silty sand layers to about five feet thick.

2.5.1 Engineering Properties

A summary of the general engineering parameters for the earth materials encountered in the explorations advanced for this study consists of:

- Field SPT N-values ranged from about 2 to 15 blows per foot (bpf) from the ground surface to a depth of about 25 feet, and 12 to 22 bpf from about 25 to 75 feet below the ground surface (Appendix A). The SPT N-values indicate the granular soils classify as very loose to loose in the upper 25 feet and loose to medium dense from 25 to 75 feet. The fine-grained silt and clay soil layers generally classify as medium stiff, with the exception of a very soft layer at a depth of 50 feet in DH-1.
- Moisture contents generally ranged from about 2 to 8 percent in the granular alluvial deposits above the groundwater level (above 37 feet) and from about 14 to 25 percent below the encountered groundwater level.
- Dry densities of the granular soil in the upper 40 feet of the site ranged from 95 to 111 pounds per cubic foot (pcf), and the densities of interbedded soils from 40 to 75 feet ranged from 112 to 118 pcf.
- The results of grainsize analyses indicate fines contents (percent passing No. 200 sieve) ranging from about 3 to 47 percent for the tested granular soil samples and from about 50 to 63 percent for cohesive materials.
- Atterberg Limit tests indicate the tested fine-grained sandy clay layers have liquid limits of 21 to 26 and plasticity indexes of 6 to 9. Those soils classify as low plasticity sandy clay and sandy to silty clay (Appendix B).
- The hydroconsolidation (collapse) potential for three silty sand soil samples from depths of 10, 25, and 30 feet was tested in accordance with ASTM D4546, Method

B. The test results are presented in Appendix B. The samples were selected for testing based on unit weight, degree of saturation, void ratio, and fines content (percent passing No. 200 sieve). The test results indicate hydroconsolidation potentials of 2.3 percent at 10 feet, 0.05 percent at 25 feet, and 0.4 percent at 30 feet. (Appendix B).

- The near surface soil materials consist of silty sand with an R-value of 70 and an anticipated low expansion index (EI of less than 20).
- The results of the soil chemistry tests are summarized below.

2.6 SOIL CHEMISTRY AND CORROSION

2.6.1 Test Results

A selected soil sample obtained from our exploration was provided to Cooper Testing Laboratories for resistivity, pH, chloride, and sulfate testing. The test results are summarized below and the laboratory test report is included in Appendix B.

Table 1. Summary of Chemical Test Results

Drill Hole	USCS Classification	Depth (feet)	Sulfate (mg/kg/%)	Chloride (mg/kg)	Resistivity (ohm-cm)	pH
DH-1	Sand with Silt	0 - 5	6/0.0006	2	16,319	7.5

2.6.2 Corrosion and Cement Considerations

As summarized in the table above, the measured pH of the tested sample (ASTM G51) is 7.5, the measured electrical resistivity (ASTM G57) is 16,319 ohm-centimeters, the chloride content (ASTM D4327) of the measured samples is 2 mg/kg, and the sulfate content (ASTM D4327) of the measured sample is x6mg/kg (0.0006 percent).

Caltrans (2012) classifies soils as non-corrosive if the earth materials have less than 500 ppm chlorides, less than 0.20 percent sulfates (i.e., 2,000 mg/kg or ppm), a pH of 5.5 or more, or an electrical resistivity of 1,000 ohm-centimeters or more. The data suggest the tested soil materials are not corrosive to underground steel. If applicable, the test results should be evaluated by a corrosion engineer to determine how underground utilities should be protected from corrosion.

The cement type should be selected with consideration of the sulfate content of the tested soils. Available sulfate content data suggest that, per Table 4-3-1 of ACI 318, Type II cement can be used for concrete that will be in contact with onsite granular soils.

2.7 GROUNDWATER CONDITIONS

Groundwater was encountered at depths of about 36 to 37.5 feet in the drill holes advanced onsite (Appendix A). Interpretation of the CPT dissipation test data indicates similar groundwater depths of about 37 feet below ground surface at the time of our exploration on April 27, 2017 (Appendix A). Historically high groundwater levels reported by the CGS (2000) indicate the groundwater levels at the project site have been within about 15 feet of the ground surface. Variations in groundwater levels and soil moisture conditions can occur as a result of rainfall, irrigation, runoff, and other factors.

2.8 DATA INTERPRETATION AND ANALYSES

Data interpretation for this study utilized the CPTs and the SPT N-values from the drill holes advanced onsite (Appendix A). Analyses of the CPT and SPT data from this study were performed using the computer program GeoLogisMiki. Selected computer printouts from the GeoLogisMiki analyses are presented in Appendix C. A complete pdf file of the analyses can be provided upon request.

The field SPT N-values presented on the drill hole logs in Appendix A were normalized to 1 ton/square foot and corrected for rig efficiency, hammer type, sampler type (no liner), and rod length as described in the Recommended Procedures for Implementation of CGS Special Publication 117A (CGS, 2008). Recent modifications to the CGS procedures by Boulanger and Idriss (2014) are incorporated into the software evaluation. We also utilized blow counts measured for the modified California sampler (MCS) in the analyses by dividing the MCS blowcount by 1.6 to provide an equivalent SPT N-value. The SPT N-value correction factors are summarized in Table 2.

Table 2. SPT N-Value Correction Factors

Correction Factor	Value	Comment
Hammer Efficiency (C_E)	1.3	Auto trip hammer 80% efficiency
Rod Length (C_R)	L<15'=0.75 L<20'=0.85 L<35'=0.95 L>35'=1.0	L= Rod Length (feet)
Sampling Method (C_S)	1.2	
Modified California Sampler (MCS) blowcounts	MCS/1.6 = SPT N-value	Equivalent SPT N-Value

2.9 POTENTIAL VARIATION OF SUBSURFACE MATERIALS

There is a potential for variation in the consistency, density, and strength/hardness of the materials from what was encountered in our explorations. The potential exists to encounter perched water, zones of poorly consolidated soils, or other conditions not indicated on the exploration logs. If significant variation in the geologic conditions is observed during construction, we recommend the geotechnical engineer, in conjunction with the project designer, evaluate the impact of those variations on the project design.

2.10 SEISMIC CONSIDERATIONS AND GEOHAZARDS

2.10.1 Faults

The project site is located in a seismically active portion of southern California and the project most likely will be subjected to strong earthquake ground motion during its lifetime. As summarized in the following table, numerous active or potentially active faults are known or postulated to exist within about 15 miles of the proposed new library site.

Table 3. Nearby Faults

Fault	Approximate Distance (miles)[1]	Maximum Moment Magnitude (Mmax)
Simi-Santa Rosa	2.1	6.8
Oak Ridge	6.0	7.1
San Cayetano	8.1	7.1
Northridge	12.1	6.8

[1] Earthquake distances and magnitudes obtained from the USGS website (2017)

2.10.2 Ground Rupture Potential

The site is not located within a State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zone) and no known active or potentially active faults cross or trend toward the site. The potential for fault rupture to affect the site is considered low.

2.10.3 Seismic Considerations for 2016 CBC

We estimated the probabilistic seismic ground acceleration at the proposed library site using the USGS web application (USGS; 2017). On the basis of the web-based analyses, the peak horizontal ground acceleration (pga) at the proposed site is estimated to be 1.035g for an earthquake with a 2,475-year return period (2 percent probability of exceedance in 50 years) assuming Site Class D soil conditions. The following table summarizes the probabilistically estimated strong ground motion parameters for the project site.

Table 4. Summary of USGS Probabilistic Seismic Hazard Deaggregation Results

Return Period (years)	Mean Magnitude (Mw)	Mean Source Distance (miles)	Peak Horizontal Ground Acceleration
2,475	6.9	5.0	1.035g

2.10.4 2016 CBC Seismic Design Parameters

In accordance with Chapter 16, Section 1613 of the 2016 CBC, the following parameters have been obtained from the USGS Seismic Design Maps web application (USGS, 2017) and shall be incorporated into the seismic design at the project site. The subsurface conditions at the site are considered to satisfy the parameters for Site Class D[1]. The associated seismic design parameters for Site Class D for use in generating the risk-targeted maximum considered earthquake and design level spectra are summarized in the following table.

Table 5. 2016 CBC Seismic Design Parameters

2013 California Building Code Section 1613	Seismic Parameter	Site Class D Values
---	Latitude	34.2857
---	Longitude	-118.8829
Figure 1613.3.1(1)	Mapped Acceleration Response Parameter (S_s)	2.755g
Figure 1613.3.1(2)	Mapped Acceleration Response Parameter (S_1)	0.968g
Section 1613.3.2	Site Class	D
Section 1613.3.3 and Table 1613.3.3(1)	Site Coefficient (F_a)	1.0
Section 1613.3.3 and Table 1613.3.3(2)	Site Coefficient (F_v)	1.5
Section 1613.3.3	PGA_M Equation 11.8-1 $PGA_M=F_{PGA}PGA$	1.035g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{MS})	2.755g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{M1})	1.452g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{DS})	1.837g
Section 1613.3.3	Adjusted Acceleration Response Parameter (S_{D1})	0.968g

2.10.5 Liquefaction and Dry Seismic Settlement Potential

Liquefaction is described as the sudden loss of soil strength because of a rapid increase in soil pore water pressures due to cyclic loading during a seismic event. In order for liquefaction to occur, three general geotechnical characteristics must be present [2] : 1) groundwater must be present within the potentially liquefiable zone; 2) the potentially liquefiable soil must meet certain grainsize and classification characteristics; and 3) the potentially liquefiable granular soil must be of low to moderate relative density. If those criteria

[1] A Site Class D soil is defined in California Building Code (CBC) as the soil having the following average parameters for the upper 100 feet of the site: 1) shear wave velocity of 600 to 1,200 ft/sec, 2) standard penetration test SPT N-value of between 15 to 50, and 3) undrained shear strength of fine-grained soil between 1,000 to 2,000 psf. SPT N-values in the upper 50 feet of the Moorpark Library site ranged from 2 to 15 for granular soils to a depth of about 25 feet and 12 to 22 from about 25 to 75 feet (Appendix A). The average SPT N-values and soil shear strength in the upper 100 feet of the site would be consistent with Site Class D soil.

[2] Based on studies by Seed and Idriss (1971) and Youd and Idriss (2000), liquefaction occurs primarily in clean granular soils that classify as sand (SP) and sand with silt (SP-SM). Dense granular soils with fines contents greater than 35% (silty sand - SM and clayey sand - SC) are less likely to liquefy. Liquefaction susceptibility criteria developed by Boulanger and Idriss (2006) indicates that fine-grained soils with a PI of 6 or less can be susceptible to liquefaction. Studies by Bray and Sancio (2006) indicates that silty soils with a PI of 12 or less could potentially liquefy.

are met and strong ground motion occurs, then those soils may liquefy, depending upon the intensity and cyclic nature of the strong ground motion. Liquefaction that produces surface effects generally occurs in the upper 40 to 50 feet of the soil column, although the phenomenon is not restricted to depths of less than 50 feet.

As described in the Earth Materials section above, the soil profile consists primarily of interbedded granular alluvial deposits of sand and silty sand to depths of about 40 feet and interbedded silty to clayey sand, sandy clay, and silt from about 40 to 75 feet (Plates 3a and 3b). Groundwater was encountered at a depth of about 37 feet during field exploration for this study. Historic high groundwater levels summarized by the CGS (2000) are about 15 feet below the ground surface. SPT N-values from the upper 25 feet of the drill holes range from 2 to 15 bpf, indicating the granular soils are very loose to medium dense in that zone. The SPT N-values from 25 to 75 feet range from 12 to 22 bpf, indicating the granular soils are medium dense and the fine-grained silt and clay soils are medium stiff within that zone.

Research by Boulanger and Idriss[2] (2006) has indicated fine-grained silt and clay soils with Plasticity Index (PI) values of 6 or less can be susceptible to liquefaction and research by Bray and Sancio (2006) indicates low plasticity silt with a PI of up 12 can liquefy during strong earthquake ground shaking. Clay soils with PI of greater than 18 generally exhibit a clay-like behavior and are considered non-liquefiable based on the criteria developed by Bray and Sancio (2006). The fine-grained sandy clay and sandy to silty clay soil layers tested for this study (Appendix B) have fines contents (percent passing the number 200 sieve) of 50 to 63 percent and PI's of 6 to 9, suggesting those layers have low plasticity and may be susceptible to liquefaction in response to strong earthquake ground shaking.

Analyses of the CPT and SPT data were performed using the program GeoLogisMiki. The input values are summarized below and selected graphics from the analyses are presented in Appendix C:

- The seismic ground motion is 1.03g for a 2 percent probability of exceedance in 50 years for the project site.
- Historic high groundwater level of 15 feet below the ground surface.
- CPT evaluation using the procedure recommended by Robertson (2009).
- SPT data evaluation using the procedure recommended by Boulanger and Idriss (2014).

Overall, the liquefaction analyses indicate the very loose to loose granular soils at the site are susceptible to liquefaction below the groundwater and dry seismic settlement above the groundwater. The estimated vertical liquefaction and dry seismic settlements are summarized in Table 6.

Seismically induced settlement or collapse can occur in soils that are loose, soft, or that are moderately dense, but weakly cemented. The onsite very loose to loose granular and silty soils above the groundwater are susceptible to seismically induced settlement. The estimated seismically induced settlement in the upper 15 feet of site is summarized in Table 6. We note the groundwater is assumed to be at 15 feet; therefore, soils below that depth are subject to liquefaction potential in the analyses even though the groundwater depth encountered by our explorations was about 37 feet below the ground surface.

Table 6. Summary of Estimated Vertical Seismic Settlement

Exploration Location	Estimated Liquefaction Settlement (inches)	Estimated Dry Seismic Settlement (inches)	Total Estimated Seismic Settlement (inches)	Estimated Lateral Displacement (inches)
CPT-1	7.5	6.9	14.4	200 inches
CPT-2	8.0	8.3	16.3	200+ inches
CPT-3	9.9	8.2	18.1	200+ inches
CPT-4	10.9	5.8	16.7	300+ inches
CPT-5	10.4	8.0	18.4	300+ inches
DH-1	13.8	34.0	37.8	108 inches
DH-2	9.4	2.4	11.8	72 inches
Range (inches)	**7.5 -13.8**	**2.4 - 34**	**11.8 – 37.8**	**-**
Average Value (inches)	**10**	**9.4**	**19**	**-**

2.10.6 Data Summary

Review of the data plots in Appendix C indicates:

- The liquefaction and dry seismic settlements estimated from the five CPTs advanced for this study are fairly consistent, ranging from 7 to 11 inches and 6 to 8 inches, respectively.
- The estimated liquefaction and dry seismic settlement estimated from the SPT data ranges 9.4 to 13.8 inches and 2.4 to 34 inches, respectively. The estimated liquefaction settlements from the SPT data are fairly consistent with CPT data with a slightly higher value for estimated settlement in DH-1 which extended to 75 feet (25 feet deeper than DH-2).
- The procedures for estimating dry seismic settlement from blowcount data are sensitive to low N-values such as was encountered in the near surface soil in DH-1. In DH-1, a three-foot-thick zone from 3.5 to 6.5 feet with an SPT N-value of 2 accounts for half (17 inches) of the estimated dry seismic settlement in that drill hole.
- The analyses presented in Appendix C indicate the loose granular soils and soft low plasticity silt/clay layers have a seismic factor of safety of less than 1 and an associated liquefaction potential to a depth of 75 feet (maximum depth explored).
- A majority of the estimated settlement from the CPT data occurs between the ground surface and a depth of about 40 to 50 feet.
- Estimated liquefaction settlement below a depth of about 40 feet is about 2 to 4 inches based on the CPT data (Appendix C).
- The total estimated liquefaction settlement in DH-1 (75 feet deep) is 13.8 inches; 4 inches of the settlement is estimated below about 50 feet. The analyses for DH-1

conservatively assumes all zones below a depth of 15 feet could liquefy except for a medium stiff clay from 66 to 69 feet.

2.10.7 Lateral Movement

The occurrence of lateral spreading is generally associated with sites where liquefaction is possible and: 1) the ground surface is sloping, or 2) there is a free-face condition such as a road cut or riverbank. Existing analytical methods of assessing potential deformations caused by lateral spreading are based on a small number of case histories and generally involve layers of liquefiable soils of greater than about three feet (one meter). The procedures are generally considered reasonable in assessing risks where significant lateral deformations are possible (deformations of three feet or greater). The ability to reasonably predict small lateral spreading deformations is, however, considered significantly limited.

As depicted on the regional geologic/topographic map for the Moorpark Quadrangle (Dibblee, 1992), the ground surface in the vicinity of the project site slopes southward at a gradient of about 2.2 percent or less (six feet over 270 feet). From High Street southward, the regional slope gradient is one percent or less to the west. As described above, based on the CPT and drill holes advanced for this study, there is a potential for liquefaction, primarily in the upper 40 to 50 feet of the site. The lateral displacements estimated from the CPT and SPT data are summarized in Table 6 and range from 72 inches to greater than 300 inches.

CGS Special Publication 117A (CGS, 2008) defines large-scale ground displacements as areas that exceed one to three feet horizontally and four to six inches vertically. The estimated lateral displacements summarized in Table 6 range from six to 25 feet, and estimated vertical settlements (combined liquefaction and dry seismic settlement) in Table 6 average 19 inches. Based on both of those criteria, ground improvement of the subsurface soils will be required prior to construction to reduce the estimated lateral displacement to acceptable levels.

2.11 HYDROCONSOLIDATION (COLLAPSE) POTENTIAL

Research by several authors including and Houston et al. (1997; 2001) and Purdue University (Howayek, 2012) indicates hydroconsolidation (collapse) typically occurs in silty and granular soil materials with densities below 105 pcf, degree of saturation of less than 25 percent, and high void ratios. In the Ventura County area, our experience indicates hydroconsolidation is commonly associated with silty soils deposited in debris-flow type environments. The depositional environment with high collapse potential previously observed in Ventura, Camarillo, and Simi Valley consists of Holocene- to Late Pleistocene-age alluvial fan deposits above the groundwater. As noted above in the Site Conditions section of this report, the proposed site is located at the mouth of tributary drainage to Arroyo Simi and is underlain by younger to older alluvial deposits; those deposits are equivalent to the Holocene- to Late Pleistocene-age fan deposits.

Based on an evaluation of the laboratory index properties (soil density, moisture content, void ratio, and fines content), three samples were selected for collapse testing per ASTM D4546, Method B. The results of those tests are presented in Appendix B and are summarized in Table 7 below. Based on published criteria (ASTM D5333), a collapse index of two percent or less is classified as slight, two to six percent is moderate, six to ten percent is moderately severe, and above 10 percent is severe. Based on the tested samples, the amount of hydroconsolidation ranges from 0.05 to 2.3 percent. The values of less than two percent are considered slight by ASTM D5333 classification and within background levels for soils in Ventura County based on our previous experience. The sample from DH-2 at 10 feet with 2.3 percent hydroconsolidation (collapse index) indicates a moderate degree of potential collapse settlement.

The typical procedure to mitigate shallow collapse potential is to overexcavate and recompact the soil. If ground improvement is performed at the site, the near-surface soils would be densified and, in our opinion, likely reduce the hydroconsolidation potential to an acceptable level (i.e., less than two percent).

Table 7. Summary of Hydroconsolidation (Collapse) Potential of Onsite Soils

Location and Depth	Soil Type	Dry Density (pcf)	Moisture Content (%)	Degree of Saturation	Void Ratio (%)	Fines Content (%)	Measured Hydroconsolidation (%)
DH-2 10 feet	Silty Sand (SM)	96.9	3.5	13	0.71	22	2.3
DH-2 25 feet	Silty Sand (SM)	89.9	5.6	18	0.84	29	0.05
DH-1 30 feet	Sand w/Silt (SP-SM)	102	2.5	11	0.62	7	0.43

2.12 EXPANSIVE SOILS

As described on the drill holes and laboratory data, the onsite surficial soils consist of sand and silty to clayey sand. The onsite granular soils are anticipated to have a low expansion potential.

3.0 OPINIONS AND RECOMMENDATIONS

3.1 SUMMARY OF SUBSURFACE SITE CONDITIONS

The geotechnical conditions for the proposed library site were evaluated based on the explorations advanced for this study supplemented by data from previous geotechnical reports from the project vicinity. Based on the work performed, the site conditions consist of:

- Generally granular sand and silty sand soil in the upper 40 feet underlain by thinly interbedded silt, clay, and clayey sand from 40 to 75 feet (maximum depth explored).
- SPT N-values from the upper 25 feet of the drill holes range from 2 to 15 bpf, indicating the granular soils are very loose to medium dense in that zone. The SPT N-values from 25 to 75 feet range from 12 to 22 bpf, indicating the granular soils are medium dense and the fine-grained silt and clay soils are medium stiff within that zone.
- Groundwater was encountered at a depth of about 37 feet during exploration. Historic high groundwater levels in the Moorpark area are about 15 feet below the ground surface.
- The site is not located within a fault rupture hazard zone as defined by the State of California, California Geological Survey.
- The site is located in a seismically active area of Ventura County and has an estimated peak ground acceleration PGA_M of 1.03g.
- The plasticity index of fine grained soils ranges from 6 to 9. Research by Bray and Sancio (2006) indicates the fine grained soils could potentially liquefy during a seismic event.
- CPT and SPT data were evaluated (Appendix C) to estimate liquefaction and dry seismic settlement using the program GeoLogisMiki and the procedures developed by Robertson (2009) and Boulanger and Idriss (2014). The combined estimated liquefaction and dry seismic settlement ranges from about 12 to 34 inches with an average of about 19 inches in the upper 75 feet at the site.
- A majority of the estimated seismically induced settlement occurs in the granular soil layers in the upper 50 feet of the site; less than two to four inches of settlement is estimated to occur below 50 feet. Based on the liquefaction analyses, the fine-grained silt and clay soil layers do not contribute to liquefaction settlement.
- Estimated lateral spreading ranges from six feet to greater than 20 feet using the procedure developed by Robertson (2009) for CPT data and Boulanger and Idriss (2014) for SPT data.
- Estimated hydroconsolidation (collapse) potential ranges from 0.05 to 2.3 percent based on the laboratory testing on three samples of onsite soil.
- Nearby sites have estimated liquefaction/dry seismic settlement 2.5 inches (AHA Site; Geotechnologies, 2016) and 2 to 9 inches (Moorpark Apartments; Gorian, 2013). Liquefaction potential was identified to depths of about 60 feet with individual zones ranging from several feet to 18 feet thick.

- CGS Special Publication 117A (CGS, 2008) and the California Building Code (CBC) typically require projects to have seismic settlement of no more than two inches total and one inch of differential settlement. Sites with estimated settlements of more than two inches are normally required to mitigate settlement to about two inches with ground improvement. Potential ground improvement options are discussed in the following sections.

3.2 GROUND IMPROVEMENT OPTIONS

As discussed above, ground improvement of the soils at the proposed library site will be required to mitigate the amount of estimated settlement to near two inches of total settlement and one inch of differential settlement. To reduce the estimated settlement to near two inches will require improving the site to a depth of approximately 50 feet. We note a 50-foot thick treatment depth would reduce the estimated settlement to less than two inches for most of the exploration locations performed for this study with the exception of DH-1. The data and analyses for DH-1 indicates up to four inches of settlement could occur from depths of 50 to 75 feet. However, in our opinion, if the upper 50 feet of soil were densified/improved, the site would have a 50-foot-thick cap of non-liquefiable improved soil to dampen any settlement below 50 feet. If the treatment depth was limited to 50 feet, a mat-type foundation may be required to reduce differential settlement to an acceptable level for the structure. The alternative would be to select a ground improvement option that could treat soil to a depth of greater than 50 feet as discussed below.

The two primary ground improvement methods to mitigate seismically induced settlements to a depth of about 50 feet with groundwater at a depth of 37 feet are: 1) vibro replacement (VR, also referred to as “stone columns”), and 2) deep soil mixing (DSM). The VR procedure consists of advancing a 30-inch diameter steel mandrel to the selected depth (approximately 50 feet) using a combination of the weight of mandrel and vibration. Once the mandrel reaches the selected depth, ¾-inch crushed rock is used to backfill the hole. The gravel is vibrated and “rammed” into the soft soil. The stone columns are placed on a grid pattern with a spacing typically in the range of six to nine feet on center. The soil displaced by the mandrel is “pushed” laterally into the adjacent soil, densifying the soil mass at the site to the point where it will resist liquefying and settlement in response to earthquake ground shaking. CPTs are advanced between columns after the VR is performed to evaluate the increase in soil strength/resistance to liquefaction. VR is an effective method of densifying granular soils to a depth of about 50 feet, but the process does not significantly improve the density of fine-grained silt and clay soils or highly interbedded fine-grained and granular soils. In our opinion, VR will be most effective in the upper 40 feet at the proposed library site.

DSM uses a large diameter auger mounted to a large drill rig or crane to advance the auger to the target depth (approximately 50 feet for the library project). Cement is mixed into the soil at a regulated rate of around 10 percent and mixed by the auger using several up and down passes of the auger. The amount of cement added to the soil is determined by laboratory testing to optimize the soil strength versus amount of cement utilized. Once the cement and soil are uniformly mixed, the auger is withdrawn and moved to the next location. The DSM columns can be placed in a variety of patterns (grid, tangent, overlapping) depending on the project requirements. For the proposed library project, one option is to place the DSM columns on a grid pattern with a center to center spacing of two to three diameters with a grade beam type foundation system supported on the columns. The column configuration will depend on the column diameter selected (typically three to six feet), cement percentage, soil type, and amount of soil improvement required. Once the columns are completed, a grade-beam type foundation can be installed on top of the DSM columns to support the structure. Other column configurations such as tangent columns, overlapping columns, etc. can be utilized depending on project requirements. The advantages of the DSM method are that it can installed to depths of greater than 50 feet and it can improve the strength of fine-grained soils.

The final design of the ground improvement system is typically performed by the specialty ground improvement contractor working with the project civil, structural, and geotechnical engineers. Other options could be considered pending an evaluation by a specialty ground improvement contractor. Both methods are established procedures and are considered feasible for the Moorpark Library site pending detailed site analyses of the proposed method and cost proposal from a qualified ground-improvement contractor. The pros and cons of the two primary methods are summarized in the following table.

Table 8. Summary of Ground Improvement Methods

Ground Improvement Method	Pros	Cons	General Cost Range
Vibro Replacement (VR) / Stone Columns	• Established procedure, excepted by agencies • Densifies granular soil between individual columns • Provides conduit to dissipate buildup of water pressure during a seismic event • multiple contractors perform procedure – multiple bids • No spoil generated during installation	• Treatment depth limited to 50 feet • Vibration could impact adjacent structures. Vibration monitoring recommended. • Limited density improvement to fine-grained silt and clay soils from 40 to 50 feet below the ground surface. • Treatment area usually extends out beyond building foundations • Ground disturbance at surface requires upper several feet of site to be recompacted	• Mob/Demob - $60,000 • $30/ft of column • Column center to center spacing typically 6 to 9 feet
Deep Soil Mixing (DSM)	• Established procedure excepted by agencies • DSM columns can be extended to depths of 75 feet if required. • Treatment area can be limited to building foundation footprint depending on site conditions	• More expensive mobilization and per foot of column cost than vibro replacement • Does not densify soil between columns • Soil between columns can settle requiring a grade-beam type foundation to span across columns • About 20 percent spoil generated during installation that needs to be disposed of.	• Mob/Demob - $100,000 to $150,000 • $50/ft of column • Replacement ratio 10%

3.2.1 Ground Improvement Limits

Typically, ground improvement is performed beneath the building footprint for "habitable structures" plus a minimal distance outside the building footprint (generally one column spacing) to provide lateral support for the structure. Habitable structures are defined by the CGS as structures with 2,000 man-hours occupancy per year. The remainder of the site beneath auxiliary structures is generally not improved unless the structures are considered an essential facility (such as an emergency back-up generator). The area outside of the building footprints beneath auxiliary structures and paved areas would be overexcavated per the recommendations in this report.

3.2.2 Surface Treatment

Installation of VR columns typically causes the upper several feet of the ground surface to heave. Once the VR columns have been installed, the upper two feet of soil in the building foundation area should be over-excavated and recompacted to 90 percent relative compaction. The compacted material could consist of onsite granular soil or crushed rock.

For DSM projects, the loose disturbed soil in the upper portion of the site is removed to expose the upper part of the DSM columns. The surface treatment beneath the grade beam foundation treatment will be specified by the project civil and structural engineers based on the column and foundation configuration.

3.3 PRELIMINARY GRADING CONSIDERATIONS

3.3.1 General Site Clearing and Grubbing

Soil containing debris, organics, trees and root systems, and other unsuitable materials should be excavated and removed from improvement areas prior to commencing grading operations. Areas should be cleared of old foundations, slabs, pavement, abandoned utilities, and soils disturbed during the demolition process. Depressions or disturbed areas left from the removal of such material should be replaced with compacted fill.

3.3.2 Subgrade Preparation

For areas within the building foundation improved with VR, the ground surface should be overexcavated to a depth of two feet below the existing ground surface and replaced with compacted fill consisting of onsite granular soils or a blanket of crushed rock.

For improved areas outside of the building foundation ground improvement area, the ground surface should be overexcavated to a depth of two feet below the existing ground surface or two feet below footing depth, whichever is deeper. The resulting surface should be scarified to a depth of eight inches and compacted to 90 percent relative compaction (RC) and the fill placed above that level. Areas underlain by asphalt concrete pavement should be scarified to a depth of 12 inches and compacted to 95 percent RC.

3.3.3 Fill Material Selection

Recommended fill material selection requirements for subgrade fill, aggregate base, and use of onsite materials are presented below. Areas or zones where the various fill materials may be used are described below.

Use of Onsite Materials. As described above, the near-surface onsite materials consist of granular silty sand soils with some gravel and cobble-size rock fragments. The material generated from the site overexcavation can be utilized as compacted fill as long as those materials satisfy criteria for general fill.

General Fill. General fill should consist of granular soil materials (SP, SW, SP-SM, and SM) free of organics, oversize rock (greater than six inches in diameter), trash, debris, and other deleterious or unsuitable materials, and should have an expansion index less than 20. The fill materials should have less than 15 percent larger than three inches in diameter.

Aggregate and Miscellaneous Base. Base materials should consist of material conforming to Caltrans Standard Specifications for Class 2 Aggregate Base, Section 26-1.02 (Caltrans, 2015) or Section 200-2.5 of the Greenbook (2015) for Processed Miscellaneous Base.

Imported Fill. Although importing fill is not anticipated, if material is imported, the imported subgrade fill materials should comply with recommendations for general fill or as appropriate for its intended use. Imported fill should be reviewed by the geotechnical engineer prior to being transported to the site.

3.3.4 Dewatering

On the basis of our subsurface exploration and previous studies nearby, we do not anticipate groundwater will be encountered during site grading activities. Although we do not anticipate the need for dewatering, groundwater levels may vary seasonally and it is possible some seepage may be encountered in the excavations following rain events.

3.3.5 Fill Placement

Fill should be placed, moisture conditioned, and compacted to a minimum of 90 percent relative compaction. In general, we recommend the moisture content of the fill should be 0 to 2 percent above the optimum. We note the tested on-site soils have low moisture contents in the range of 2 to 8 percent. On the basis of the test results, water will need to be added during grading to bring the moisture content up near the optimum moisture content of about 10 to 11 percent. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material prior to placing the next layer.

Rock, cobbles, and other oversized material greater than six inches in dimension in any direction should be removed from the fill material being placed. The contractor should be prepared to screen all native materials prior to placement as compacted fill. Rocks should not be nested and voids should be filled with compacted material. Organics, foreign matter, and other deleterious materials also should be removed from any material used in constructed fills.

Fill and backfill materials should be placed in layers that can be compacted with the equipment being used. Fill should be spread in lifts no thicker than approximately eight inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction depending on the equipment being used.

3.3.6 Compaction

Fill placement and grading operations should be performed according to the City of Moorpark, Greenbook Specification 300-4, and the grading recommendations of this report. Relative compaction should be assessed based on the latest approved edition of ASTM D1557. The building and general site improvement over-excavation and upper 1-foot of paved areas (subgrade plus base materials) should be compacted to 95 percent relative compaction. We recommend general fill be compacted to a minimum of 90 percent relative compaction. Recommended specified relative compaction should extend to a minimum of three feet horizontally beyond the limits of the improvements.

3.4 SHALLOW FOUNDATION DESIGN

The following sections describes preliminary shallow foundation design parameters for the site assuming the seismic geohazards have been mitigated through ground improvement. Depending on the level of improvement and building design criteria, it may be necessary so support the proposed structure on a mat type foundation. The decision on the preferred foundation type should be coordinated with the project civil and structural engineers based on building settlement tolerances. The following sections provides preliminary shallow foundation parameters consisting of isolated and continuous footings designed in accordance with current CBC and Greenbook requirements assuming that those values are suitable for the proposed structure.

3.4.1 Allowable Bearing Pressure

Continuous and isolated spread footings will be supported on recompacted onsite materials underlain by alluvium. For these conditions, we recommend shallow footings be designed using a maximum allowable bearing pressure of 1,500 pounds per square foot (psf). The allowable value incorporates a factor of safety of at least 3. The toe-pressure below retaining walls or eccentrically loaded footings can exceed the recommended bearing pressure, provided the resultant pressure is within the middle-third of the footing. In accordance with 2016 CBC Section 1806.1, the bearing values indicated above are for static loads (including the total of dead and frequently applied live loads), and may be increased for short duration loading (including the effects of wind or seismic forces) as allowed in 2016 CBC Section 1605.3.2.

3.4.2 Minimum Embedment Depth and Width

In general, footings should be embedded to at least two feet below the adjacent grade and have a minimum width of 18 inches. Isolated pad footings should be at least three feet in least-dimension.

3.4.3 Sliding and Passive Resistance

Ultimate sliding resistance (friction) generated at the interface of concrete foundations and compacted soils can be computed by multiplying the total dead weight structural load by a coefficient of 0.40. The ultimate net passive resistance developed from lateral bearing of foundations against compacted backfill or undisturbed native soil can be estimated using an equivalent fluid weight of 300 pcf. The passive resistance for the upper one-foot of soil should be neglected unless the soils are confined at the ground surface by slab-on-grade or pavement.

Sliding resistance and passive pressure may be used together without reduction, when used with the recommended minimum factors of safety. For static conditions, minimum factors of safety of 1.5 and 2.0 are recommended for foundation overturning and sliding, respectively. The factor of safety for sliding can be reduced to 1.5, if passive resistance is neglected. The factor of safety for transient (seismic, wind) conditions should be at least 1.1.

3.4.4 Settlements

Static Settlements. Static settlements will generally occur in response to foundation loads on the foundation support material. The structure should be designed to accommodate static differential settlements of at least one-half-inch over a distance of 30 feet (i.e., a distortion ratio of approximately 1/720) for similarly sized and loaded footings.

Seismic Settlements. Seismically induced settlements are discussed previously in this report. We anticipate the alluvial soils underlying the proposed excavation could experience seismic settlement of 19 inches without ground improvement and up to four inches with ground improvement to 50 feet with associated differential settlements of two-inches across the site.

3.5 SLAB-ON-GRADE

At-grade floor slab thickness should be designed by the structural engineer, but should not be less than six inches thick. Control joints should be specified by the project structural engineer. The structural engineer should determine reinforcement requirements, but, at a minimum, reinforcement of on-grade floor slabs should consist of No. 4 bars at 18 inches each way, placed above slab mid-height with preferably about 1½- to 2-inches of clear cover. Means should be provided to maintain reinforcement location during construction and concrete placement.

Proper concrete placement in accordance with applicable specifications and curing of concrete slabs inhibits moisture migration. The concrete slab water cement ratio should be maintained during concrete mixing and placement. The project architect and design engineer should select the desired concrete properties based on the concrete slab-on-grade performance requirements.

The slab-on-grade should incorporate a moisture seal beneath the slab in areas where the concrete slab will be covered with flooring. The moisture seal should be bedded in sand per ACI criteria.

3.6 CONSTRUCTION CONSIDERATIONS

3.6.1 Existing Utilities

We recommend any existing utilities be removed from the grading areas and relocated as necessary. The removal should consist of the excavation of the existing trench backfill and subsequent placement of new compacted fill. Excavation work required for the abandonment of utilities is anticipated to be relatively nominal but should be considered in the construction documents.

Trenches should be excavated no closer than a 1 horizontal to 1 vertical (1h:1v) projection up from the bottom of the excavation in areas where an existing utility/pipeline parallel's or subparallels the trench excavation. The minimum clear distance between an existing utility and the trench should be evaluated by the contractor. We recommend existing utility/pipelines be supported/protected or the trench be shored to prevent loss of lateral support for existing utility/pipelines when: 1) the trench is closer than a 1h:1v projection to the existing utility, 2) the stability of the existing utility is in question, or 3) there is a potential for sloughing of the trench sidewalls adjacent to the existing utility.

3.6.2 Excavation Conditions

Subsurface materials encountered in our exploratory holes consisted of very loose to loose silty sand to sand (granular) sediments to the anticipated excavation depths. We expect excavations in those soils can be made using conventional heavy-duty equipment in good working order.

3.6.3 Temporary Slopes and Excavations

The contractor should be responsible for the design of temporary slopes. Subsurface materials encountered in our exploratory holes consisted of very loose to loose granular sediments to the anticipated excavation depths. Temporary slopes should be braced or sloped according to the requirements of OSHA.

As input to design, excavations without shoring that are shallower than 10 feet likely will be classified as Type C and should be sloped no steeper than 1.5h:1v as deemed appropriate based upon classification Type determined in the field per OSHA guidelines. OSHA requires excavations greater than 20 feet deep be designed by a qualified professional. We recommend all temporary excavations be monitored for signs of instability and appropriate actions (such as flattening the slope, providing shoring, and controlling groundwater, if encountered) should be undertaken if evidence of potential instability is observed

3.7 PRELIMINARY PAVEMENT DESIGN

3.7.1 Subgrade Preparation

The finished subgrade surface exposed after overexcavation should be scarified to a depth of 12 inches, moisture-conditioned to within 0 to 2 percent of optimum moisture, and compacted to a relative compaction of at least 90 percent (i.e., 90 percent of the maximum dry density determined from ASTM D1557).

3.7.2 Fill Material Selection

Recommended fill material selection requirements for subgrade fill, aggregate base, and use of onsite materials are presented below. Areas or zones where the various fill materials may be used are described below.

Subgrade Fill. General fill should be free of organics, oversize rock (greater than 3 inches in diameter), trash, debris, and other deleterious or unsuitable materials, and should have an expansion index less than 20.

Aggregate and Miscellaneous Base. Aggregate or miscellaneous base material should be placed below the asphalt pavement. Base materials should consist of imported material conforming to Caltrans Standard Specifications for Class 2 Aggregate Base, Section

26-1.02 (Caltrans, 2015) or Section 200-2.5 of the Greenbook (2015) for Processed Miscellaneous Base.

Use of Onsite Materials. Materials generated during excavation and grading in pavement areas are generally anticipated to consist primarily of granular soil materials. Material derived from the overexcavation can be used as subgrade as long as those materials satisfy criteria presented above for subgrade fill.

Imported Fill. Imported subgrade fill materials should comply with recommendations for subgrade fill or as appropriate for its intended use. Imported fill should be reviewed by the geotechnical engineer prior to being transported to the site.

3.7.3 Pavement Section

A flexible pavement design section was estimated using the County of Ventura pavement design procedures for assumed Traffic Index (TI) of 5, an R-value of 70 for the tested onsite sandy subgrade materials, and our experience. The recommended asphalt pavement sections based on the assumed TIs and the R-value test data are presented in the following table. Asphalt pavement materials should be compacted to at least 95 percent relative compaction.

Table 9. Asphalt Pavement Section

R-Value	Traffic Index	Thickness of Asphalt Concrete (in)	Thickness of Aggregate Base (in)
70	5	3	4

3.8 SITE DRAINAGE

Site grading should be provided such that positive drainage away from improvements is provided. Water should not be allowed to pond near the improvements; we recommend the construction of finish slopes of 1 to 2 percent away from improvements.

3.9 STORMWATER INFILTRATION

Recent regulatory agency requirements mandate stormwater generated on a new project site be infiltrated into the onsite soils. While this concept may have merit from an environmental standpoint, it increases the potential to cause foundation damage to onsite improvements due to higher groundwater levels, reduced soil strength, hydroconsolidation of onsite soils, and moisture infiltration into buried structures. If onsite stormwater disposal is implemented at the site, the design needs to consider the locations of existing and proposed structures and impacts to offsite improvements.

The liquefaction analyses performed for this study indicates up to about 12 inches of liquefaction related settlement could occur in response to the design seismic event. Infiltration of stormwater could increase groundwater levels beneath the site and reduce the shear strength of the soils which would increase the potential for liquefaction related settlement. In addition, the study indicated the potential for hydroconsolidation (collapse) of the onsite soil as high as 2.3 percent at a depth of 10 feet in areas not mitigated by ground improvement. A collapse of 2 percent over a depth of 35 feet (depth above current groundwater level) is equivalent to a collapse settlement of about 8 inches. Previous experience with collapse related settlements indicates concentrated water infiltration can cause hydroconsolidation of soils with collapse potential. Those concentrated settlements are typically associated with leaking water or sewer pipelines, but in our opinion, concentrated stormwater infiltration in a discrete basin has the potential to cause hydroconsolidation of the soils. The settlement contours from soil hydroconsolidation related settlement measured in previous forensic studies in the Ventura area documented a radial settlement pattern extending up to about 100 feet from the water infiltration source.

If storm water is infiltrated at the proposed library site, we suggest the project civil engineer consider the above factors in the design process. If concentrated stormwater infiltration is proposed in a discrete basin, that basin should be located away from project structural elements and offsite improvements (including buried utilities) that could be impacted by settlement. A setback distance of at least 100 feet from a discrete infiltration location is likely a reasonable starting point for infiltration design. Another alternative would be a diffuse infiltration system that does not concentrate infiltration in a specific location.

4.0 LIMITATIONS

4.1 REPORT USE

This preliminary report has been prepared for the exclusive use of the City of Moorpark for evaluation of the liquefaction potential and need for ground improvement to mitigate potential settlements that may occur as a result of earthquake-induced ground shaking at the library site. This preliminary report is intended to provide a summary of the site conditions, geohazard assessment, proposed ground improvement mitigations, and preliminary foundation recommendations. A supplemental geotechnical design report will be required as part of project design once the building type and location have selected, ground improvement option identified and foundation support conditions determined. The findings, conclusions, and recommendations presented herein were prepared in accordance with generally accepted geotechnical engineering practices of the project region. No other warranty, express or implied, is made.

Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the project as described in this report, the conclusions and recommendations in this report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report are modified or validated in writing by OGI.

4.2 HAZARDOUS MATERIALS

This report does not provide information regarding the presence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere.

4.3 LOCAL PRACTICE

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

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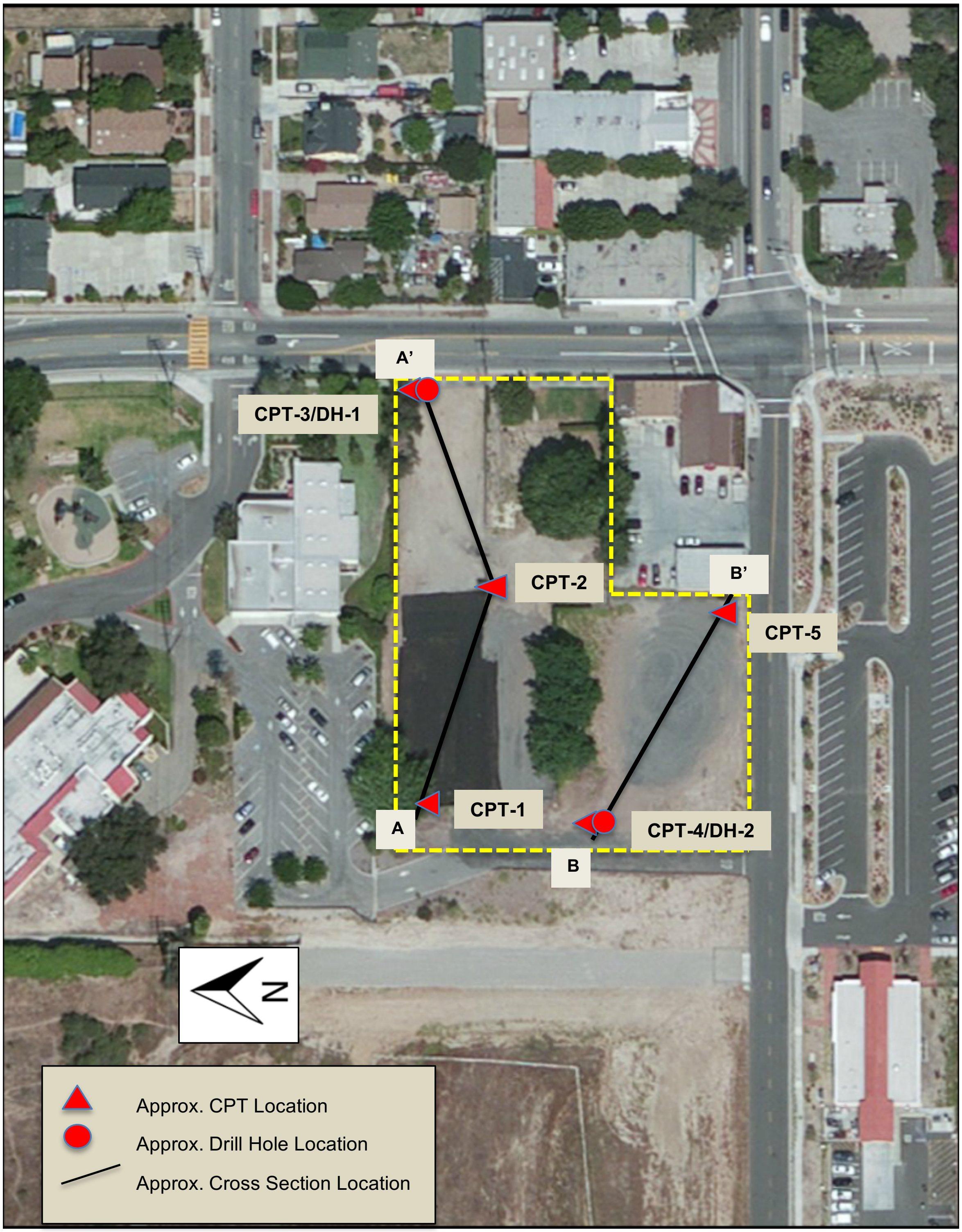
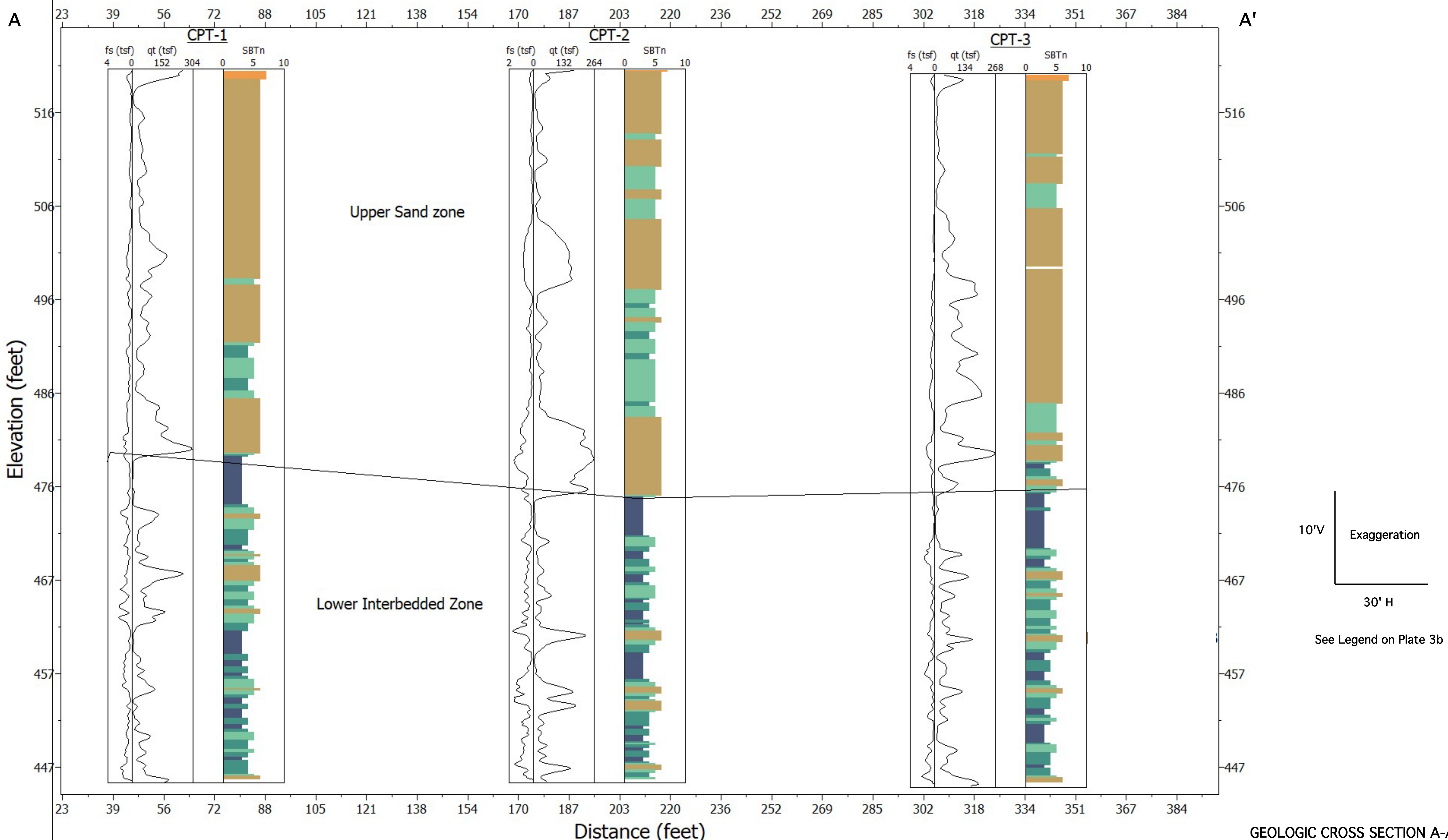
PLATES

Source: Google Earth, 2017

VICINITY MAP
Proposed New Library Site
Moorpark, California



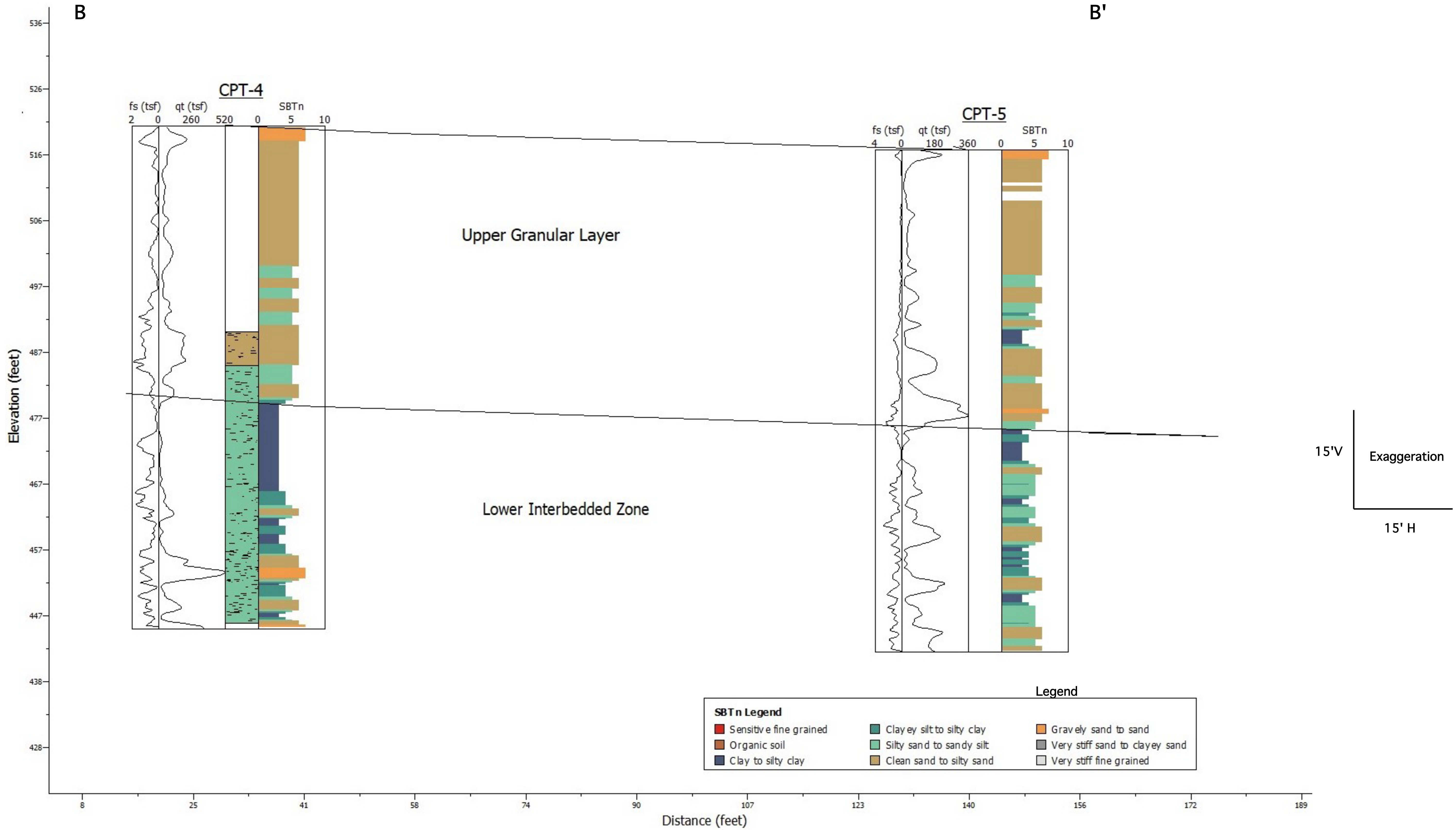
EXPLORATION LOCATIONS
Proposed New Library Site
Moorpark, California



NOTE: Elevations are approximate and are based on USGS 7.5 minute topographic map of Moorpark quadrangle.

GEOLOGIC CROSS SECTION A-A'
Proposed New Library Site
Moorpark, California

PLATE 3a



NOTE: Elevations are approximate and are based on USGS 7.5 minute topographic map of Moorpark quadrangle.

GEOLOGIC CROSS SECTION B-B'
Proposed New Library Site
Moorpark, California

PLATE 3b

APPENDIX A

LOG OF DRILL HOLE DH-1

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
			1		**ARTIFICIAL FILL (af)** Silty Fine SAND (SM): pale brown, dry, with gravel					
2					**ALLUVIUM (Qal)?** SAND (SP): very loose, moderate yellowish brown, damp					
			2	(5)		98	4			
4			3	2	Fine SAND with Clay (SP-SC): very loose, dark brown, damp					
6										
8			4	(7)	- loose, at 7'	105	8			
10			5	4	Silty Fine to Medium SAND (SM): very loose, moderate yellowish brown, damp, with scattered course grains, and with few fine rounded gravel to 1/2"-dia.		6		15	
12			6	(10)	Clayey SILT with Sand (ML): medium stiff, moderate to dark brown, damp	111	15			
					SAND with Clay (SP-SC): loose, moderate brown, damp, with scattered coarse sand					
14			7	7	SAND with Silt (SP-SM): loose, moderate yellowish brown, damp, with fine rounded gravel to 1/2"-dia.		5		7	
16										
18										
			8	(23)	- with medium dense, dark brown sand with clay, from 19' to 21.25'	108	2			

CONTRACTOR:	S/G Drilling, Inc.	TOTAL DEPTH (ft):	75.5'
METHOD:	8"-dia. Hollow-stem-auger	WATER DEPTH (ft):	37.5'
BACKFILL:	Cuttings with Portland	LOGGED BY:	L Prentice
DATE:	May1-2, 2017	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-1 (Continued)

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
22								
24	9	9	Silty Fine to Medium SAND (SM): loose, pale yellowish brown, damp, with few scattered coarse sand and 3/4" gravel fragments		5		23	
26								
28								
30	10	(37)	Fine to Medium SAND with Silt (SP-SM): medium dense, pale yellowish brown, damp	102	3			
32								
34	11	20	- with 1.5"-thick moderate yellowish brown clayey fine sand, at 34.5'		8		7	
36								
38								
	12	2	Clayey SAND (SC)/Sandy CLAY (CL): very loose, moderate brown, wet; shut down after sampling for 5 min.; measured water at 37.5'		21		50	

CONTRACTOR:	S/G Drilling, Inc.	TOTAL DEPTH (ft):	75.5'
METHOD:	8"-dia. Hollow-stem-auger	WATER DEPTH (ft):	37.5'
BACKFILL:	Cuttings with Portland	LOGGED BY:	L Prentice
DATE:	May1-2, 2017	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-1 (Continued)

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
42										
44			13	(12)	- loose, at 44' - flowing/caving sand below 44'; adding water to augers prior to sampling below 49'	112	19			p 0.5
46										
48										
50			14	(36)	Clayey SAND (SC): medium dense, moderate brown, wet - sand slough in sampler, blow counts may be affected	112	17		25	
52										
54			15	21	Fine to Medium SAND (SP): loose to medium dense, pale yellowish brown, wet, with moderate brown clayey fine sand in sampler shoe; sand slough in sampler		14		3	
56			15b							
58										
			16	14	Clayey SAND (SC): medium dense, moderate brown, wet		16		25	

CONTRACTOR:	S/G Drilling, Inc.	NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.	TOTAL DEPTH (ft):	75.5'
METHOD:	8"-dia. Hollow-stem-auger		WATER DEPTH (ft):	37.5'
BACKFILL:	Cuttings with Portland		LOGGED BY:	L Prentice
DATE:	May1-2, 2017		CHECKED BY:	C Prentice

LOG OF DRILL HOLE DH-1 (Continued)

LOCATION: See Location Map
SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	SAMPLE NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
67	17	7	Fine Sandy CLAY (CL): medium stiff, moderate to dark brown, wet slightly micaceous, silty		24		63	
69	18	22	Fine to Medium Clayey SAND (SC): medium dense, moderate brown, wet, with few coarse sand					
74	19	20	Silty Fine SAND (SM): medium dense, moderate brown, wet		25		24	

CONTRACTOR: S/G Drilling, Inc.
METHOD: 8"-dia. Hollow-stem-auger
BACKFILL: Cuttings with Portland
DATE: May1-2, 2017

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

TOTAL DEPTH (ft): 75.5'
WATER DEPTH (ft): 37.5'
LOGGED BY: L Prentice
CHECKED BY: C Prentice

LOG OF DRILL HOLE DH-2

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	MATERIAL SYMBOL	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
			1		**ARTIFICIAL FILL (af)** Silty Fine SAND (SM): with gravel, medium dense, grayish brown, dry to damp					
2			2	15						
4					**ARTIFICIAL FILL (af)/ALLUVIUM (Qal)?** Clayey SAND (SC): medium dense, dark brown, dry to damp					
			3	(15)	**ALLUVIUM (Qal)** Silty Fine SAND (SM): loose, moderate brown, dry to damp	97	4			
6										
					- loose, damp, fine to medium grained, at 7'					
8			4	6			4		22	
					- fine to medium grained, darker, at 9'					
10			5	(14)	- with dark brown fine silty lenses, at 9.75'	101	4			
12					Fine to Medium SAND with Silt (SP-SM): loose, moderate brown, damp					
			6	7			3		12	
14					Silty Fine SAND (SM): loose, moderate brown, damp					
			7	(15)	- with medium stiff, moderate brown silt with slight mottling and few fine root hairs and minor fine caliche, at 14 to 15'	106	5		32	
16					- fine to medium grained with few scattered coarse sand, at 15'					
18										
			8	13	medium dense, pale yellowish brown, at 19'					

CONTRACTOR:	S/G Drilling, Inc.	TOTAL DEPTH (ft):	50.5'
METHOD:	8"-dia. Hollow-stem-auger	WATER DEPTH (ft):	36'
BACKFILL:	Cuttings with Portland	LOGGED BY:	L Prentice
DATE:	May1-2, 2017	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-2 (Continued)

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	SAMPLE	NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
22									
24		9	(15)	- loose, at 24' - with finely laminated sandy silt and silt in sampler shoe, at 25.5'	90	6		29	
26									
28									
30		10	21	Fine to Medium SAND (SP): medium dense, pale yellowish brown, damp, with few coarse sand and few angular gravel fragments to about 1/2"-dia.					
32									
34		11	(24)	Clayey Fine to Medium SAND (SC): medium dense, dark brown, moist to wet Medium to Coarse SAND (SP): medium dense, moderate brown, moist to wet	97	18		27	
36									
38									
		12	WOH	Sandy Silty CLAY (CL-ML): very soft, dark brown, wet		21		52	p 0.1

CONTRACTOR:	S/G Drilling, Inc.	TOTAL DEPTH (ft):	50.5'
METHOD:	8"-dia. Hollow-stem-auger	WATER DEPTH (ft):	36'
BACKFILL:	Cuttings with Portland	LOGGED BY:	L Prentice
DATE:	May1-2, 2017	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

LOG OF DRILL HOLE DH-2 (Continued)

LOCATION: See Location Map

SURFACE EL. (ft): (ref. MSL datum)

DEPTH (feet)	SAMPLE NUMBER	BLOW COUNT	MATERIAL DESCRIPTION	DRY DEN. (pcf)	MOISTURE CONTENT %	PLASTICITY (LL/PI)	% PASSING No. 200	TV or PP (tsf)
			- shut down after sampling for 5 min.; measured water at 36'					
			- very stiff, at 41'					
42	13	(27)		118	16		47	p 2.3 p 2.7
			Medium to Coarse Clayey SAND (SC): medium dense, dark brown, wet					
44	NR	(10)	- no recovery after sampling at 44'					
46								
48								
50	14	(19)	- no recovery after sampling at 49'; recovered sample with SPT.		17		27	
52								
54								
56								
58								

CONTRACTOR:	S/G Drilling, Inc.	TOTAL DEPTH (ft):	50.5'
METHOD:	8"-dia. Hollow-stem-auger	WATER DEPTH (ft):	36'
BACKFILL:	Cuttings with Portland	LOGGED BY:	L Prentice
DATE:	May1-2, 2017	CHECKED BY:	C Prentice

NOTE: The log and data presented herein are a simplification of actual subsurface conditions encountered at the time of exploration at the specific location explored. Subsurface conditions may differ at other locations and at this location with the passage of time.

MOORPARK LIBRARY
Moorpark, California

Summary of Sampling Details

Symbol	Number	Blowcount Push, or grab	Sampler Type
	1	Bulk	Bulk Sample
	2	23	Standard Penetration Test (SPT) Sampler (1-3/8" ID/2" OD) driven
	3	(23)	Modified California Liner Sampler driven (2-3/8" ID/3" OD)
	4	Push	Thin-walled sampler pushed (2-7/8" ID/3" OD)

Blowcount Informtion

Blowcount	Description
63	63 blows for 1' penetration after initial 6" seating
89/11	89 blows for 11" penetration after initial 6" seating
33/6	33 blows for 6" drive after initial 6" seating
Ref	>50 blows for initial 6" seating
(23)	Blowcounts for modified California sampler

Material Symbols and Classifications

- LEAN CLAY (CL)
- FAT CLAY (CH)
- Sandy CLAY (CL)
- SILT (ML)
  Sandy SILT (ML)
- ELASTIC SILT (MH)
- Sandy SILT (ML)
- Silty SAND (SM)
- SAND with Silt (SP-SM)
  SAND with Clay (SP-SC)
- SAND (SP)
- Clayey SAND (SC)
- GRAVEL (GP)
- CLAYSTONE
- SILTSTONE
- SANDSTONE
- VOLCANIC
- DOLOMITIC
- SILICEOUS
- PAVING AND BASE MATERIALS
- CONCRETE

Other Symbols

- Groundwater
- Strata break

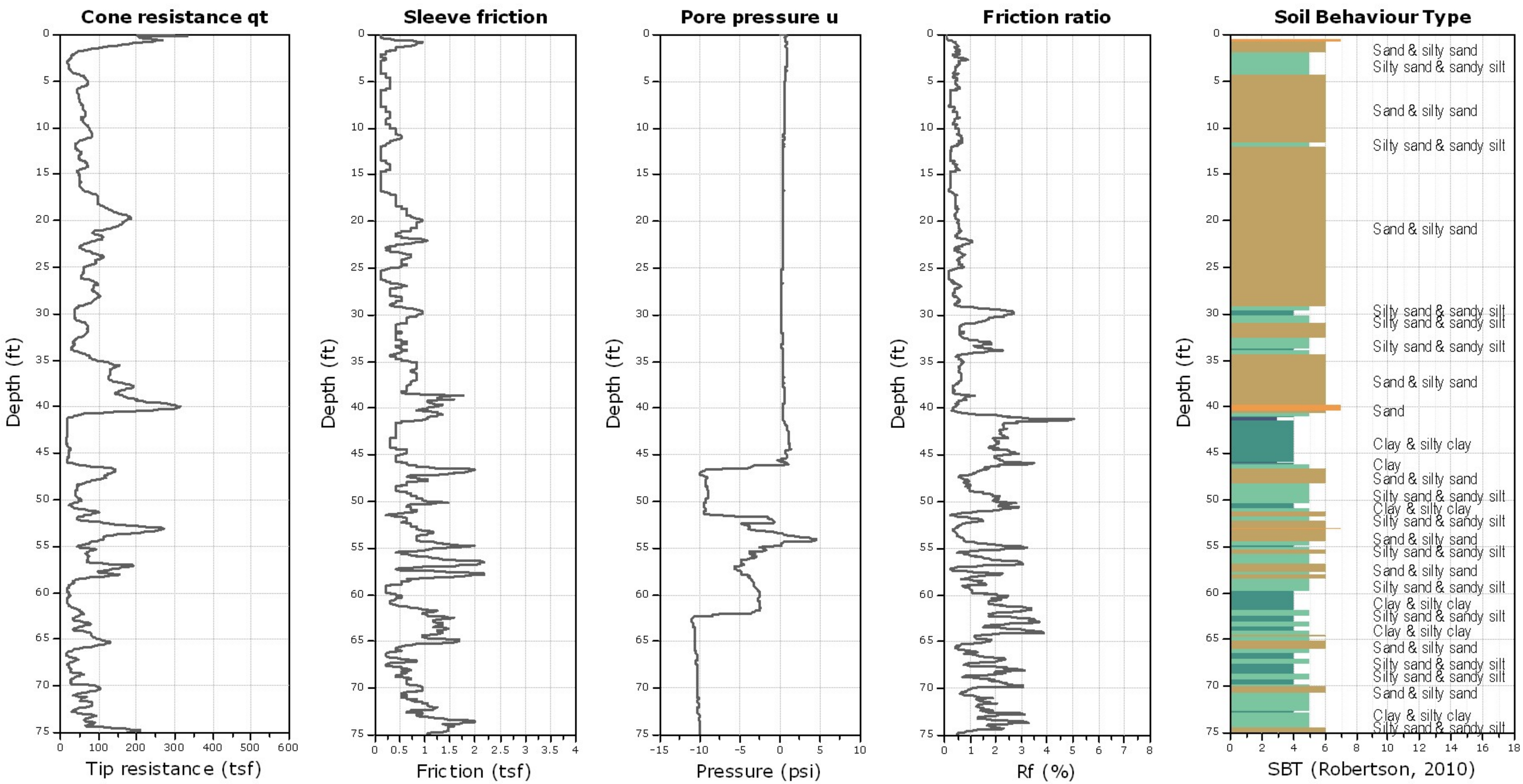
SUMMARY OF TERMS AND SYMBOLS USED ON LOGS

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **Oakridge Geoscience, Inc./MoorPark Library Project**
Location: **W. High St & Moorpark Ave Moorpark, CA**

Total depth: 75.29 ft, Date: 4/27/2017
Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 4/28/2017, 11:49:28 AM
Project file: C:\OakridgeMoorPk4-17\Plot Data\Plots.cpt

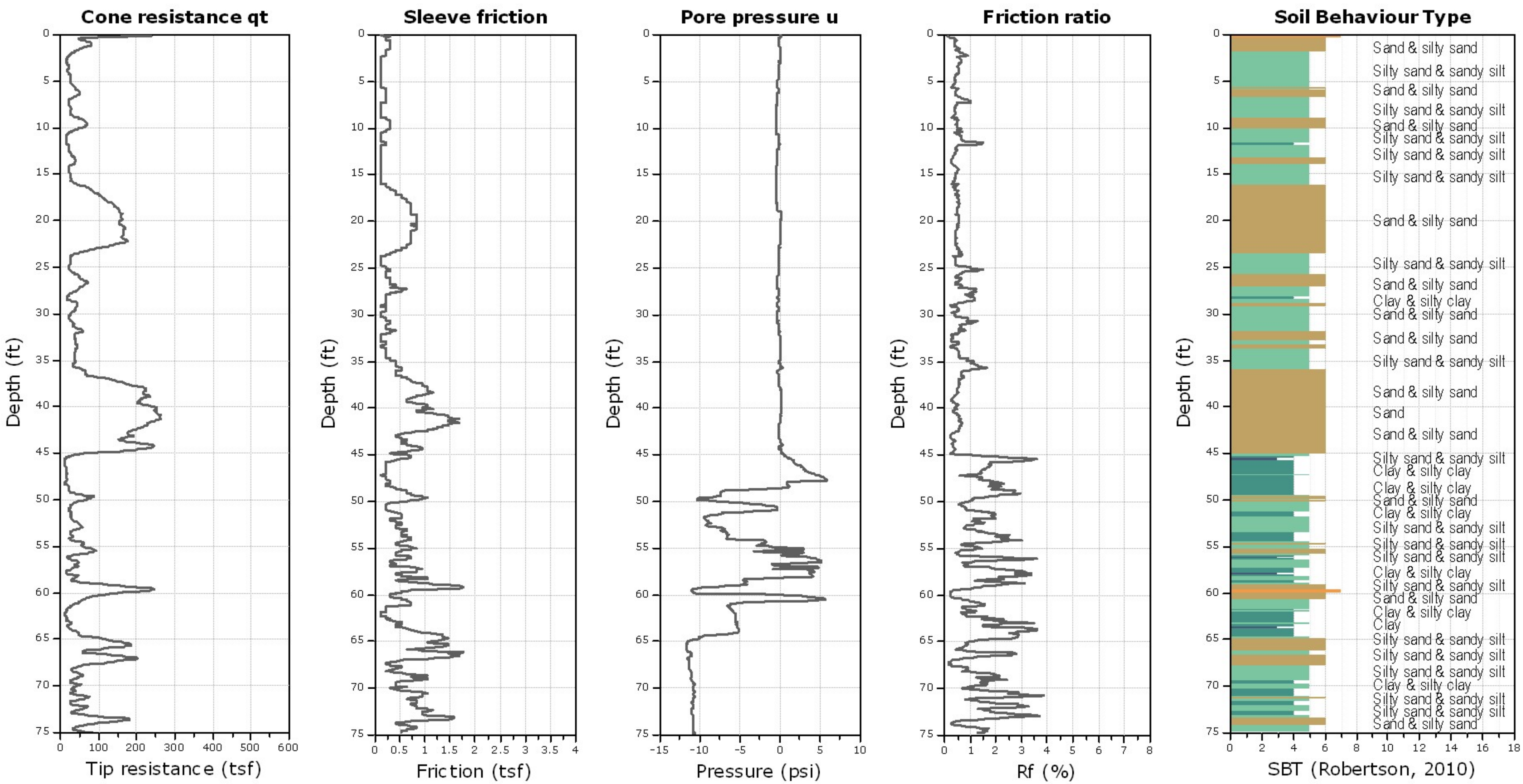
PLATE A-4

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-2

Project: **Oakridge Geoscience, Inc./MoorPark Library Project**
Location: **W. High St & Moorpark Ave Moorpark, CA**

Total depth: 75.14 ft, Date: 4/27/2017
Cone Type: Vertek

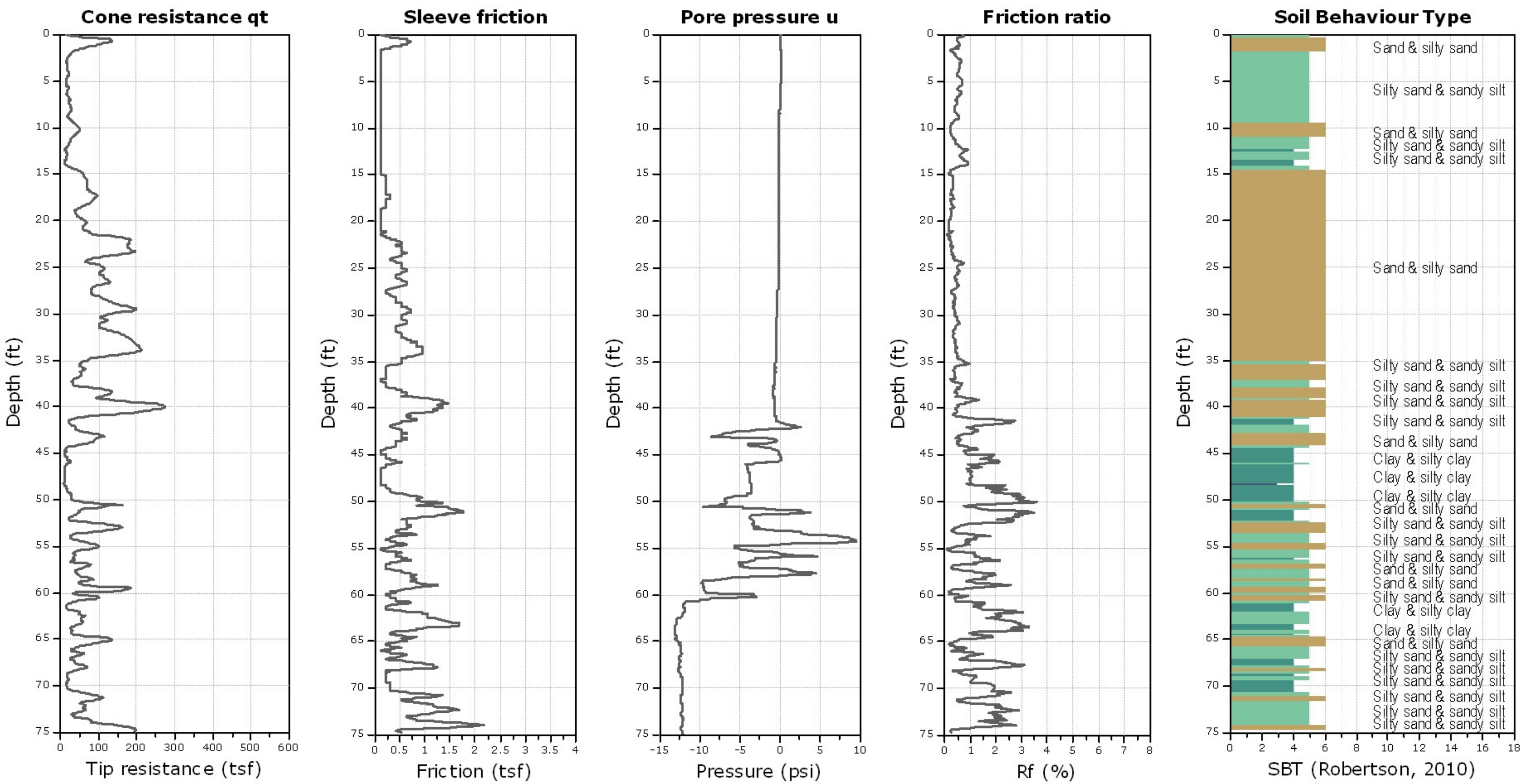


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-3

Project: **Oakridge Geoscience, Inc./MoorPark Library Project**
Location: **W. High St & Moorpark Ave Moorpark, CA**

Total depth: 75.07 ft, Date: 4/27/2017
Cone Type: Vertek

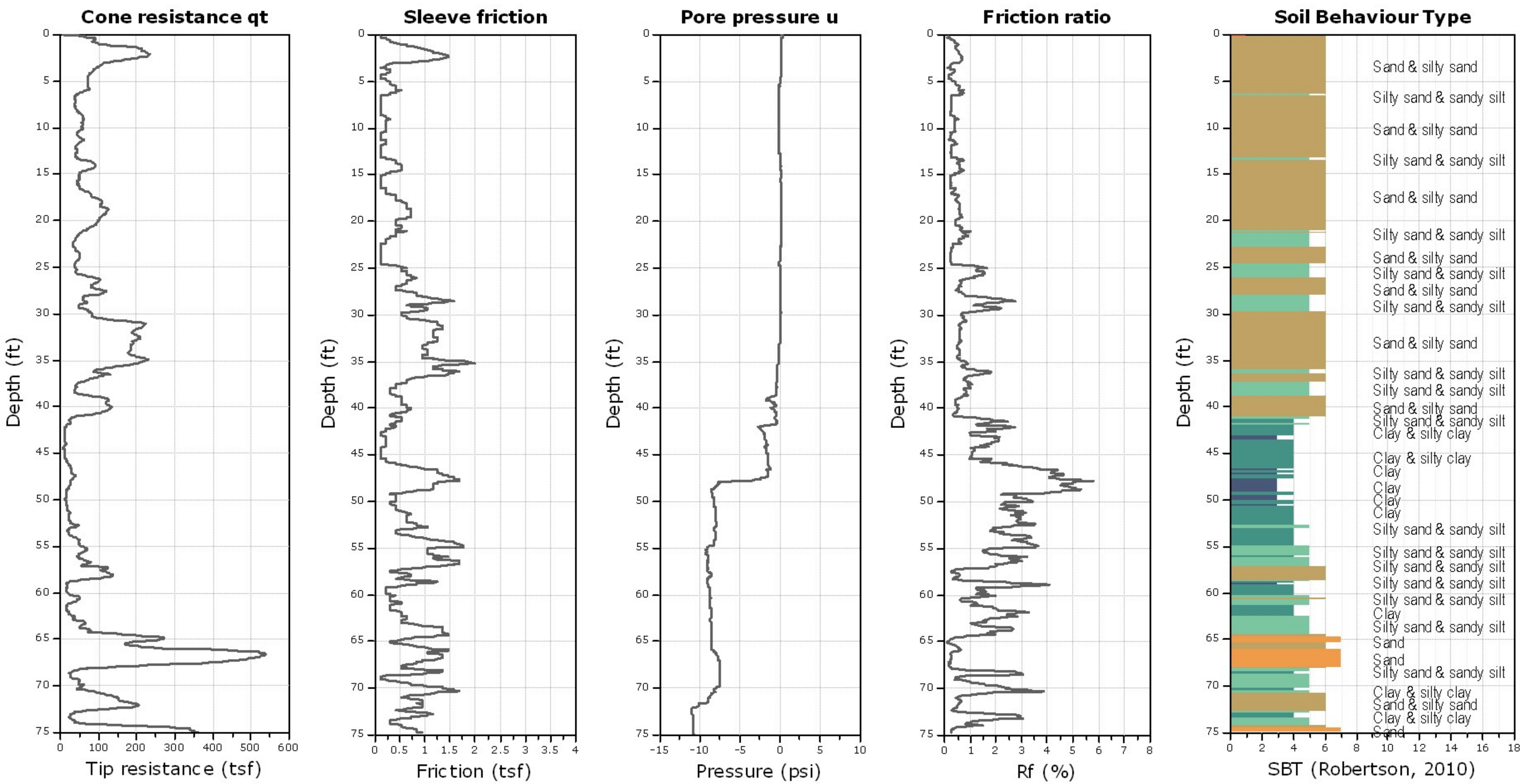


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-4

Project: **Oakridge Geoscience, Inc./MoorPark Library Project**
Location: **W. High St & Moorpark Ave Moorpark, CA**

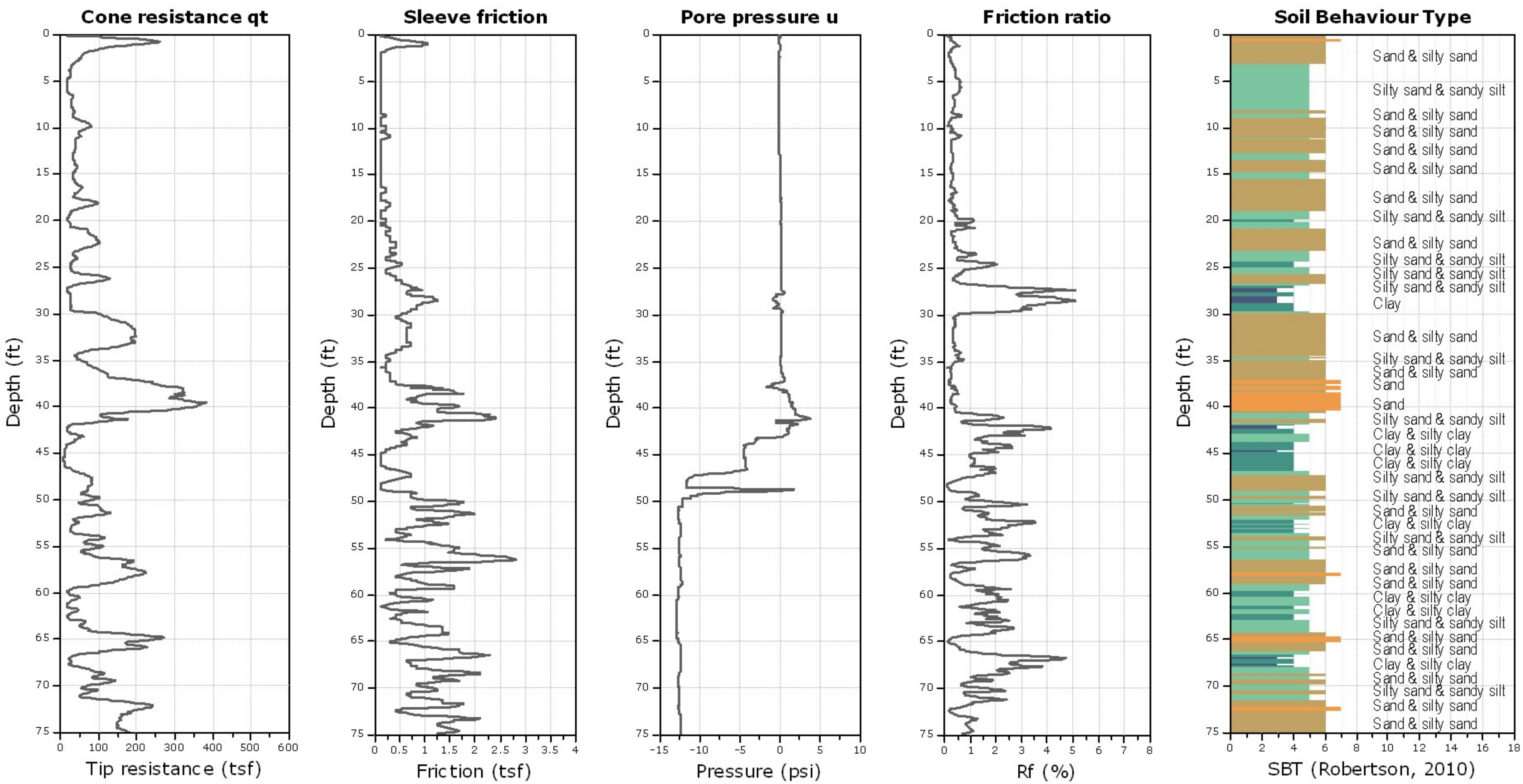
Total depth: 75.16 ft, Date: 4/27/2017
Cone Type: Vertek



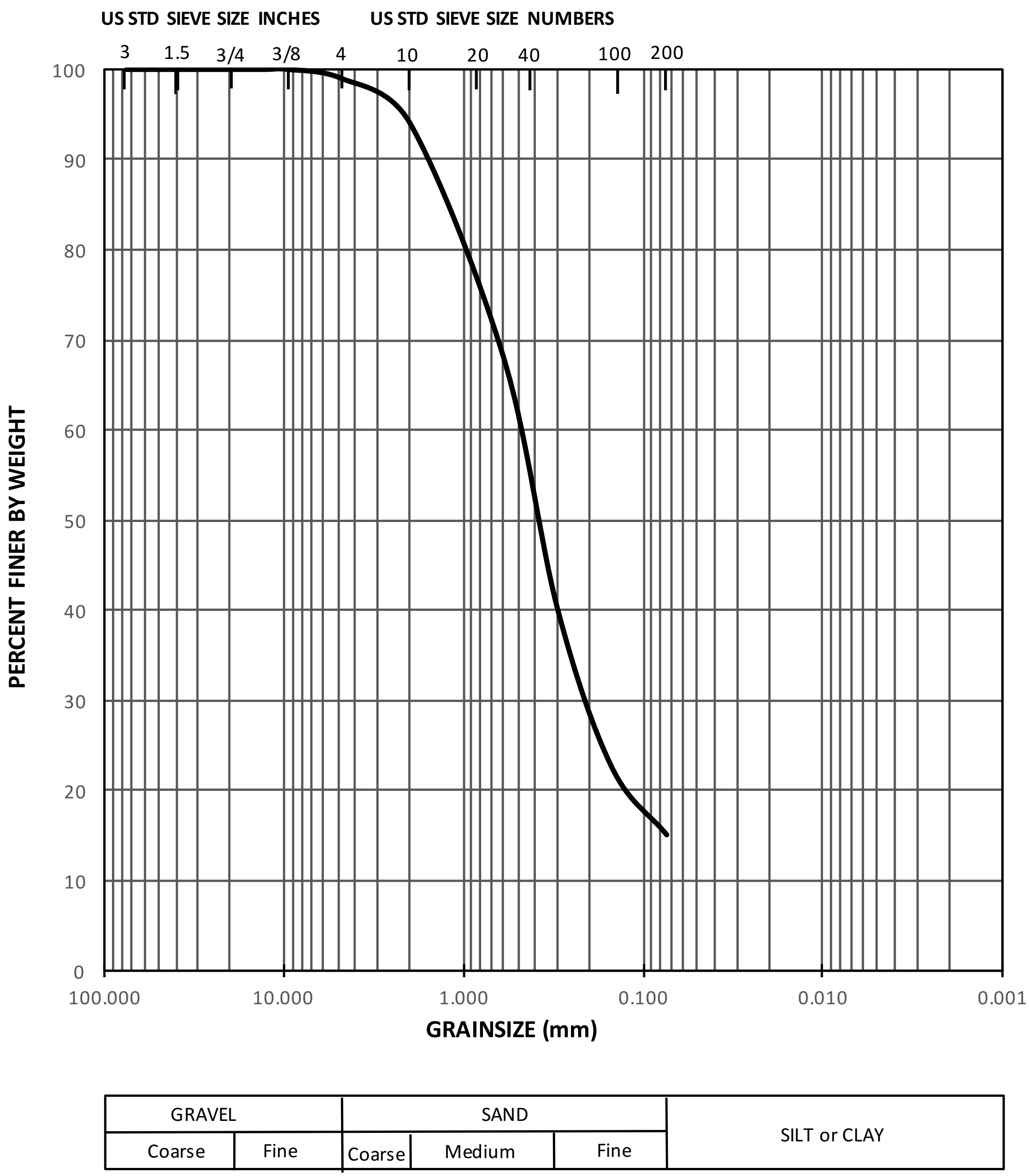
Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-5
Total depth: 75.27 ft, Date: 4/27/2017
Cone Type: Vertek

Project: **Oakridge Geoscience, Inc./MoorPark Library Project**
Location: **W. High St & Moorpark Ave Moorpark, CA**

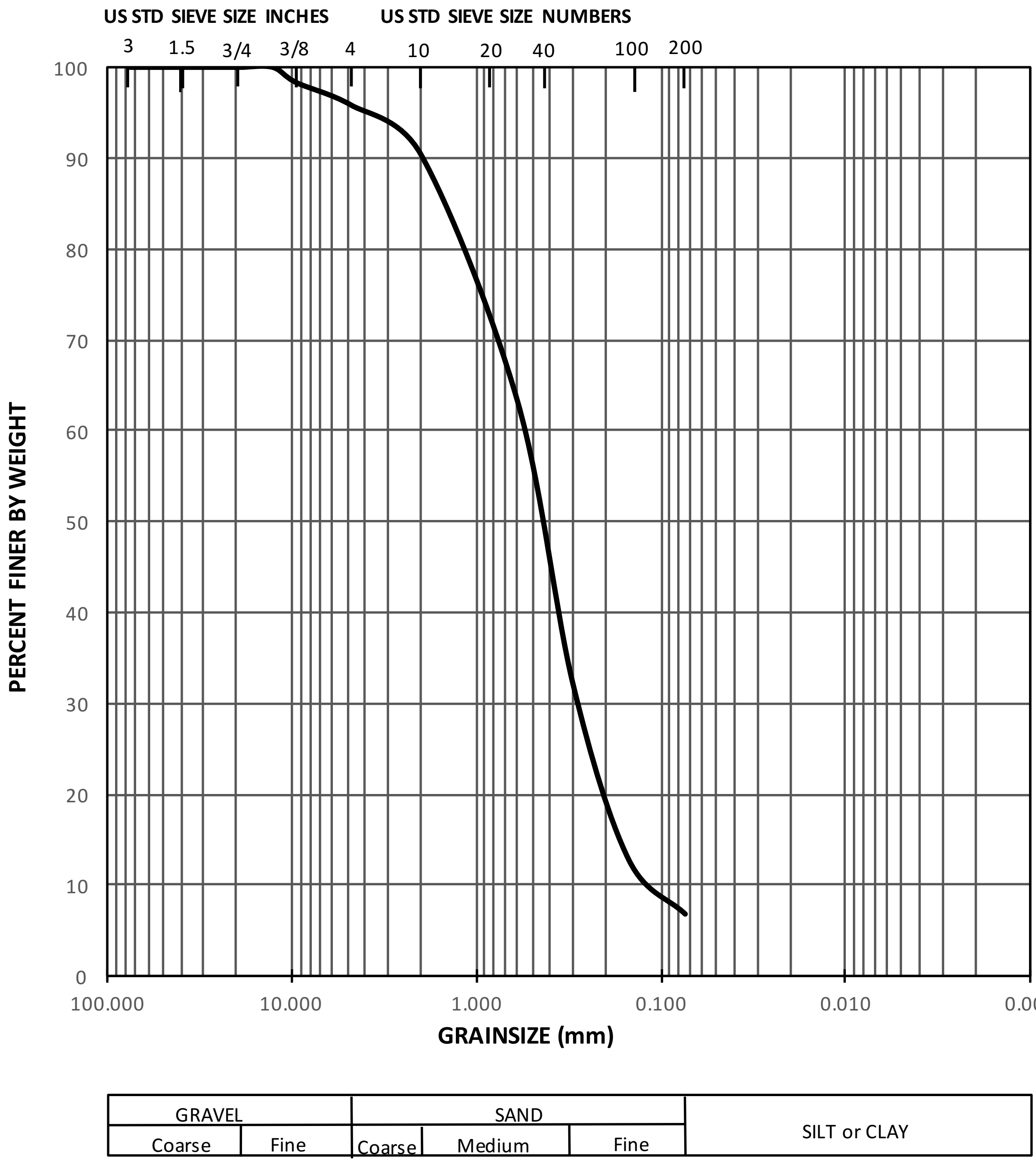


APPENDIX B



LOCATION	DH-1	CLASSIFICATION	PASSING NO. 200 (%)
DEPTH	10'	Silty Fine to Medium SAND (SM)	15

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

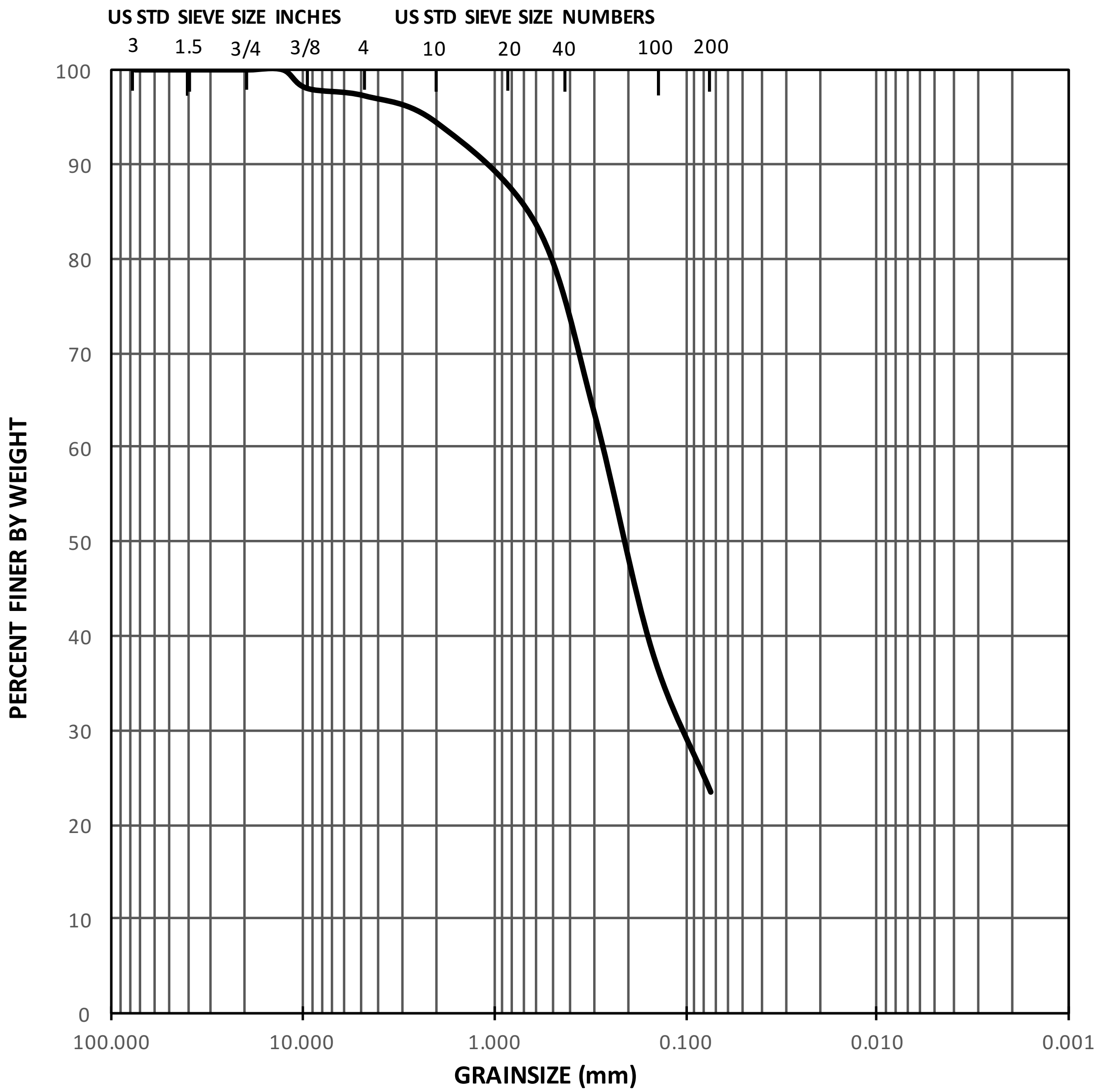


LOCATION DH-1
DEPTH 15'

CLASSIFICATION
Fine to Medium SAND with Silt (SP-SM)

PASSING NO. 200 (%)
7

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

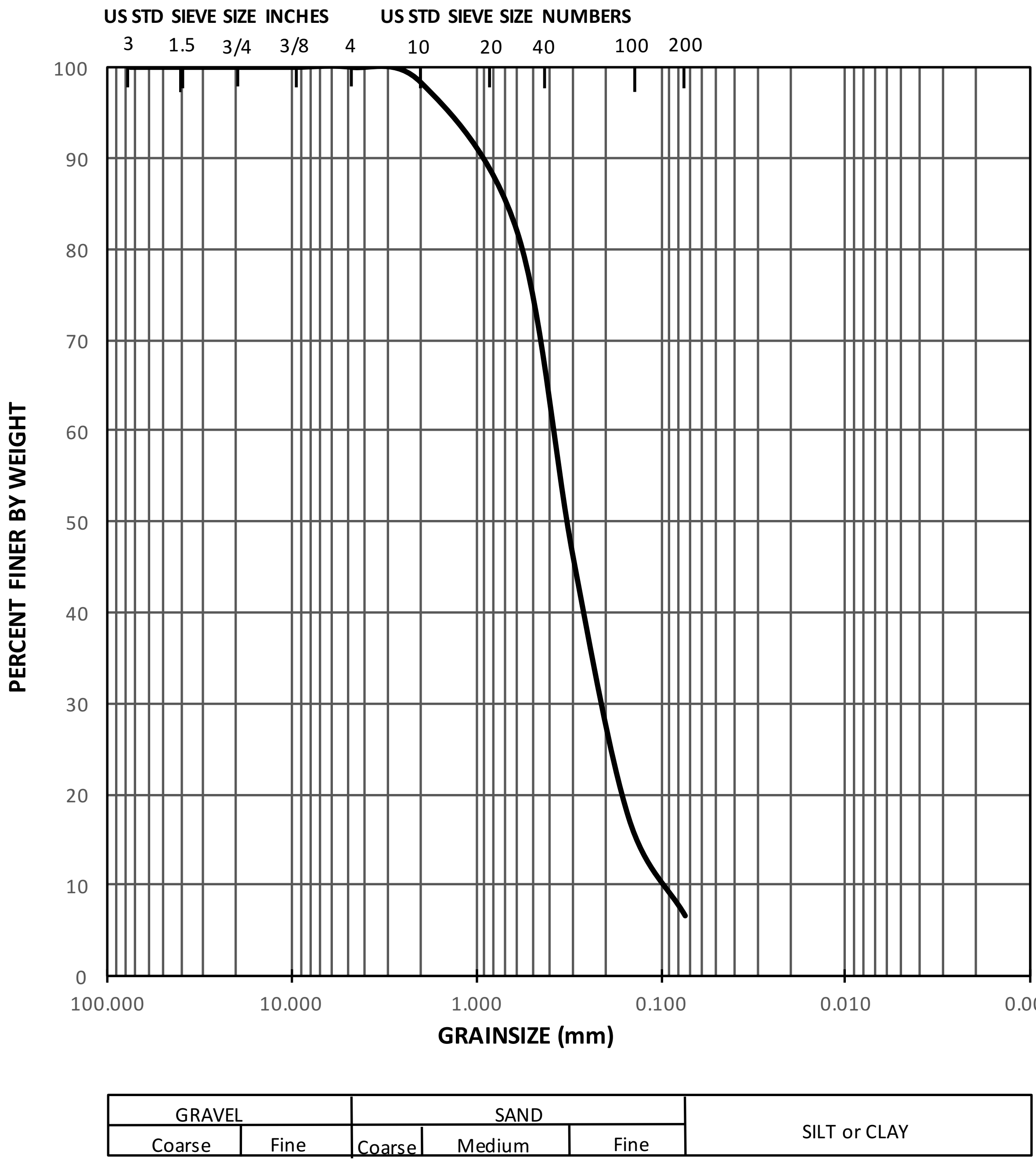
LOCATION DH-1
DEPTH 25'

CLASSIFICATION
Silty Fine to Medium SAND (SM)

PASSING NO. 200 (%)
23

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1c



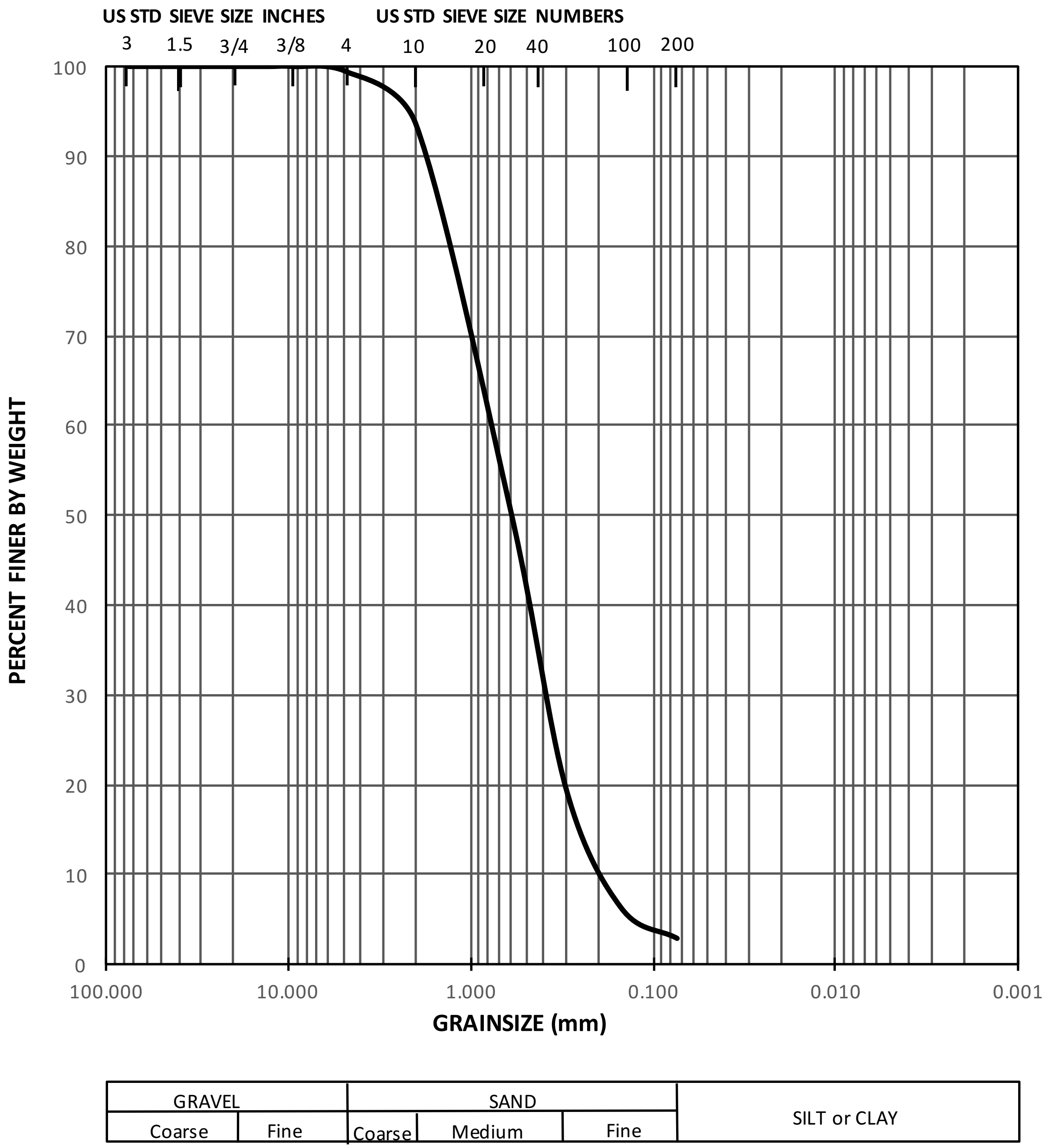
LOCATION DH-1
DEPTH 35'

CLASSIFICATION
Fine to Medium SAND with Silt (SP-SM)

PASSING NO. 200 (%)
7

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1d



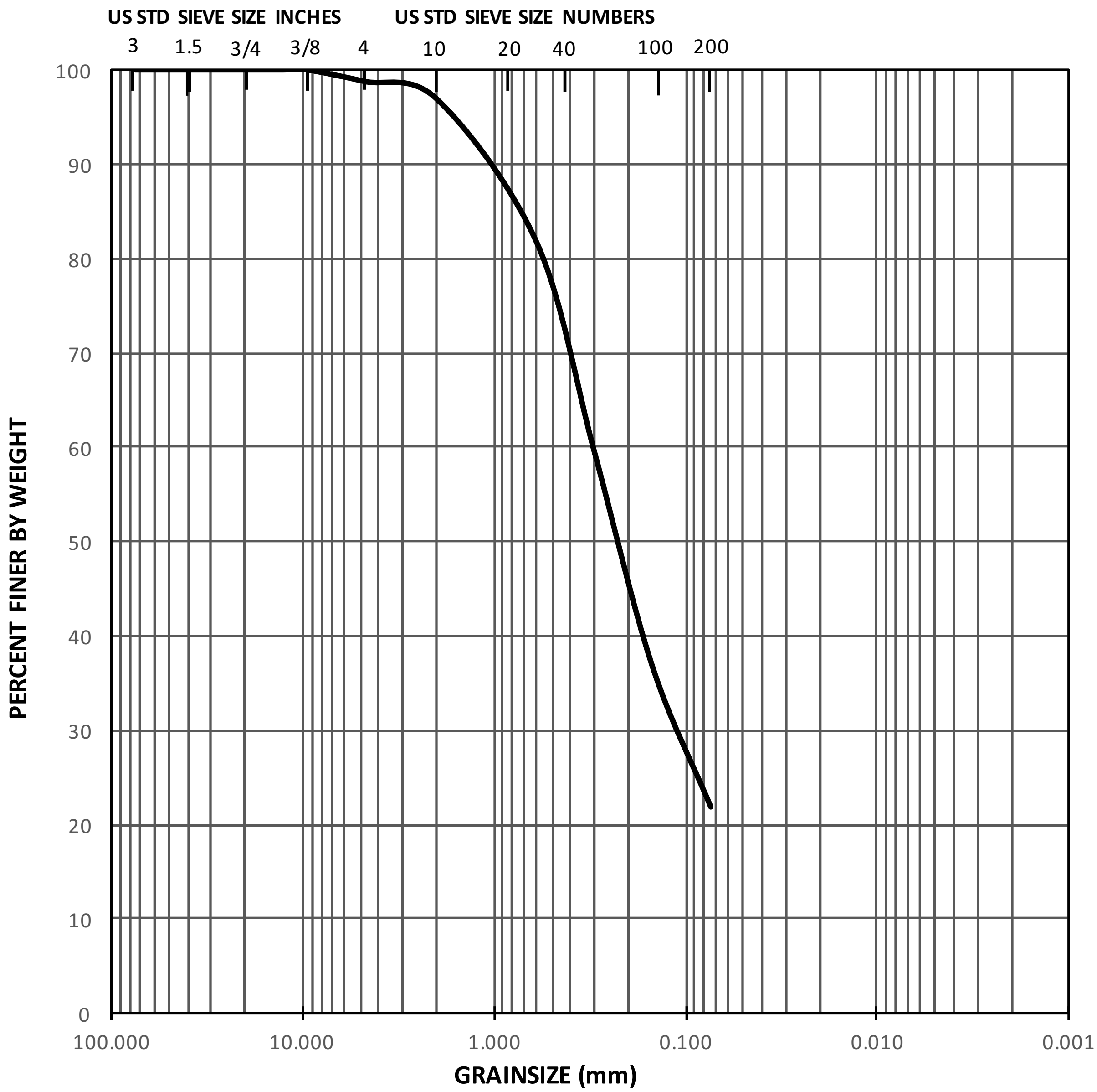
LOCATION DH-1
DEPTH 55'

CLASSIFICATION
Fine to Medium SAND (SP)

PASSING NO. 200 (%)
3

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1e



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

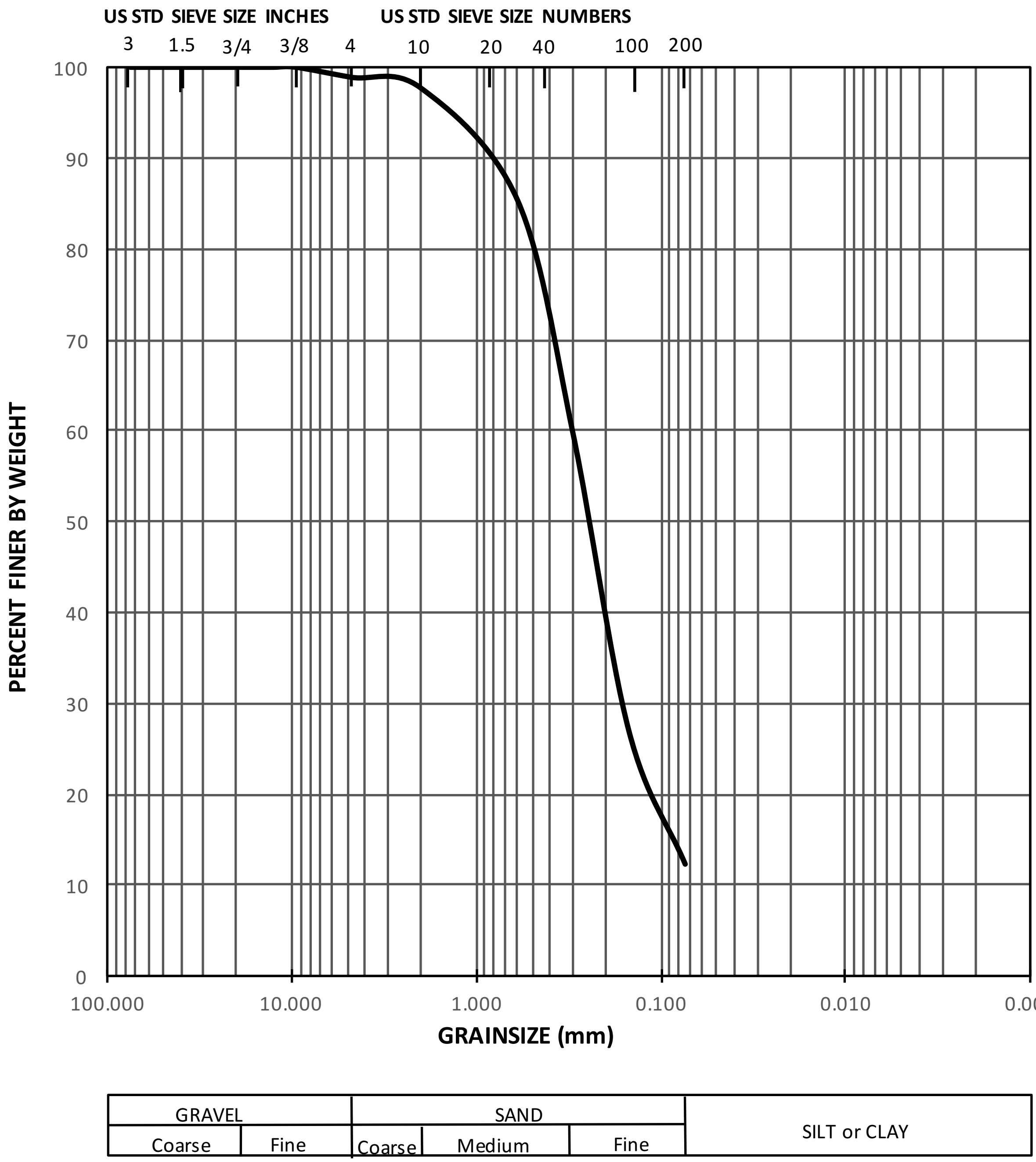
LOCATION DH-2
DEPTH 8'

CLASSIFICATION
Silty Fine to Medium SAND (SM)

PASSING NO. 200 (%)
22

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1e



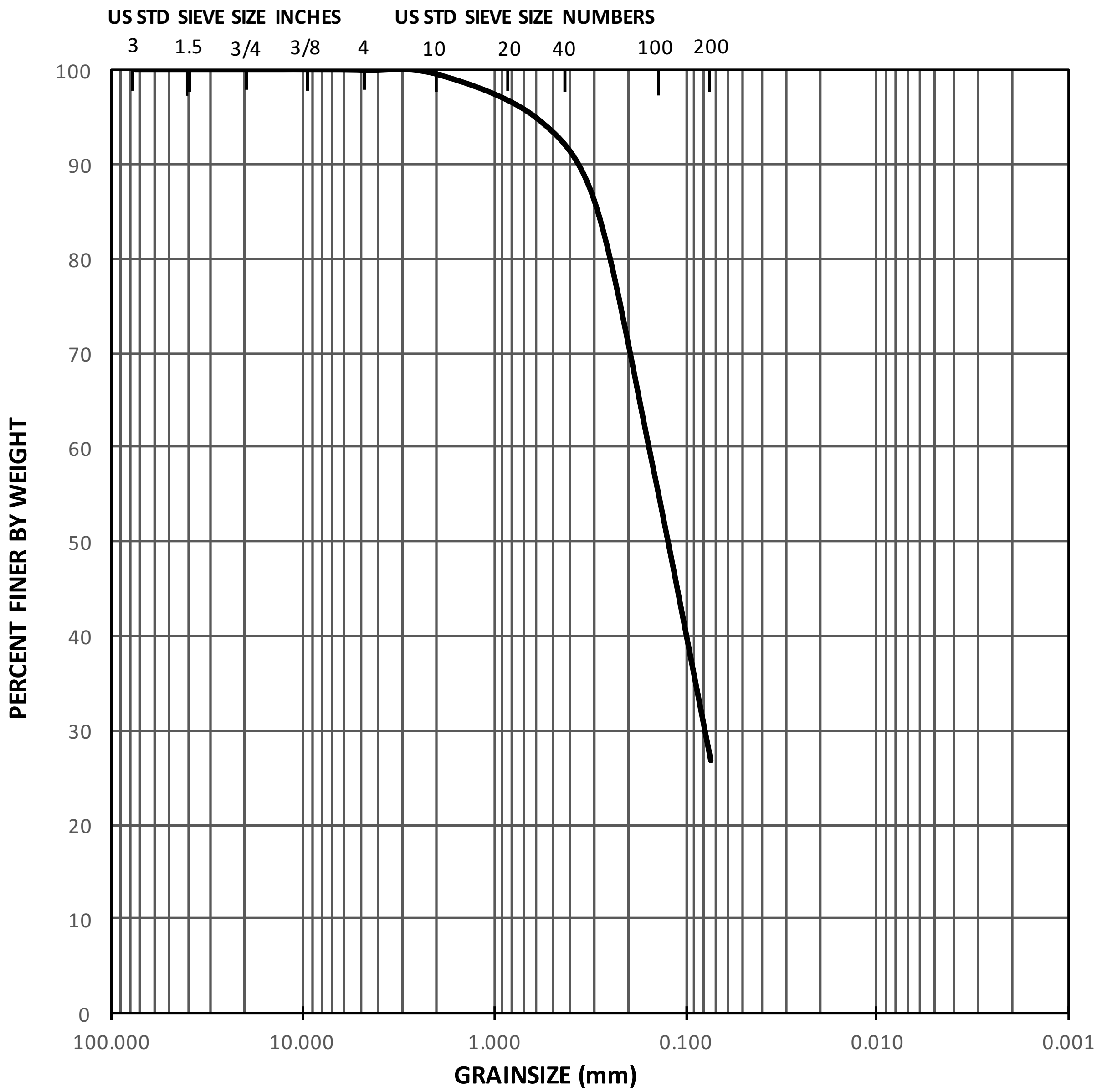
LOCATION	DH-2
DEPTH	13'

CLASSIFICATION
Fine to Medium SAND with Silt (SP-SM)

PASSING NO. 200 (%)
12

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1e



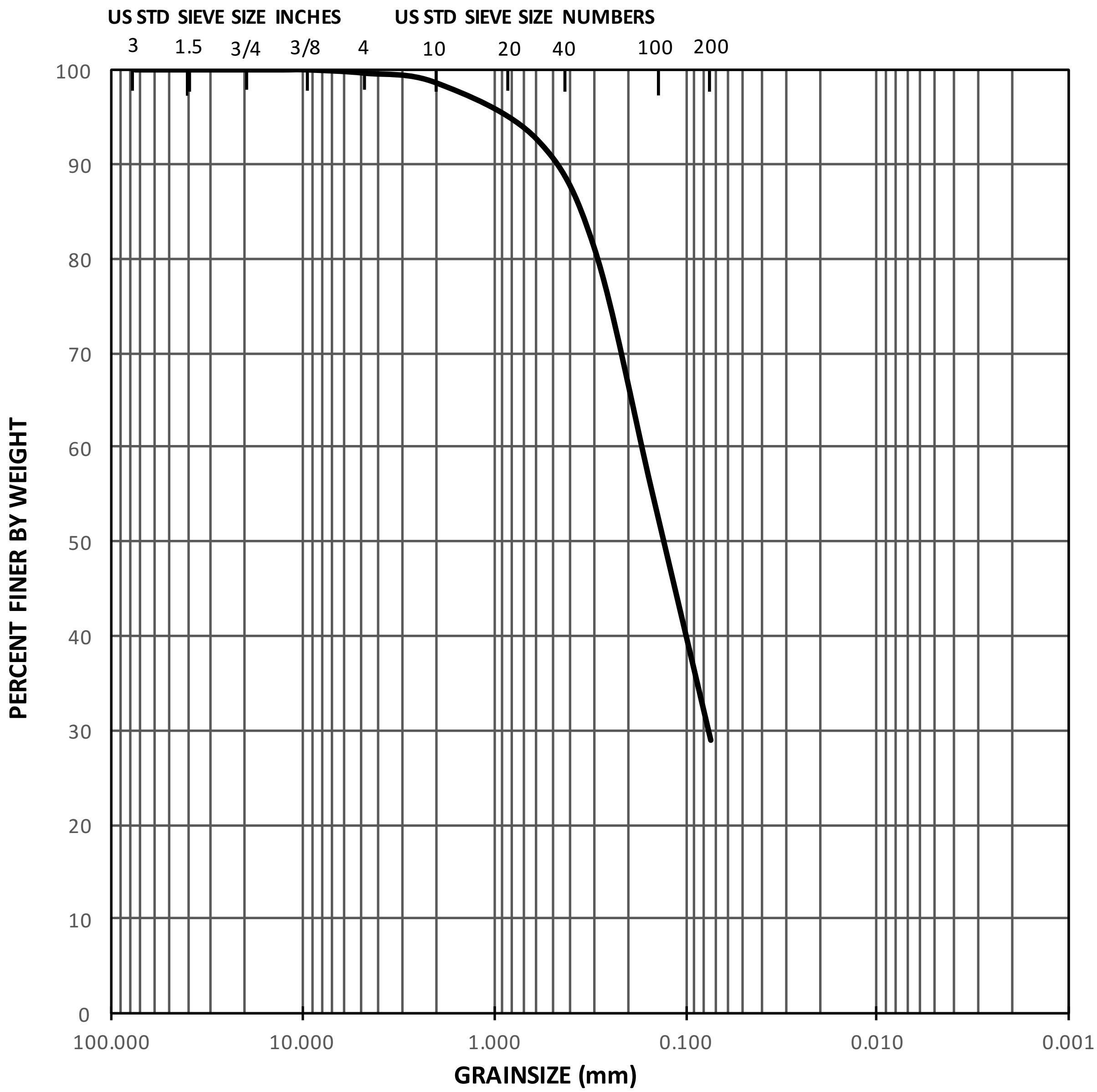
GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LOCATION DH-2
DEPTH 15'

CLASSIFICATION
Silty Fine SAND (SM)

PASSING NO. 200 (%)
27

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

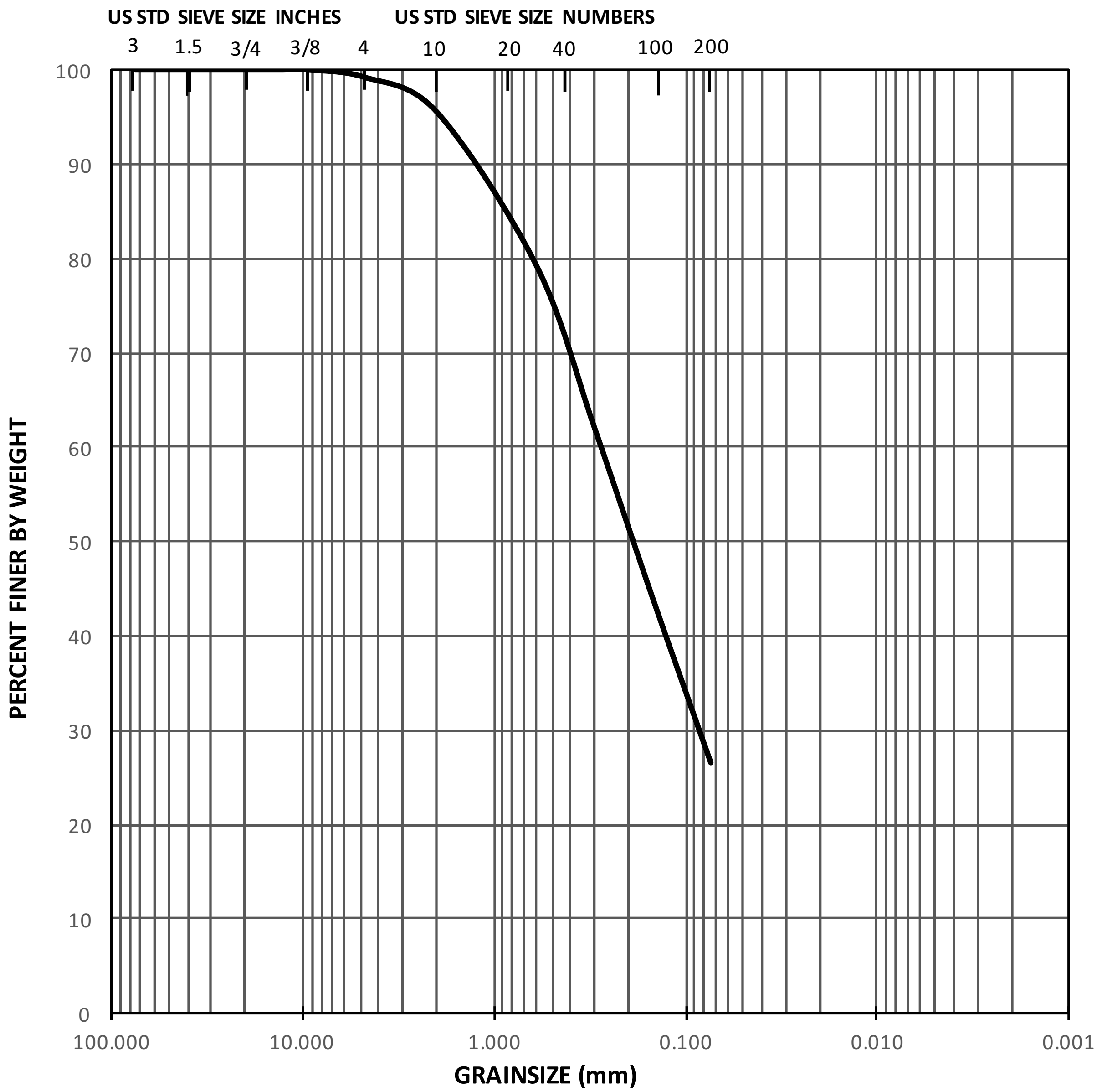
LOCATION DH-2
DEPTH 25'

CLASSIFICATION
Silty Fine SAND (SM)

PASSING NO. 200 (%)
29

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1g



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

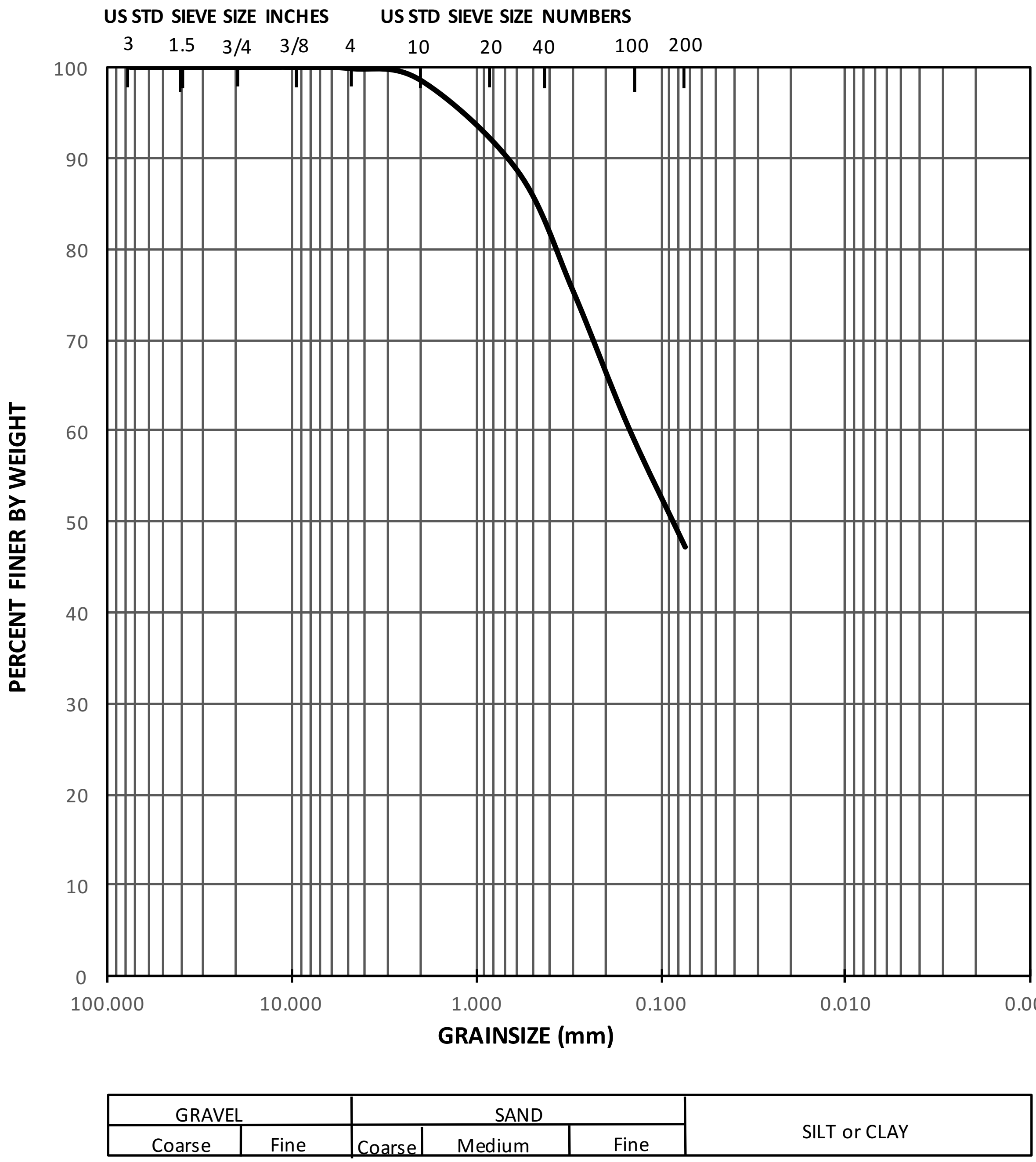
LOCATION	DH-2
DEPTH	34'

CLASSIFICATION
Clayey Fine to Medium SAND (SC)

PASSING NO. 200 (%)
27

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

PLATE B-1h



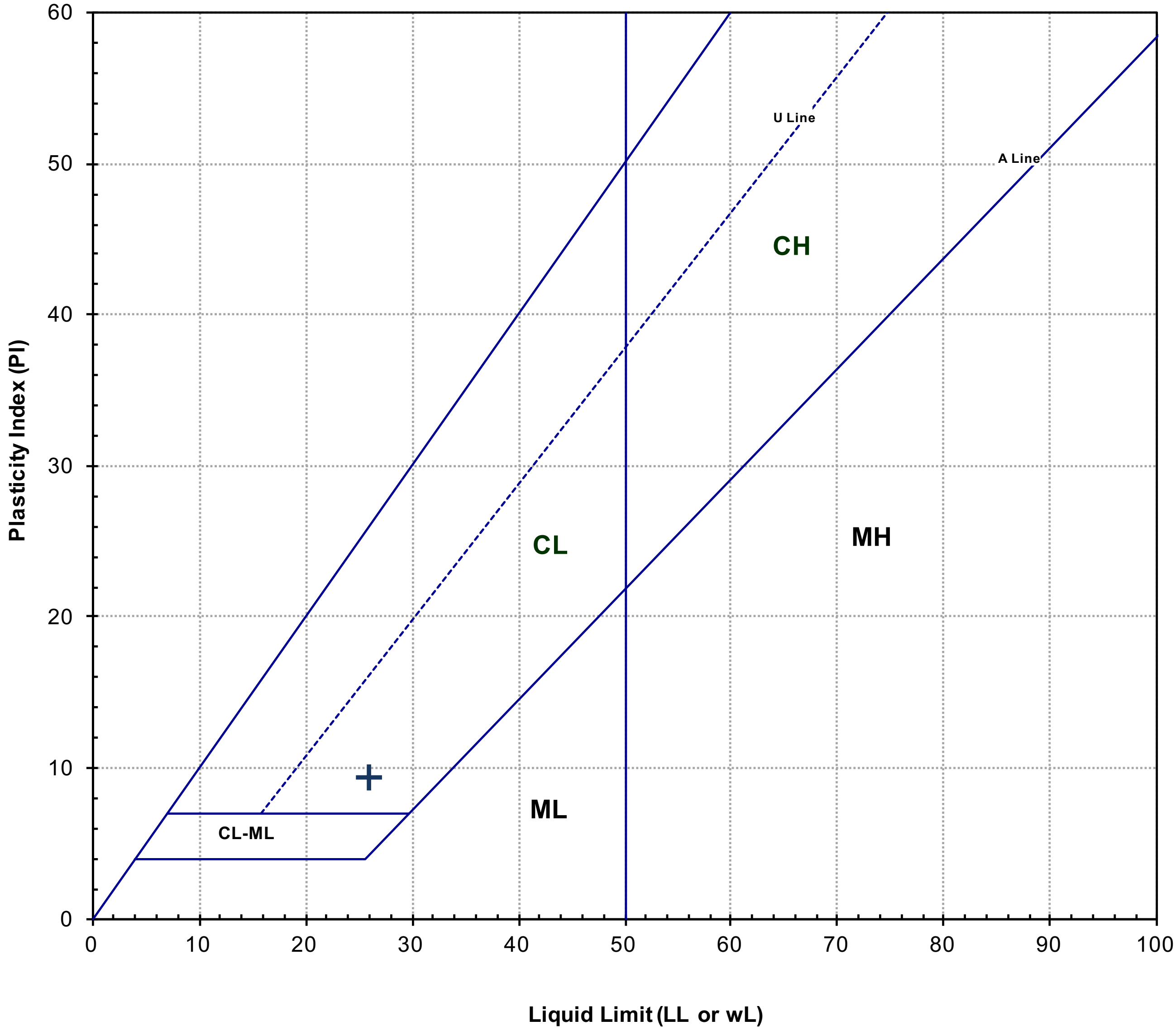
LOCATION	DH-2
DEPTH	42'

CLASSIFICATION
Sandy Silty CLAY (CL-ML)

PASSING NO. 200 (%)
47

GRAINSIZE DISTRIBUTION
Moorpark Library
Moorpark, California

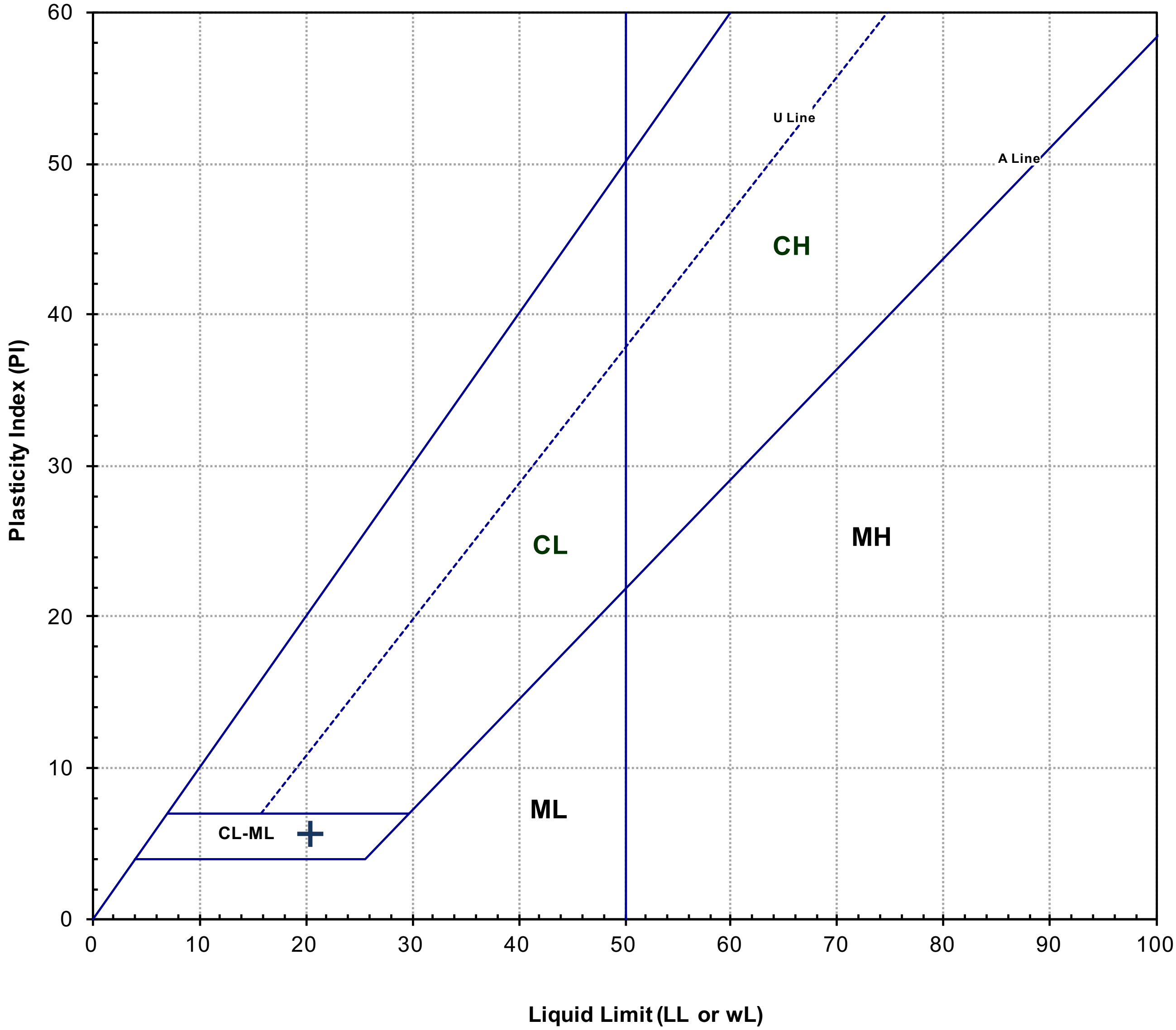
PLATE B-1i



		CLASSIFICATION	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
LOCATION	DH-1				
DEPTH	67'	Clayey SAND (SC)	26	17	9

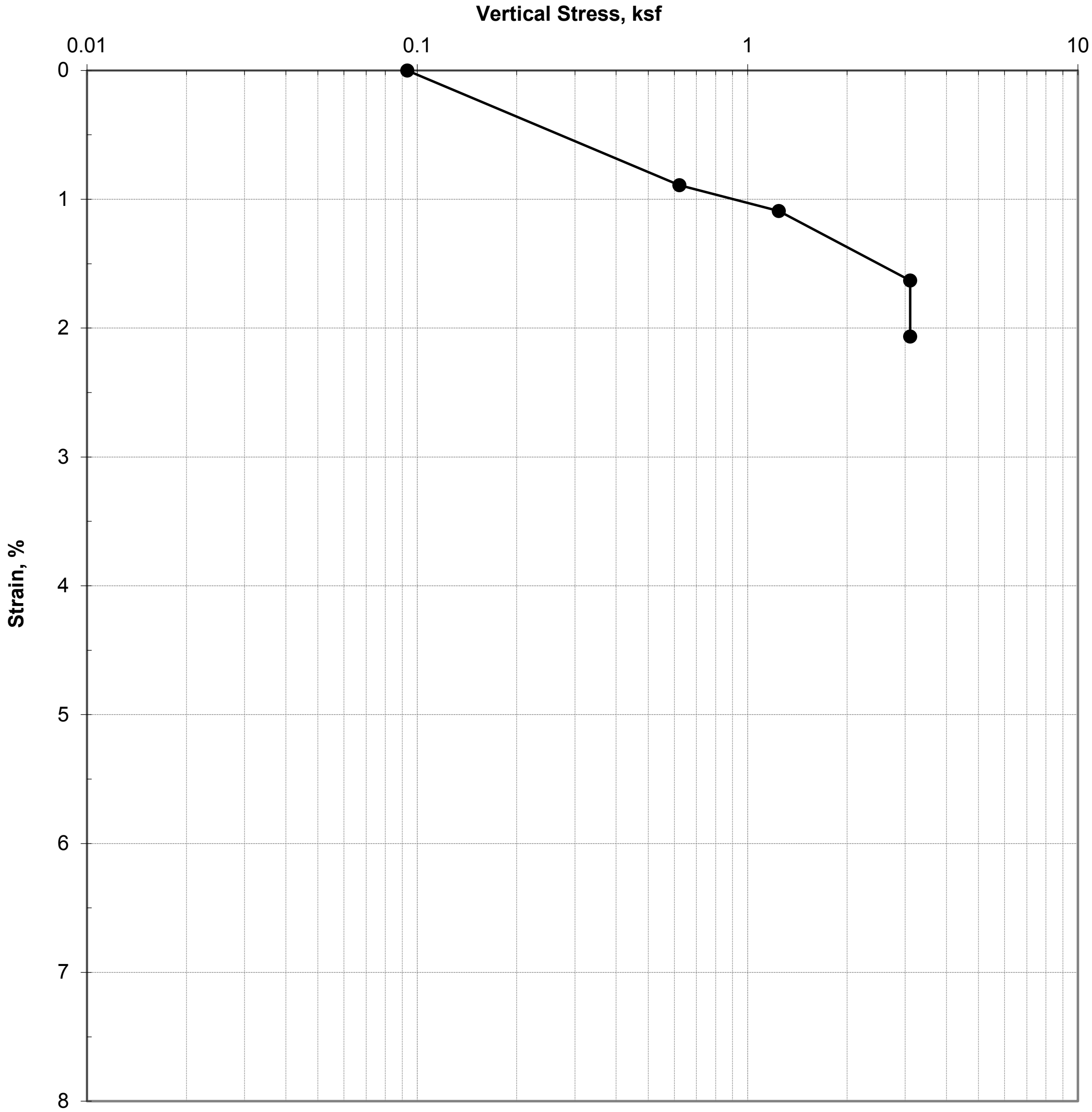
MOORPARK LIBRARY
Moorpark, California

PLATE B-2a



		CLASSIFICATION	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)
LOCATION	DH-2				
DEPTH	40'	Sandy Silty CLAY (CL-ML)	21	15	6

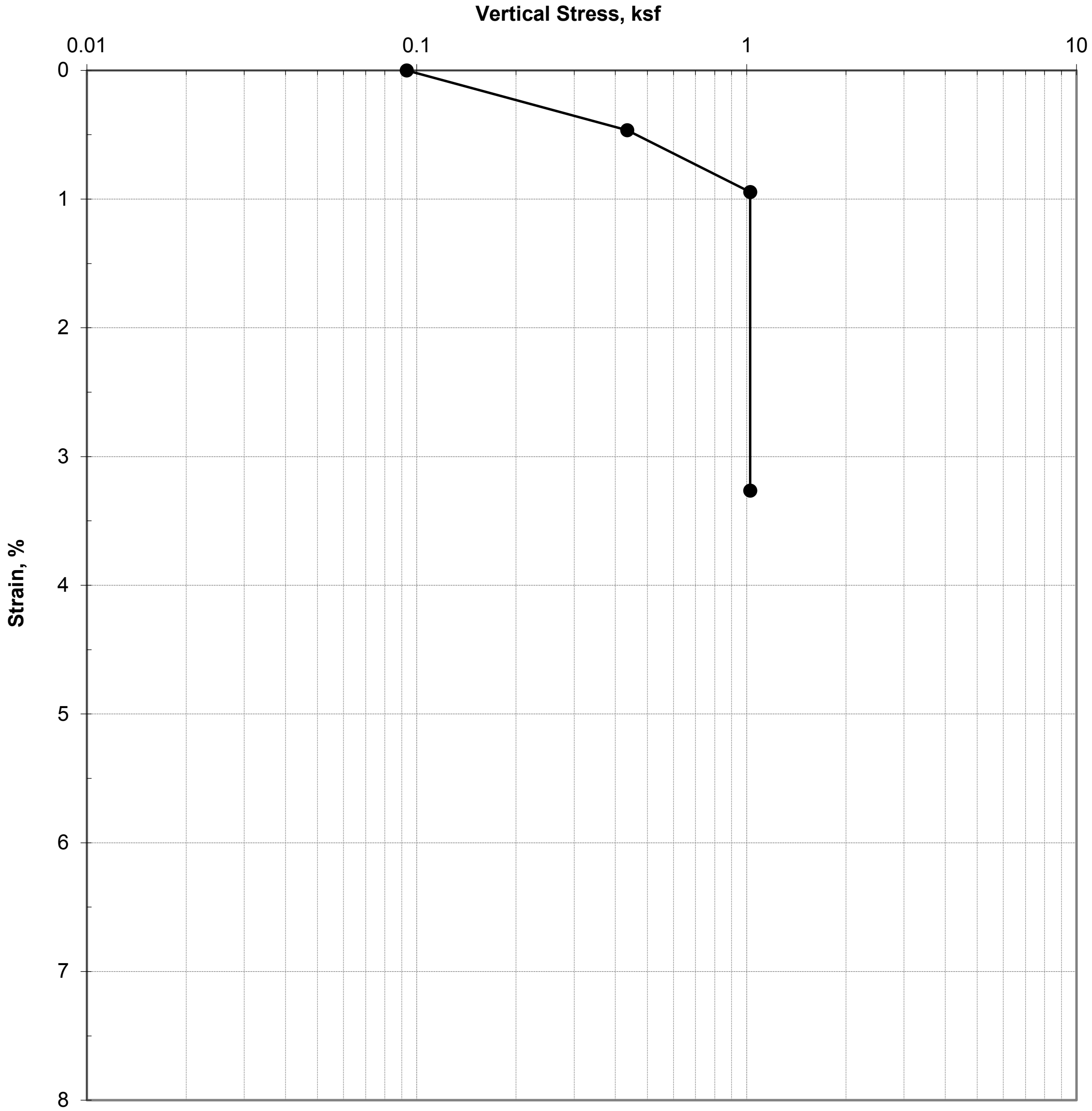
MOORPARK LIBRARY
Moorpark, California



SAMPLE ID	Boring, Sample #, Depth	DH-1 , #10 , 30.0 ft		SUMMARY	Preconsolidation Pressure, ksf	---
	USCS Classification:	Poorly-graded SAND (SP): yellow, dry			Inundation Increment, ksf	3.11
PROPERTIES		Initial	Final		Liquid Limit	---
					Plastic Limit	---
					Plasticity Index	---
	Water Content, %	2.5%	18.6%		Passing #200	---
	Dry Unit Weight, pcf	102.0	104.1		Estimated Gs	2.65
	Saturation, %	11%	84%	REMARKS	Test Method: ASTM D4546, Method B	
	Void Ratio	0.62	0.59		030.003 - Moorpark Library	
	Diameter, in	2.42	2.42		After adding water the specimen	
	Height, in	1.00	0.98		collapsed 0.43% at a stress of 3.11ksf.	

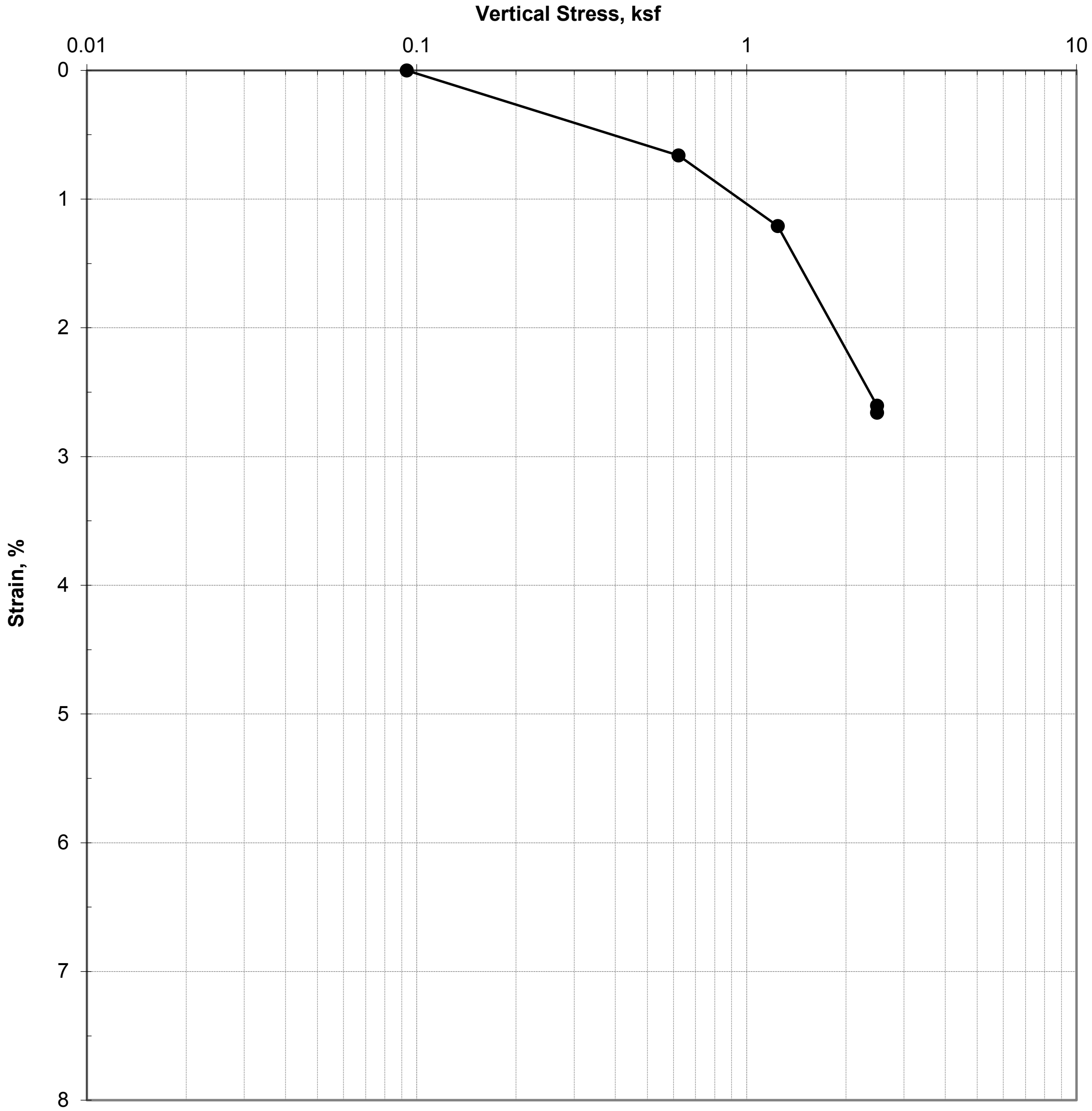
ONE DIMENSIONAL COLLAPSE TEST

PLATE B-3a



SAMPLE ID	Boring, Sample #, Depth	DH-2 , #5 , 10.0 ft		SUMMARY	Preconsolidation Pressure, ksf	---
	USCS Classification:	Poorly-graded SAND with silt (SP-SM): light brown, dry, lightly cemented			Inundation Increment, ksf	1.03
PROPERTIES		Initial	Final		Liquid Limit	---
	Water Content, %	3.5%	21.0%		Plastic Limit	---
	Dry Unit Weight, pcf	96.9	100.1		Plasticity Index	---
	Saturation, %	13%	85%		Passing #200	---
	Void Ratio	0.71	0.65		Estimated Gs	2.65
	Diameter, in	2.42	2.42	REMARKS	Test Method: ASTM D4546, Method B 030.003 - Moorpark Library	
	Height, in	1.00	0.97		After adding water the specimen collapsed 2.32% at a stress of 1.03ksf.	

ONE DIMENSIONAL COLLAPSE TEST



SAMPLE ID	Boring, Sample #, Depth	DH-2 , #9 , 25.0 ft		
	USCS Classification:	Poorly-graded SAND (SP): yellow brown, moist, fine		
PROPERTIES		Initial	Final	
	Water Content, %	5.6%	24.6%	
	Dry Unit Weight, pcf	89.9	92.4	
	Saturation, %	18%	82%	
	Void Ratio	0.84	0.79	
	Diameter, in	2.42	2.42	
	Height, in	1.00	0.97	

SUMMARY	Preconsolidation Pressure, ksf	---
	Inundation Increment, ksf	2.49
	Liquid Limit	---
	Plastic Limit	---
	Plasticity Index	---
	Passing #200	---
	Estimated Gs	2.65
REMARKS	Test Method: ASTM D4546, Method B 030.003 - Moorpark Library After adding water the specimen collapsed 0.05% at a stress of 2.49ksf.	

ONE DIMENSIONAL COLLAPSE TEST

R-value Test Report (Caltrans 301)

Job No.:	903-017	**Date:**	05/22/17	**Initial Moisture,**	6.6
Client:	Oakridge Geoscience	**Tested**	PJ	**R-value**	**70**
Project:	Moorpark Library - 030.003	**Reduced**	RU		
Sample	DH-1 @ 0-5'	**Checked**	DC	**Expansion Pressure**	**0 psf**
Soil Type:	Brown SAND w/ Silt				

Specimen Number	A	B	C	D
Exudation Pressure, psi	200	360	478	
Prepaired Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	60	50	45	
Weight of Soil & Mold, grams	3137	3143	3132	
Weight of Mold, grams	2083	2090	2089	
Height After Compaction, in.	2.50	2.50	2.42	
Moisture Content, %	11.9	11.0	10.6	
Dry Density, pcf	114.2	115.0	118.2	
Expansion Pressure, psf	0	0	0	
Stabilometer @ 1000				
Stabilometer @ 2000	34	26	22	
Turns Displacement	4.90	4.95	5.15	
R-value	65	72	74	

Remarks:

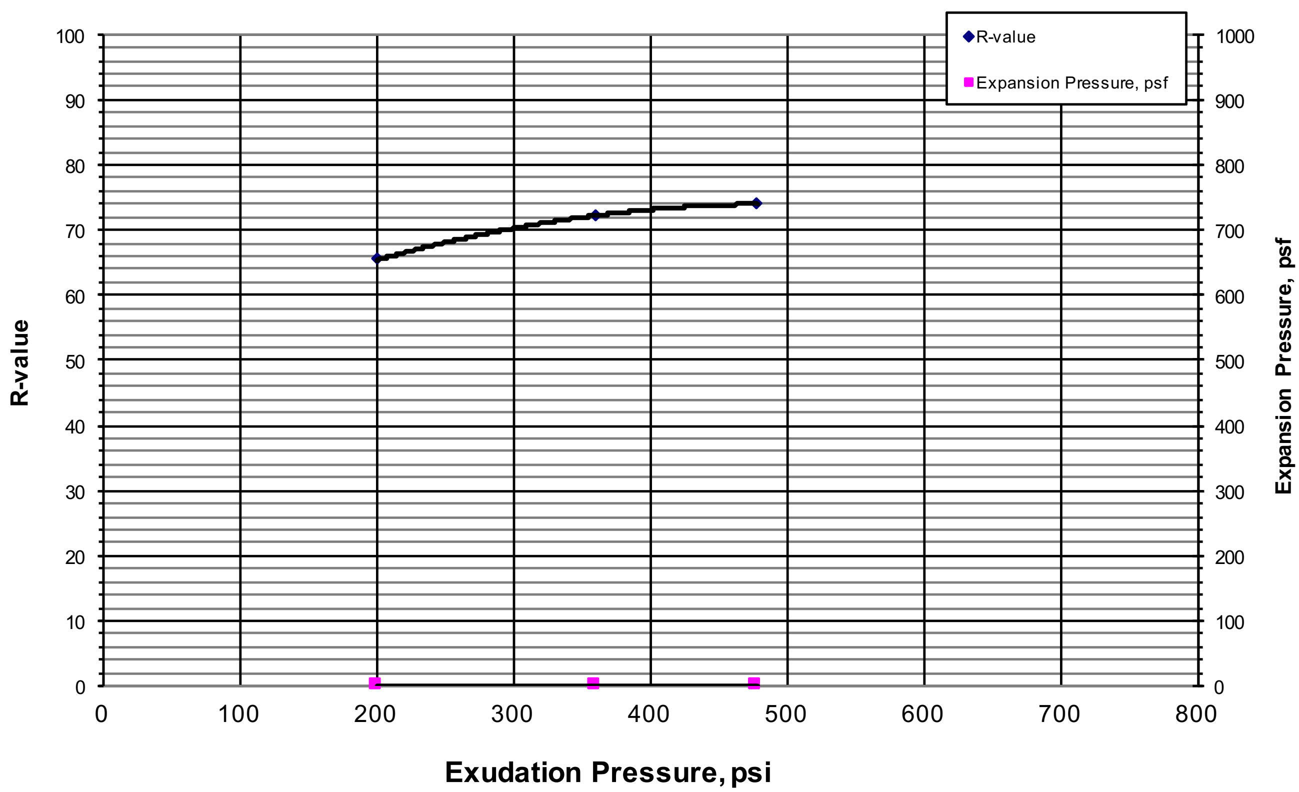


PLATE B-4

Corrosivity Tests Summary

CTL # 903-017 **Date:** **Tested By:** PJ **Checked:** PJ
Client: Oakridge Geoscience **Project:** Moorpark Library **Proj. No:** 030.003
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
DH-1	1	0-5	-	-	16,319	2	6	0.0006	7.5	-	-	-	3.3	Brown SAND w/ Silt

PLATE B-5

APPENDIX C

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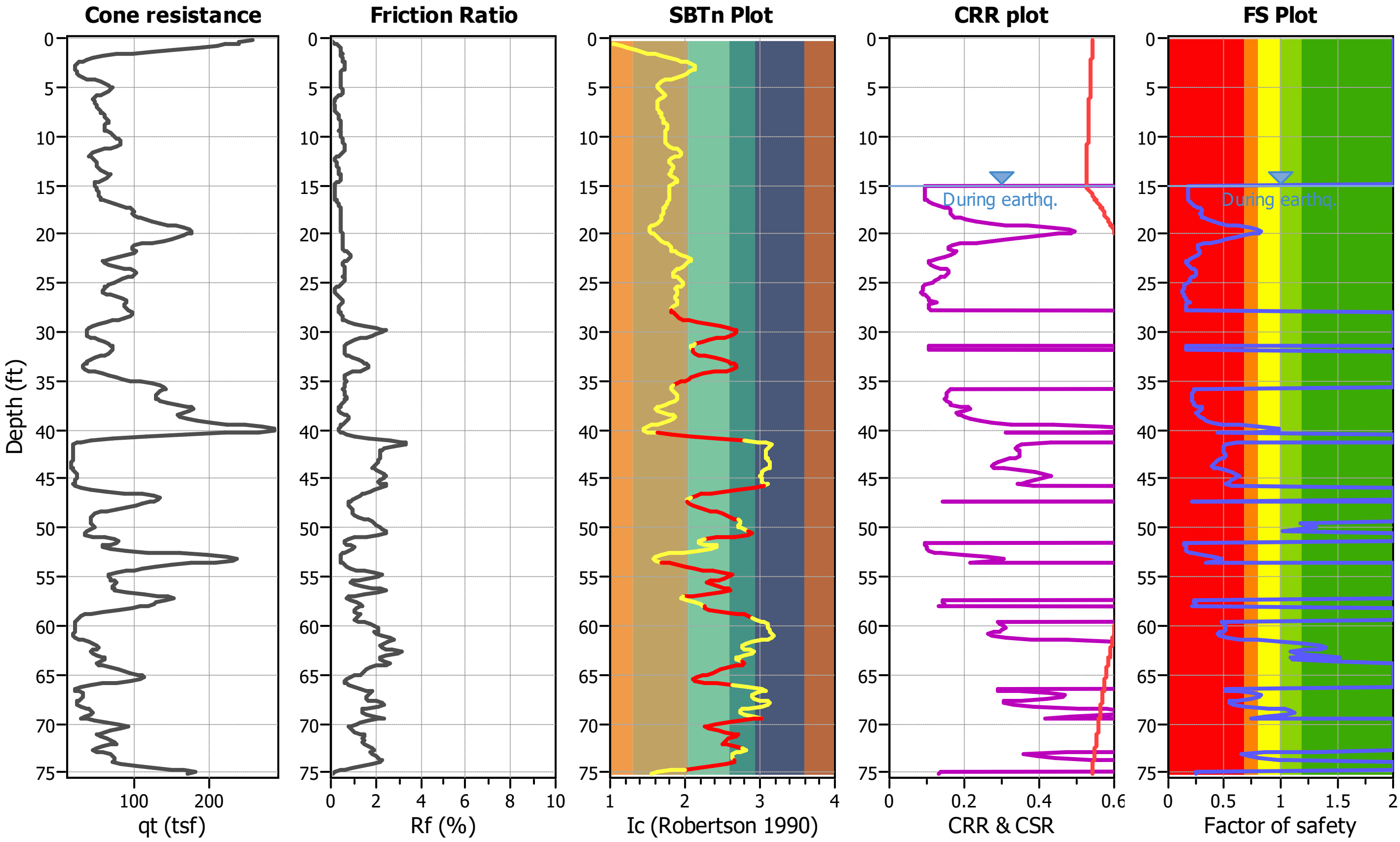
LIQUEFACTION ANALYSIS REPORT

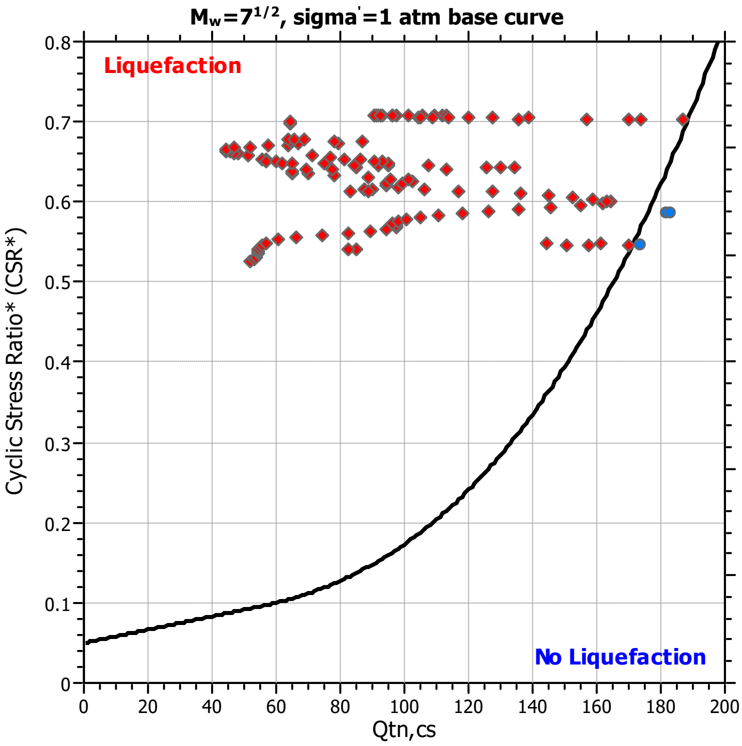
Project title : Moorpark Library **Location : High Street and Moorpark Avenue, Moorpark, Caliornia**

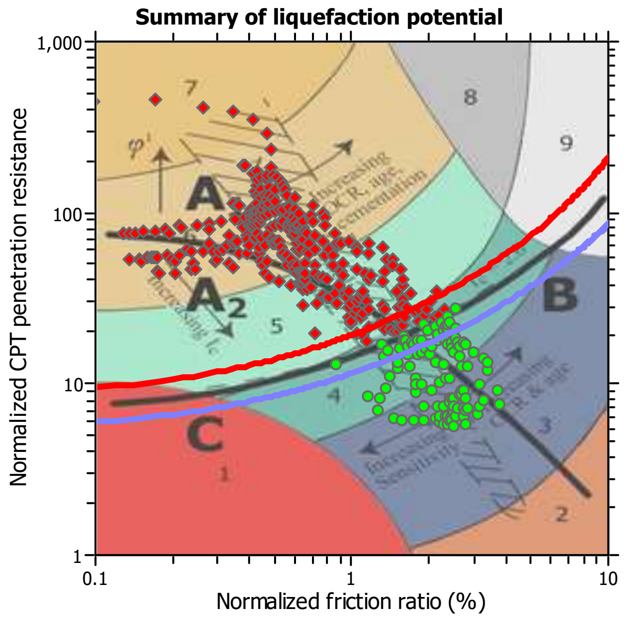
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	37.50 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	No		

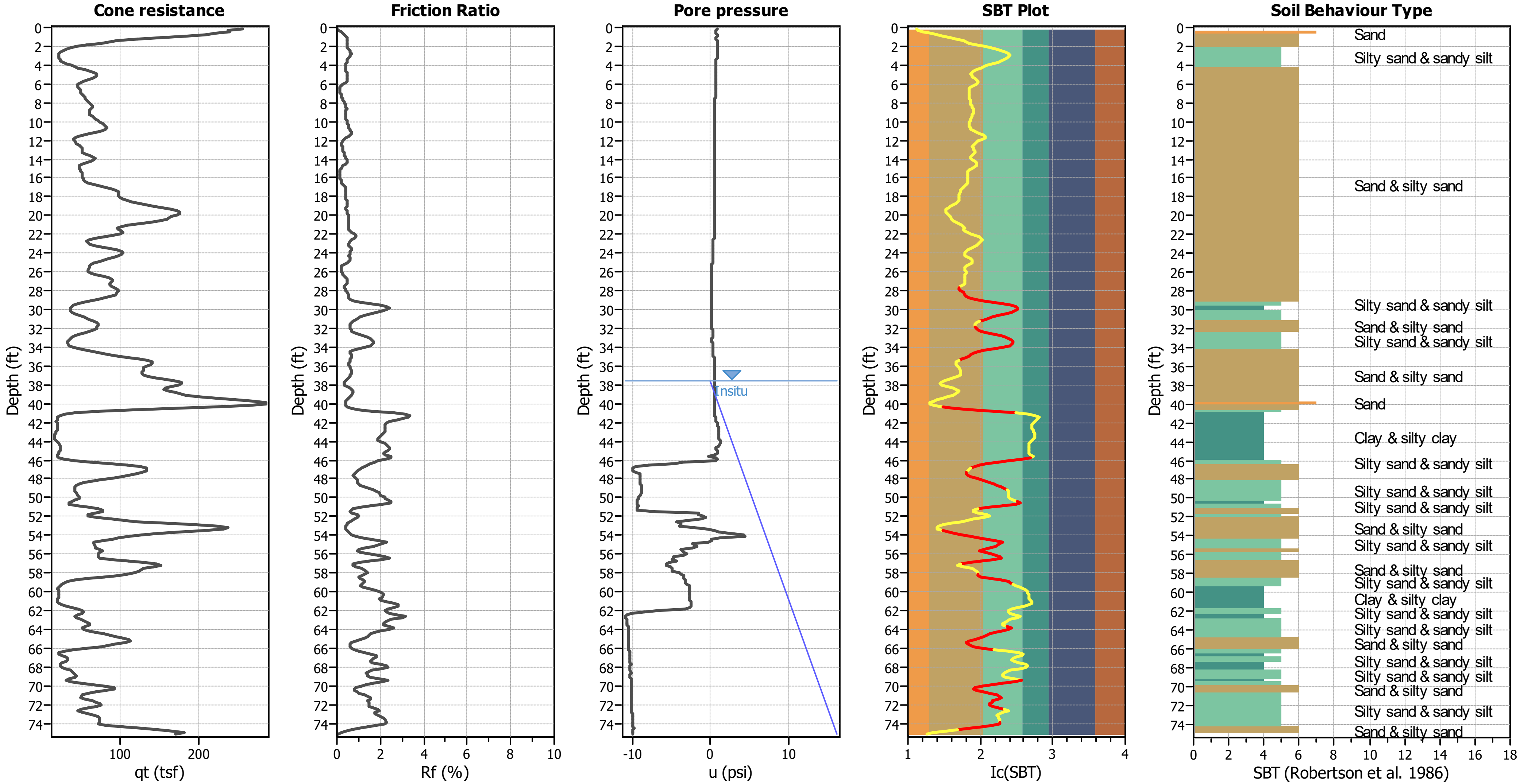






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



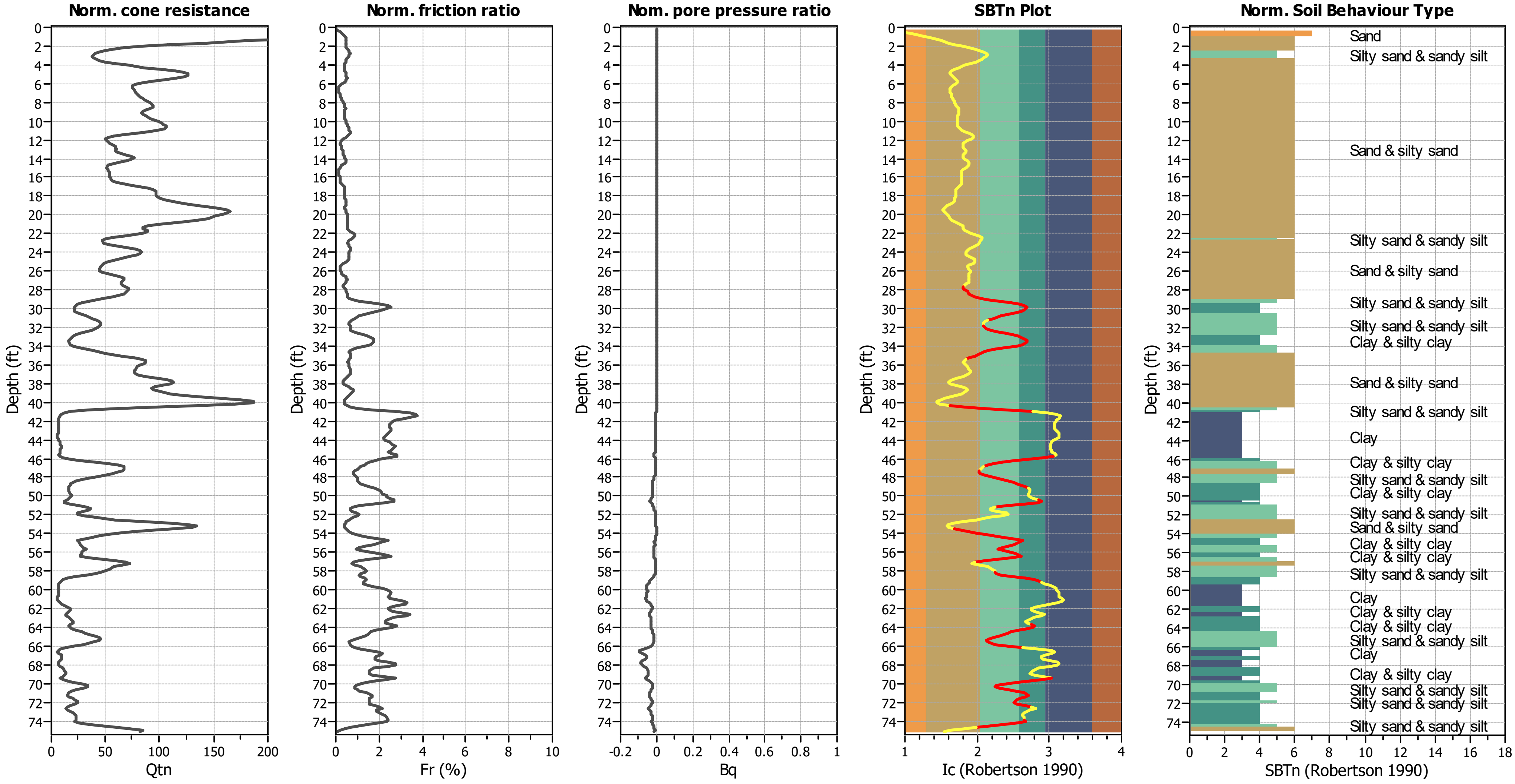
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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)



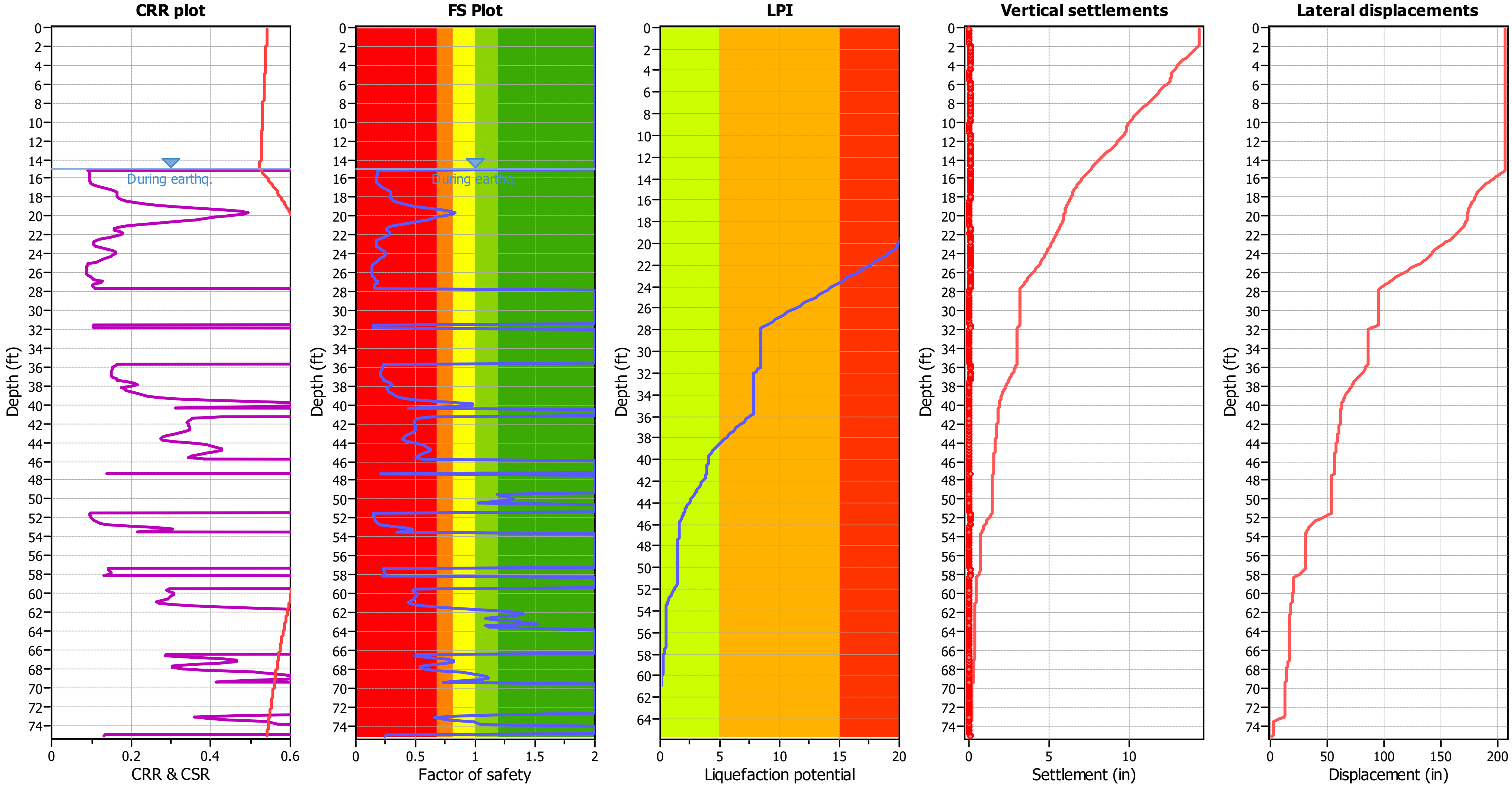
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 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.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

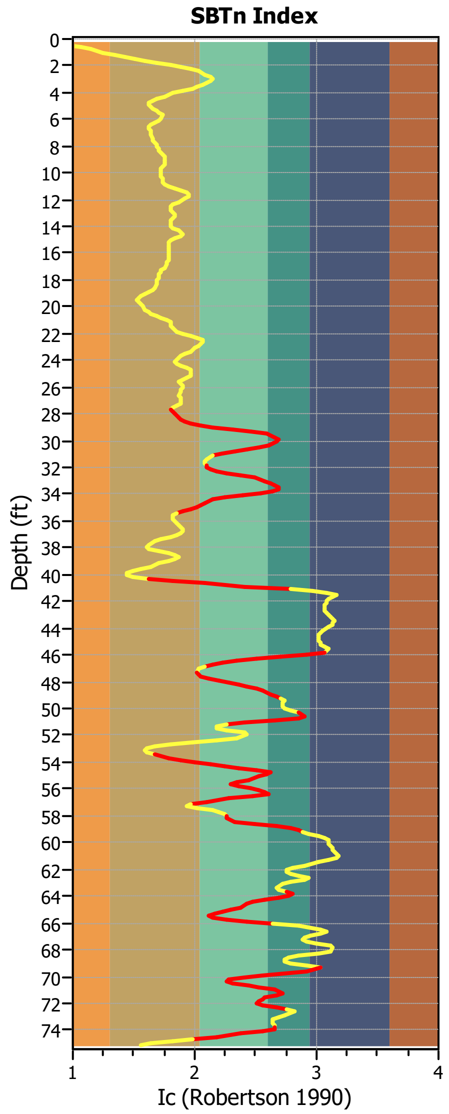
- Very high risk
- High risk
- Low risk

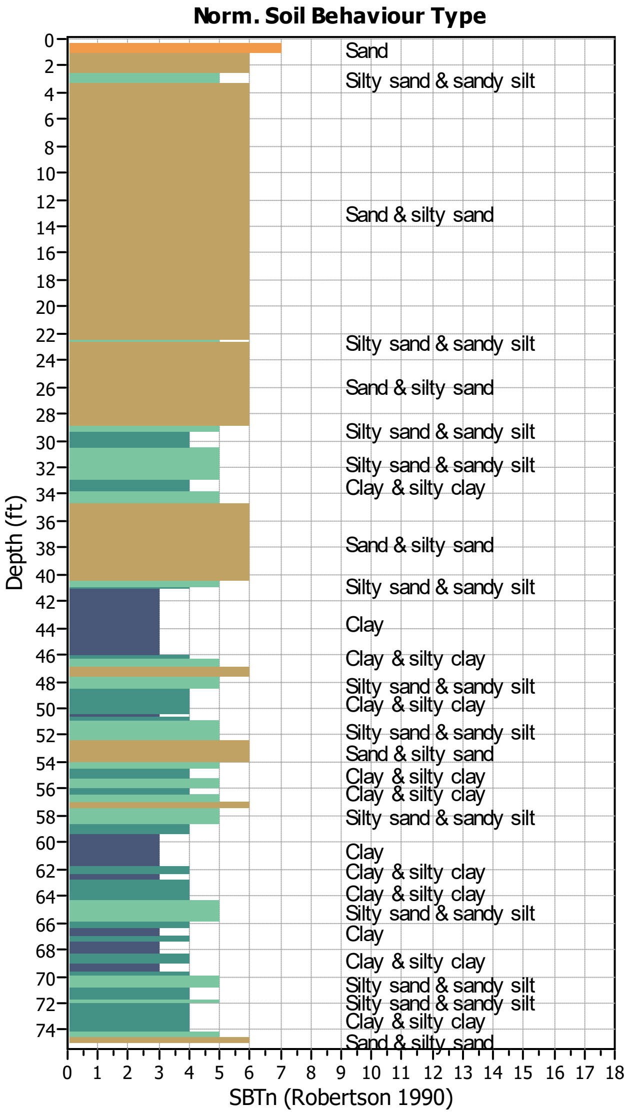
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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.





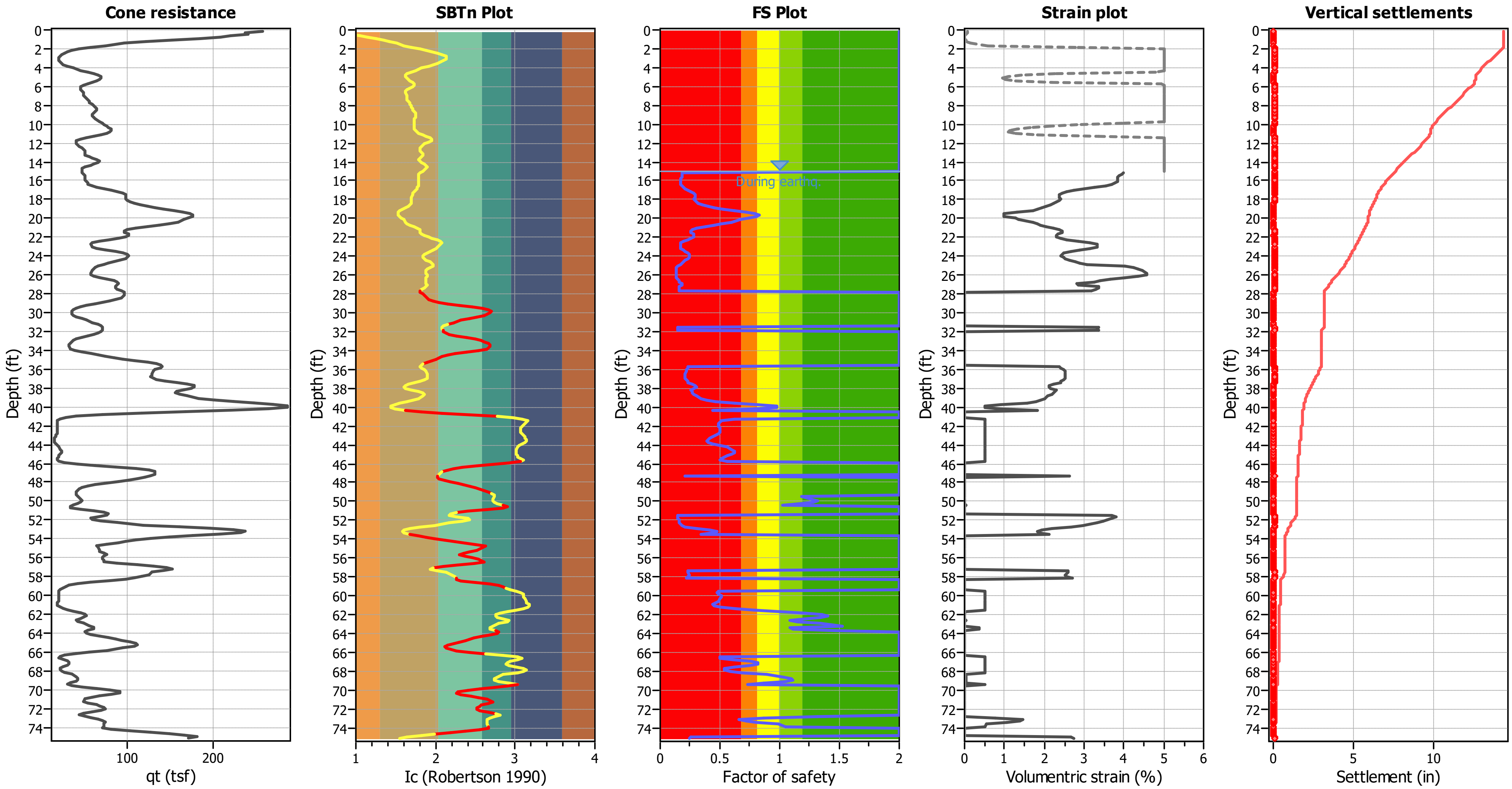
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0100
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	458
Total points excluded:	156
Exclusion percentage:	34.06%
Number of layers detected:	20

Estimation of post-earthquake settlements

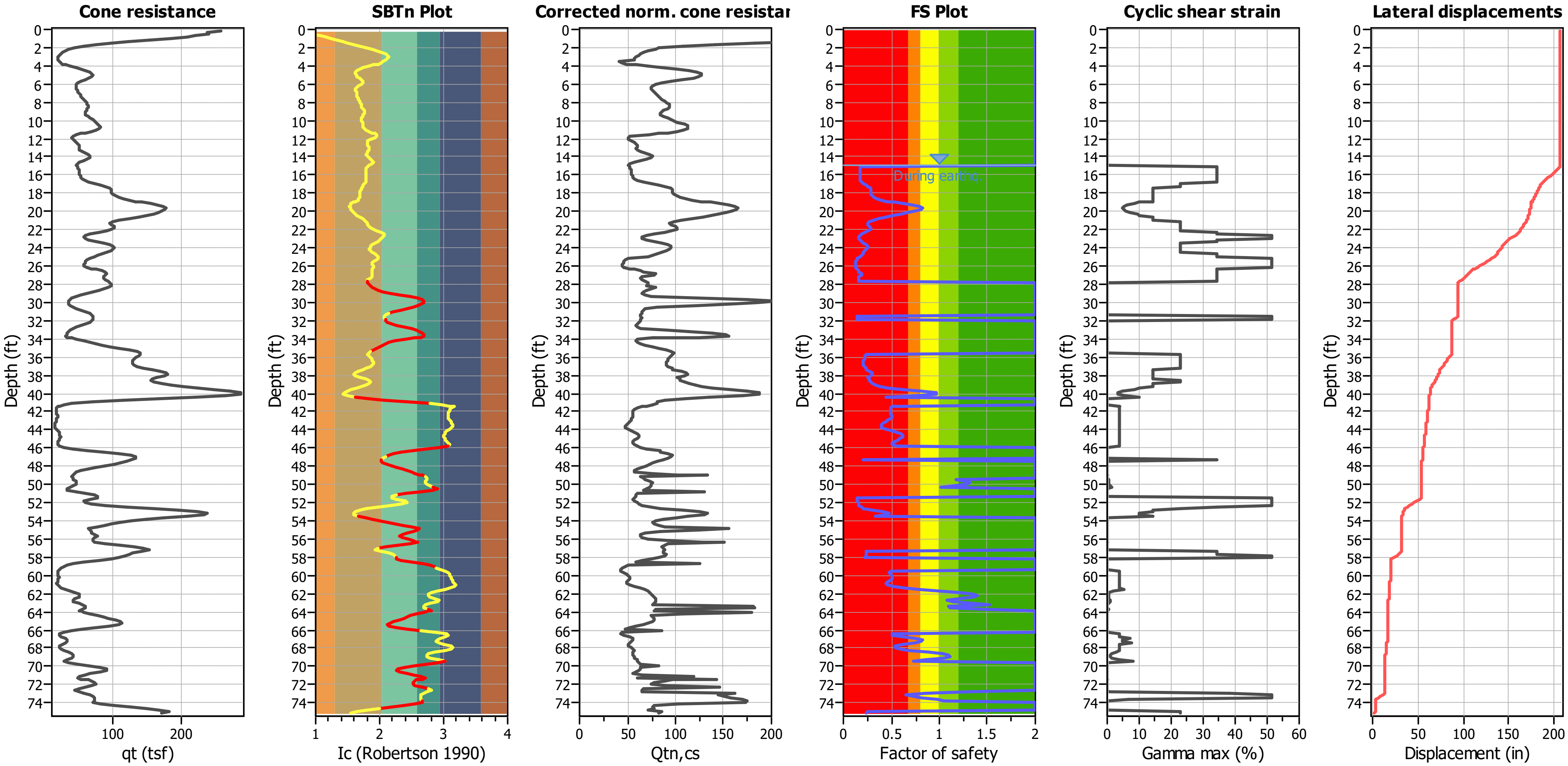


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 2.50 %)

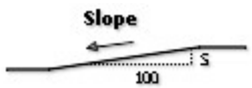


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



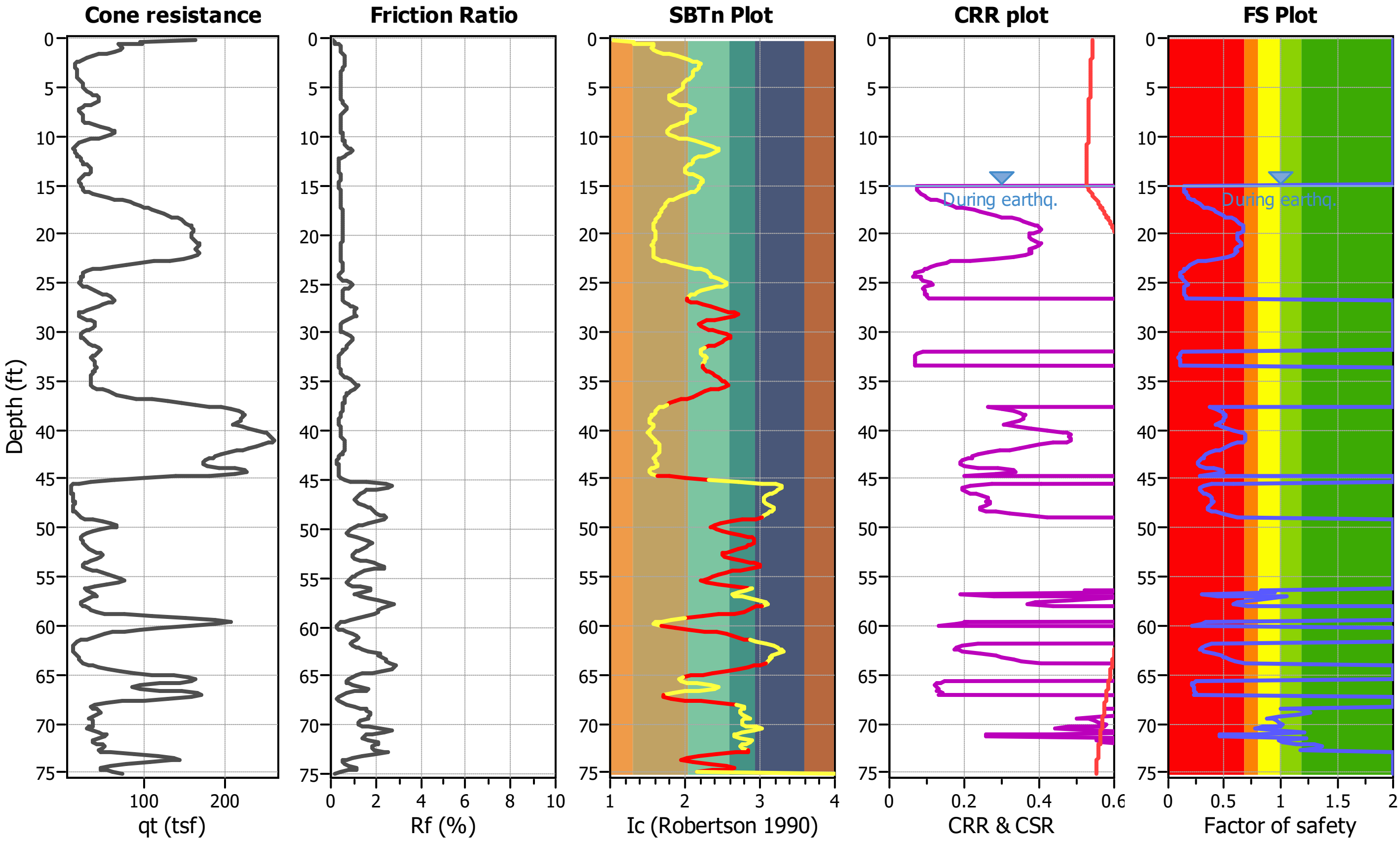
LIQUEFACTION ANALYSIS REPORT

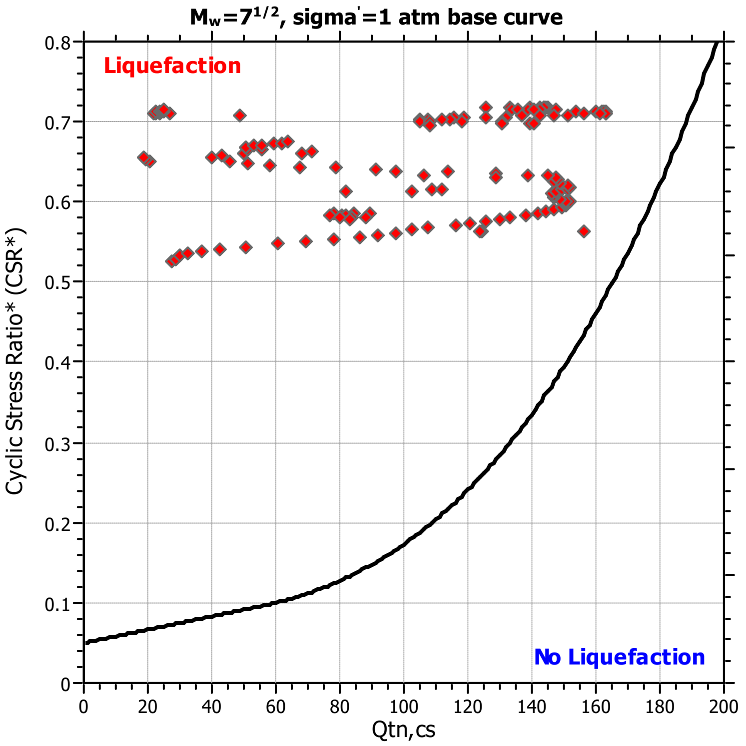
Project title : Moorpark Library **Location : High Street and Moorpark Avenue, Moorpark, Calionia**

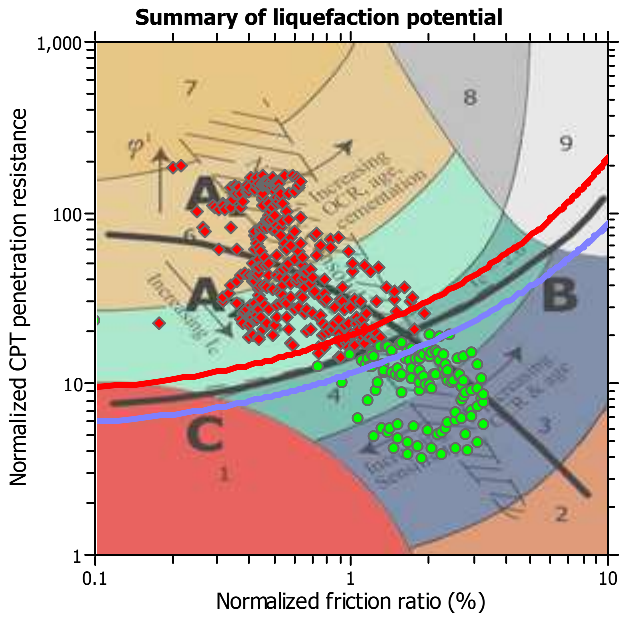
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	37.50 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	No		

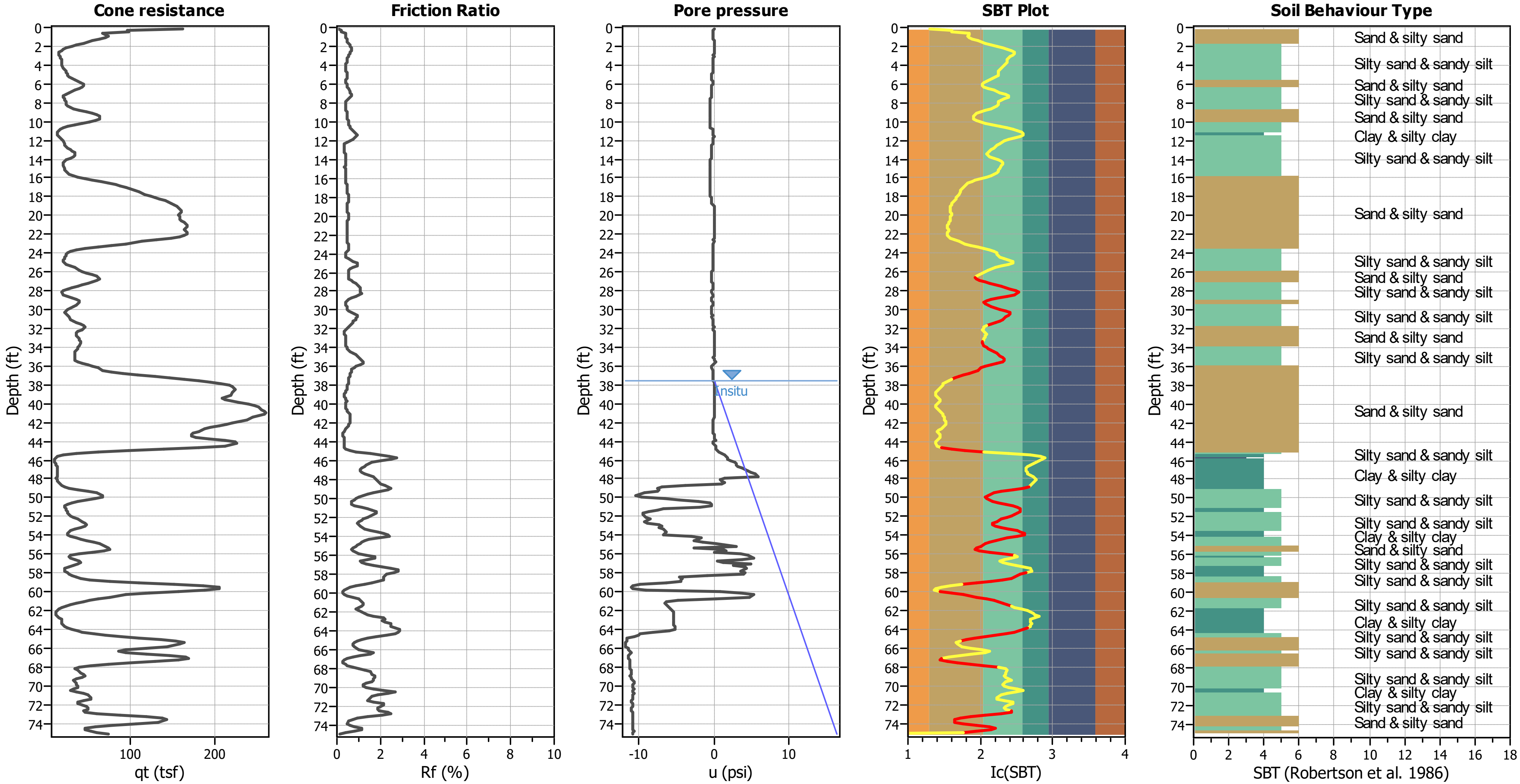






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



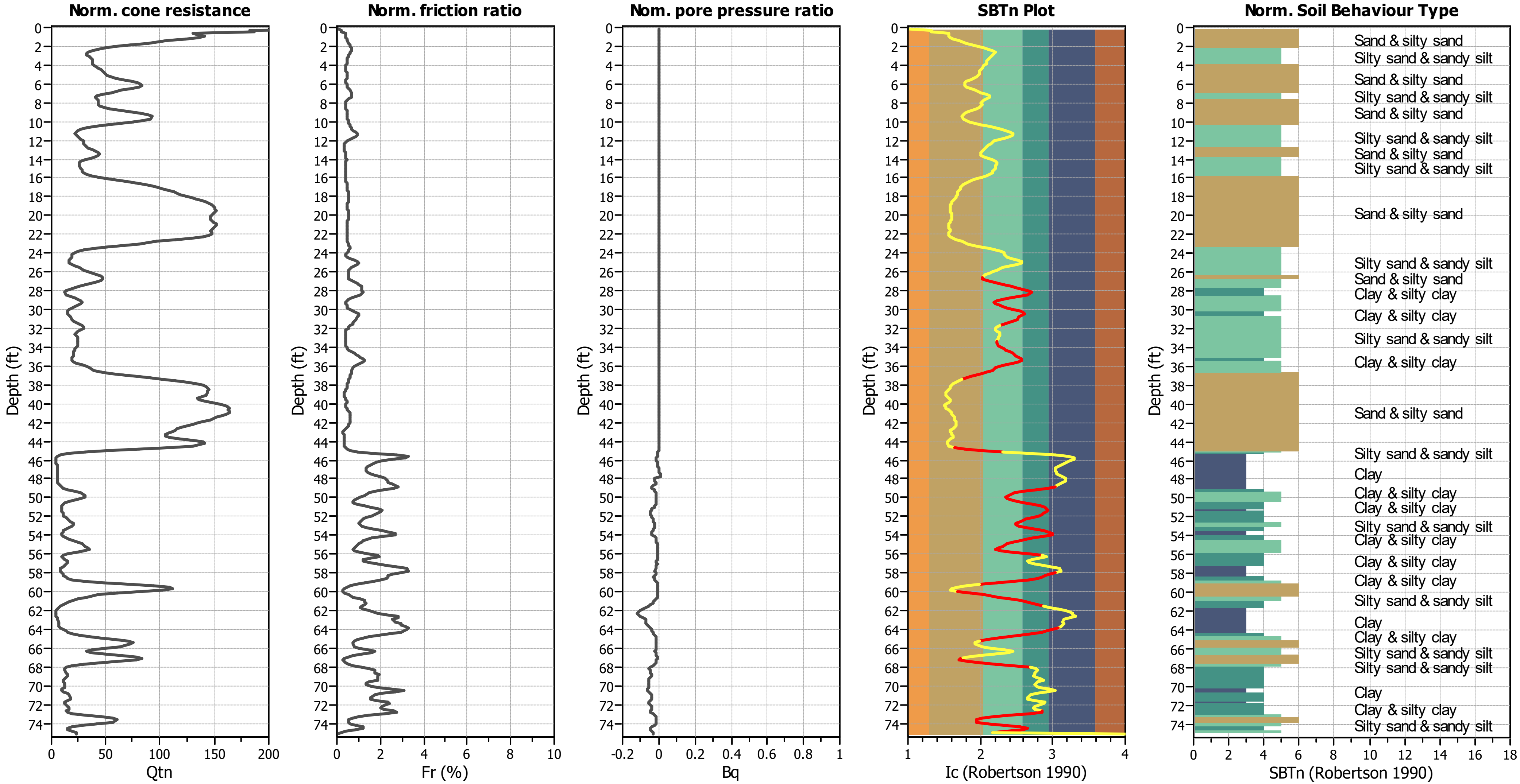
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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)



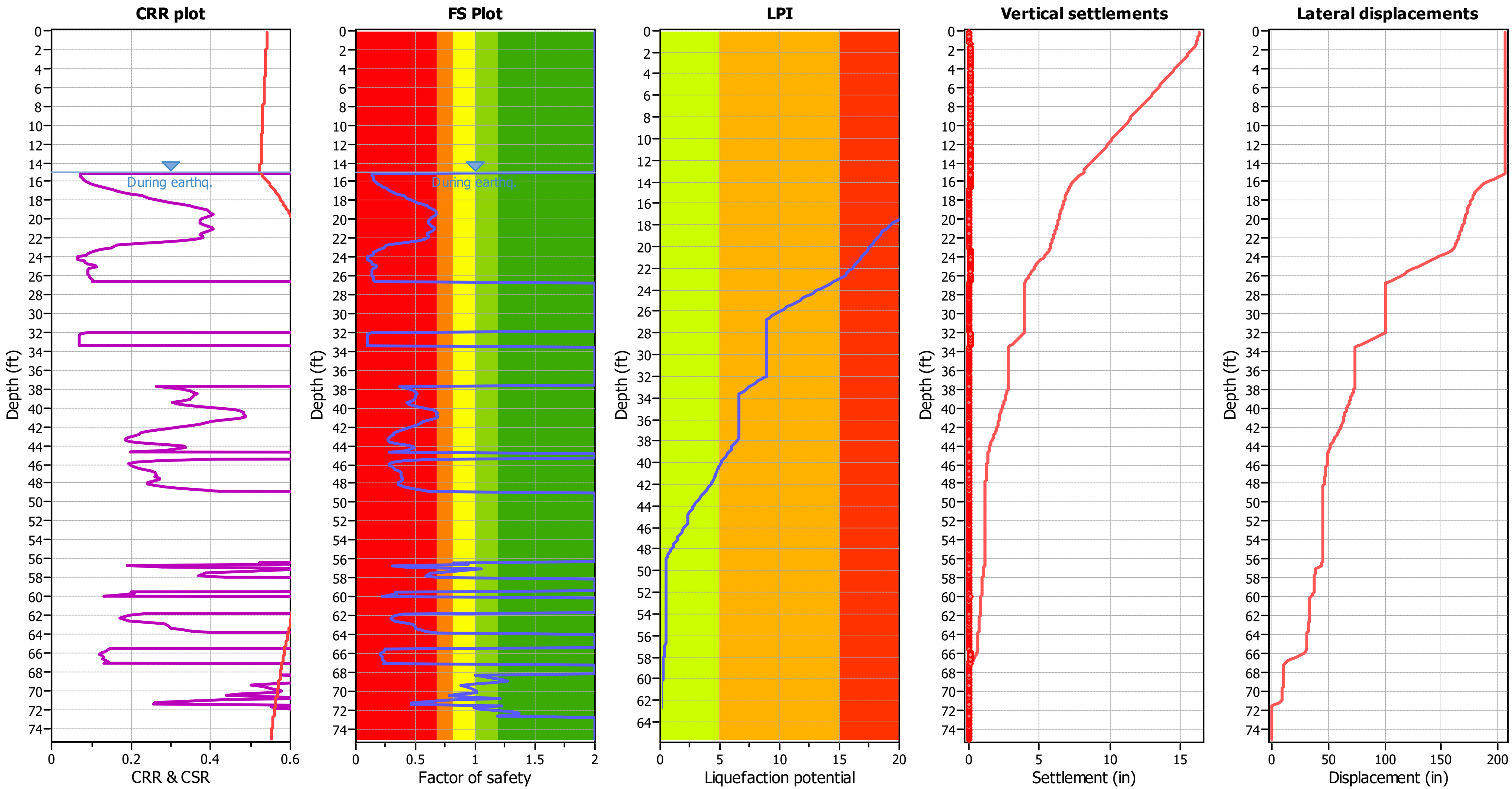
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

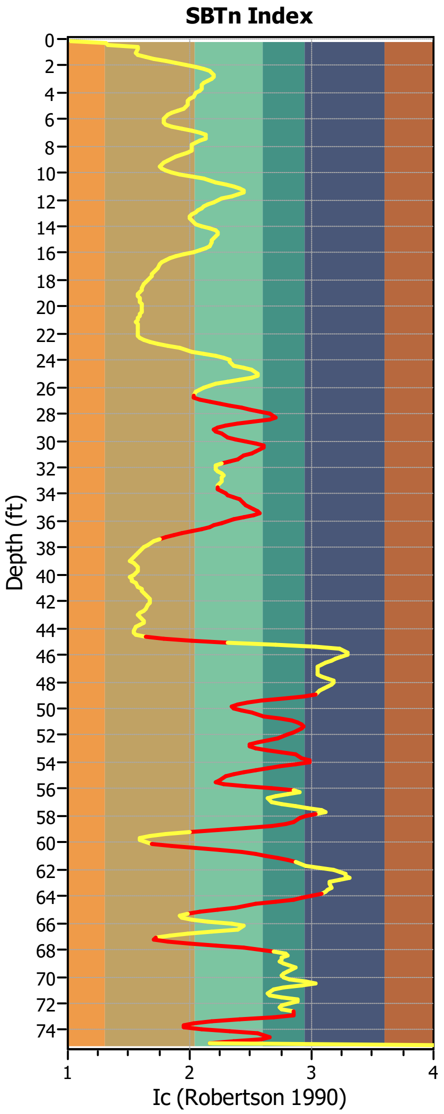
- Very high risk
- High risk
- Low risk

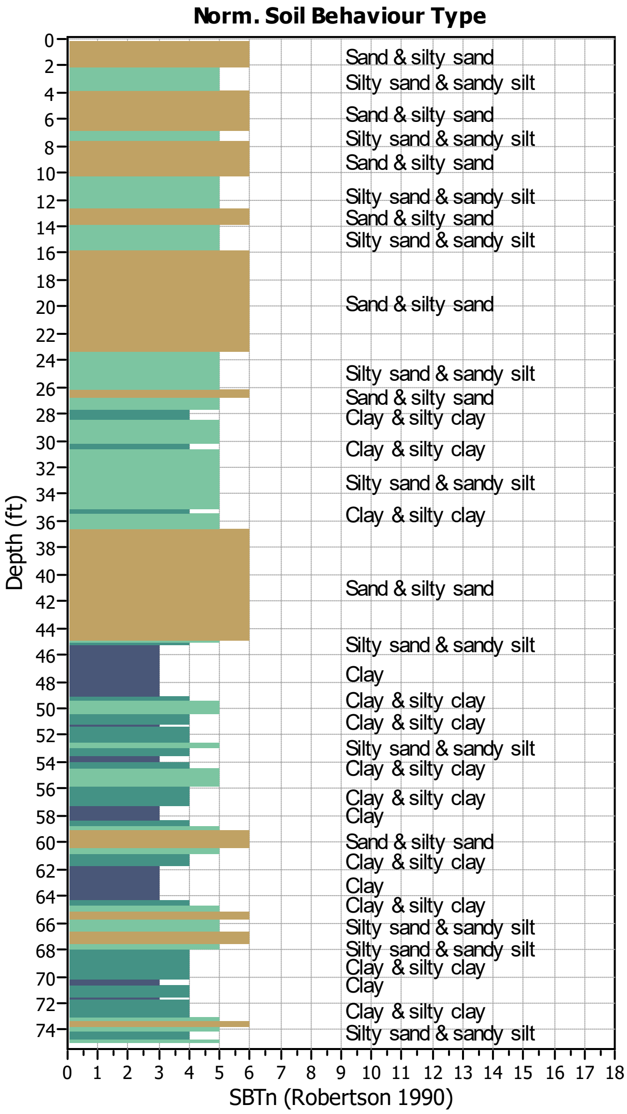
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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.





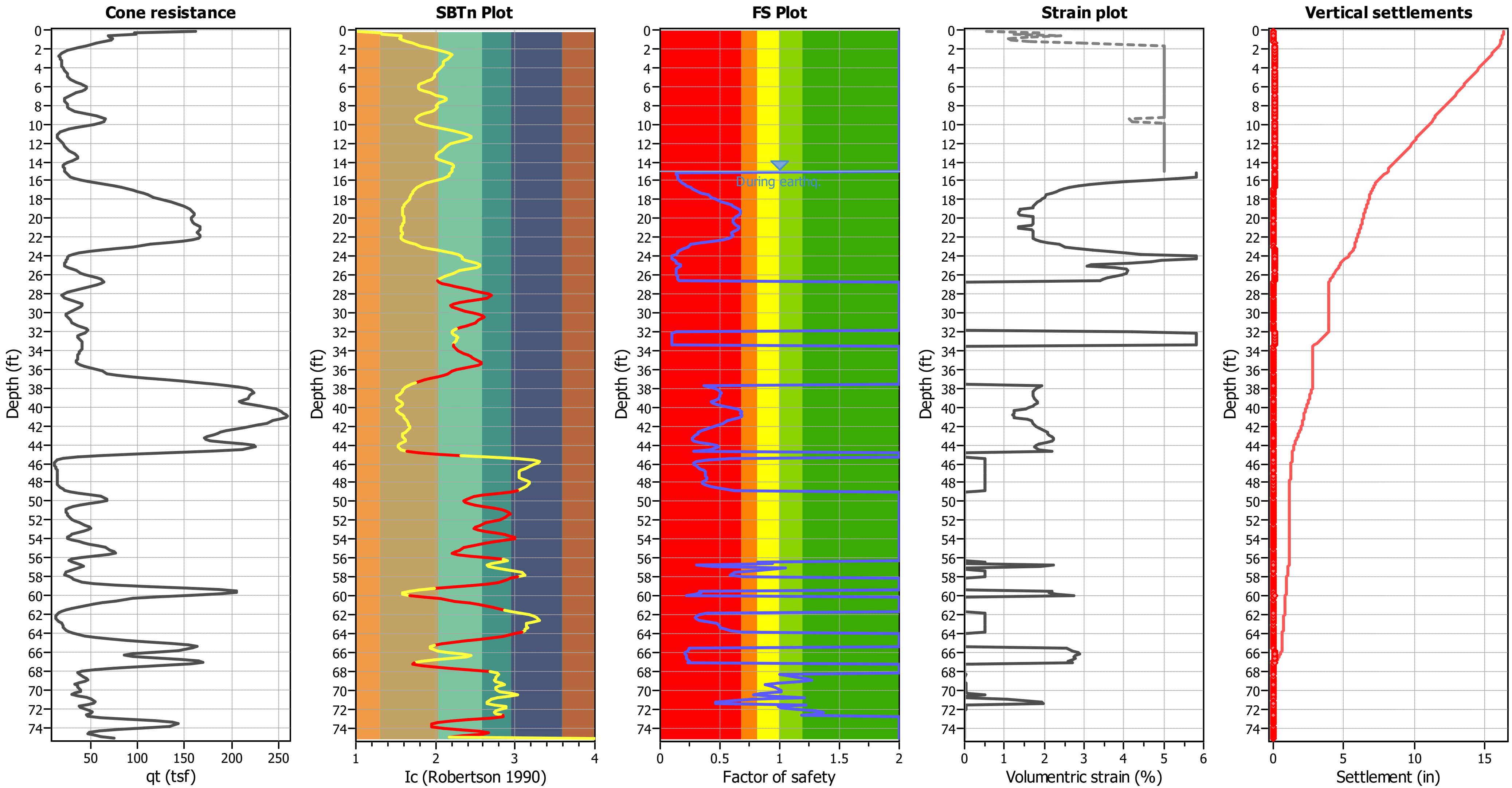
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0100
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	458
Total points excluded:	160
Exclusion percentage:	34.93%
Number of layers detected:	20

Estimation of post-earthquake settlements

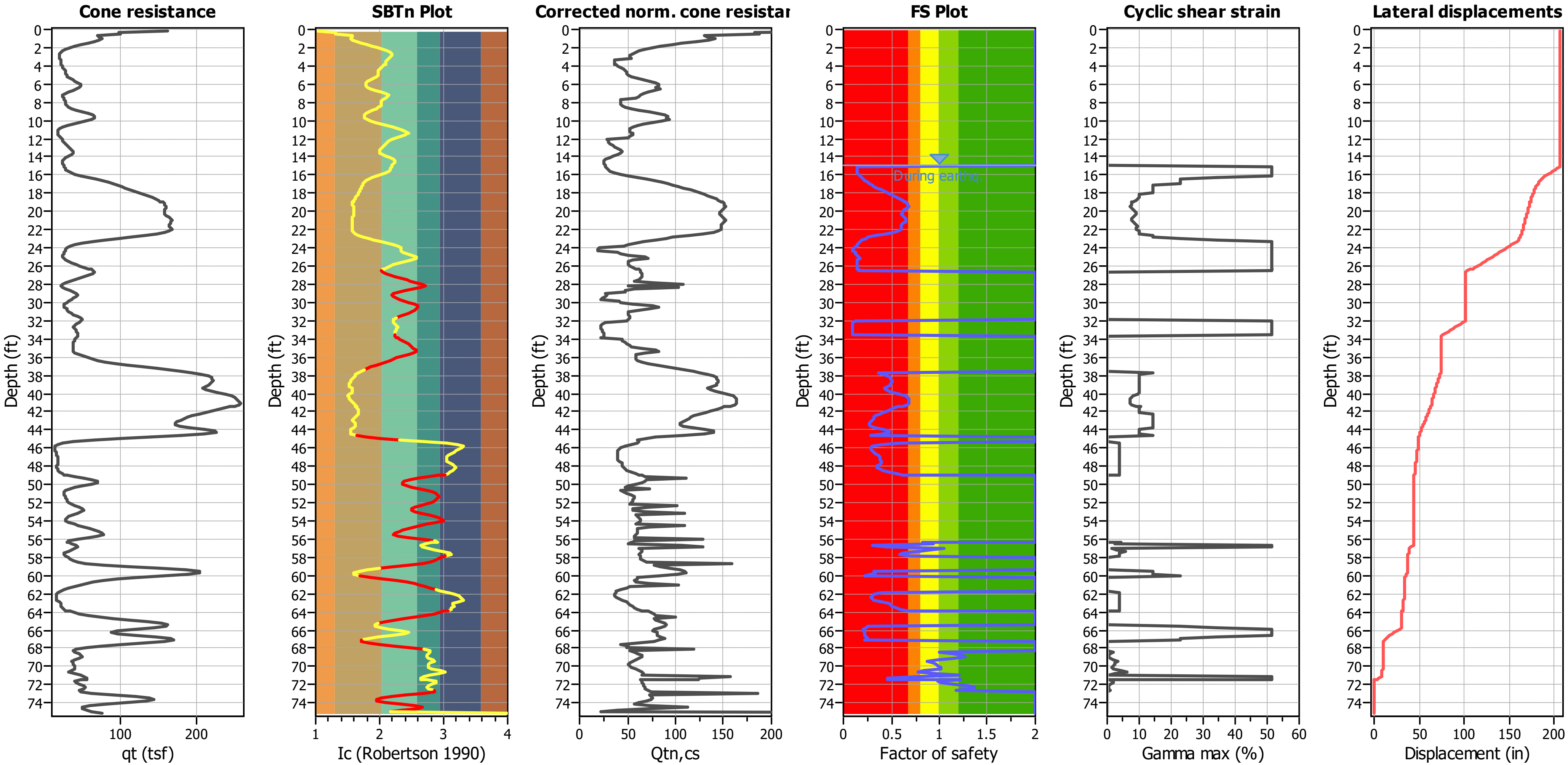


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 2.50 %)

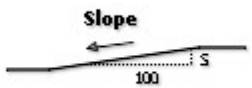


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



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http://www.geologismiki.gr

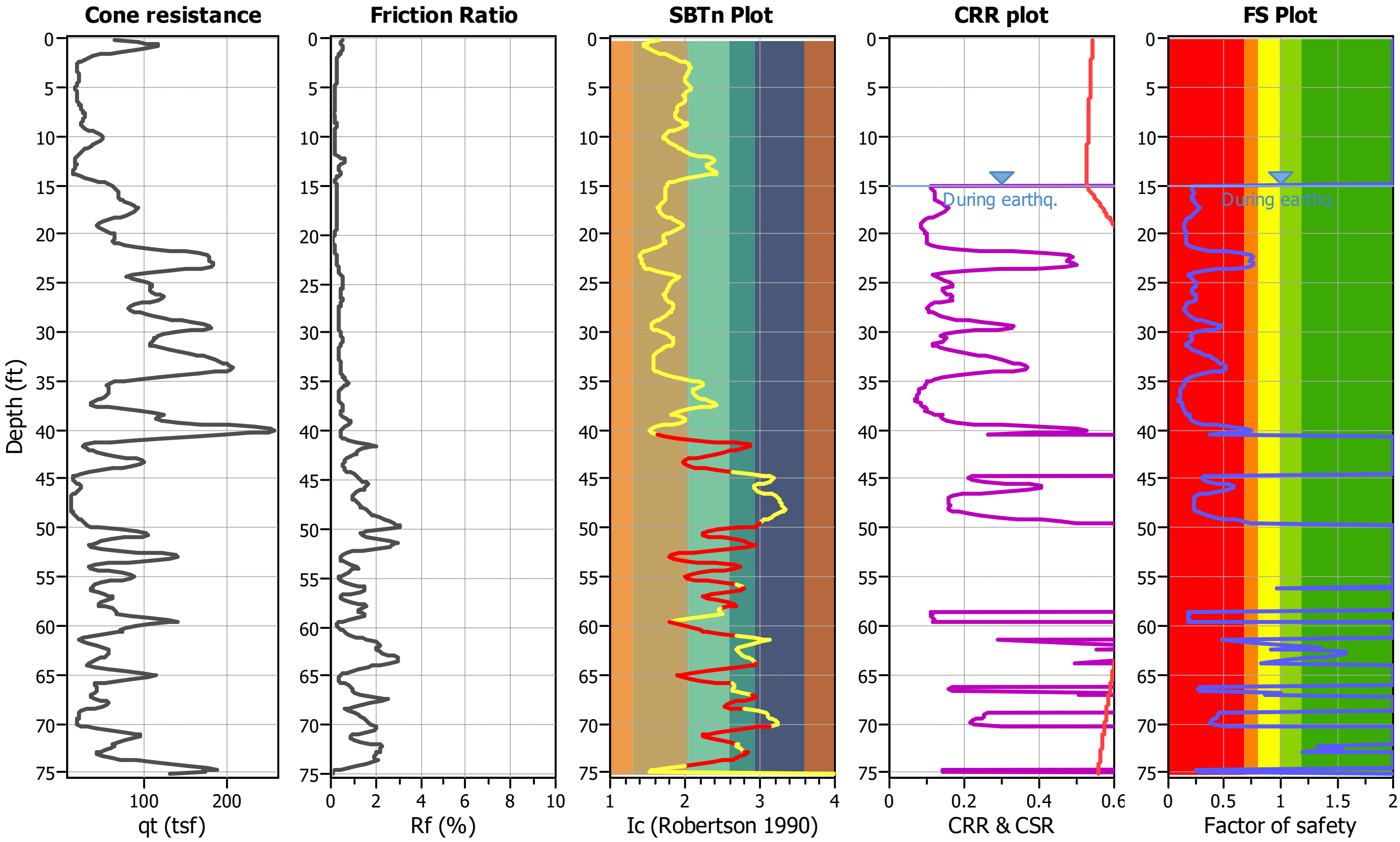
LIQUEFACTION ANALYSIS REPORT

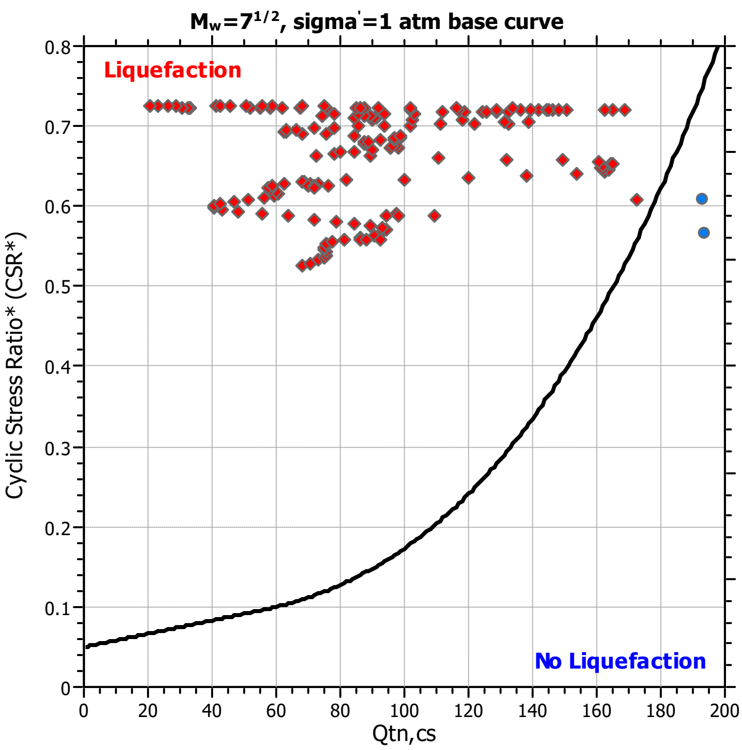
Project title : Moorpark Library **Location : High Street and Moorpark Avenue, Moorpark, Caliornia**

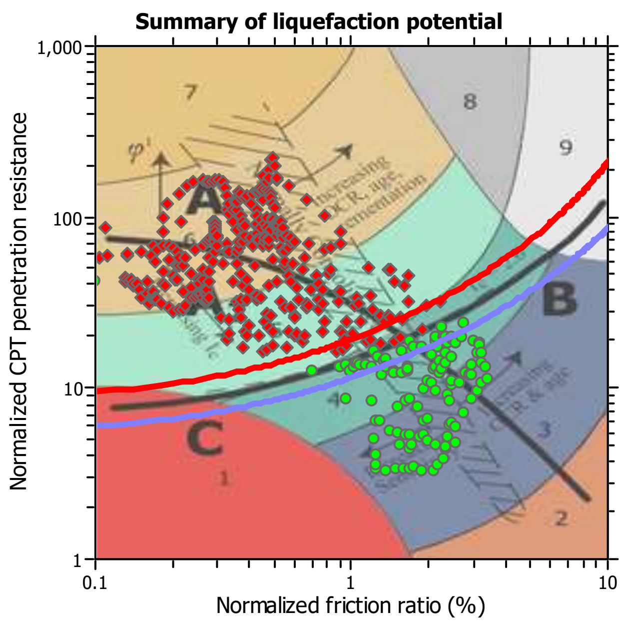
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	37.50 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	No		

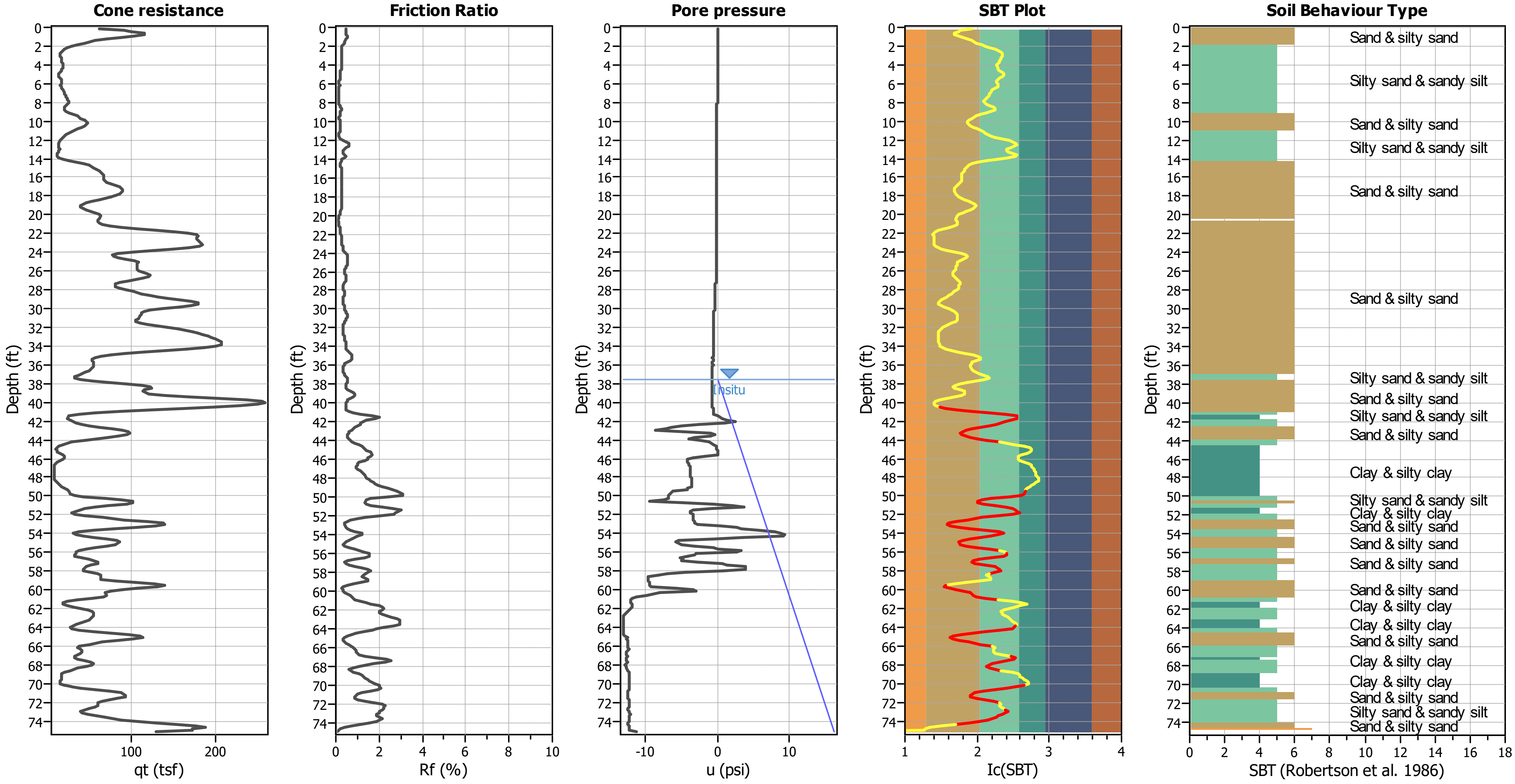






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



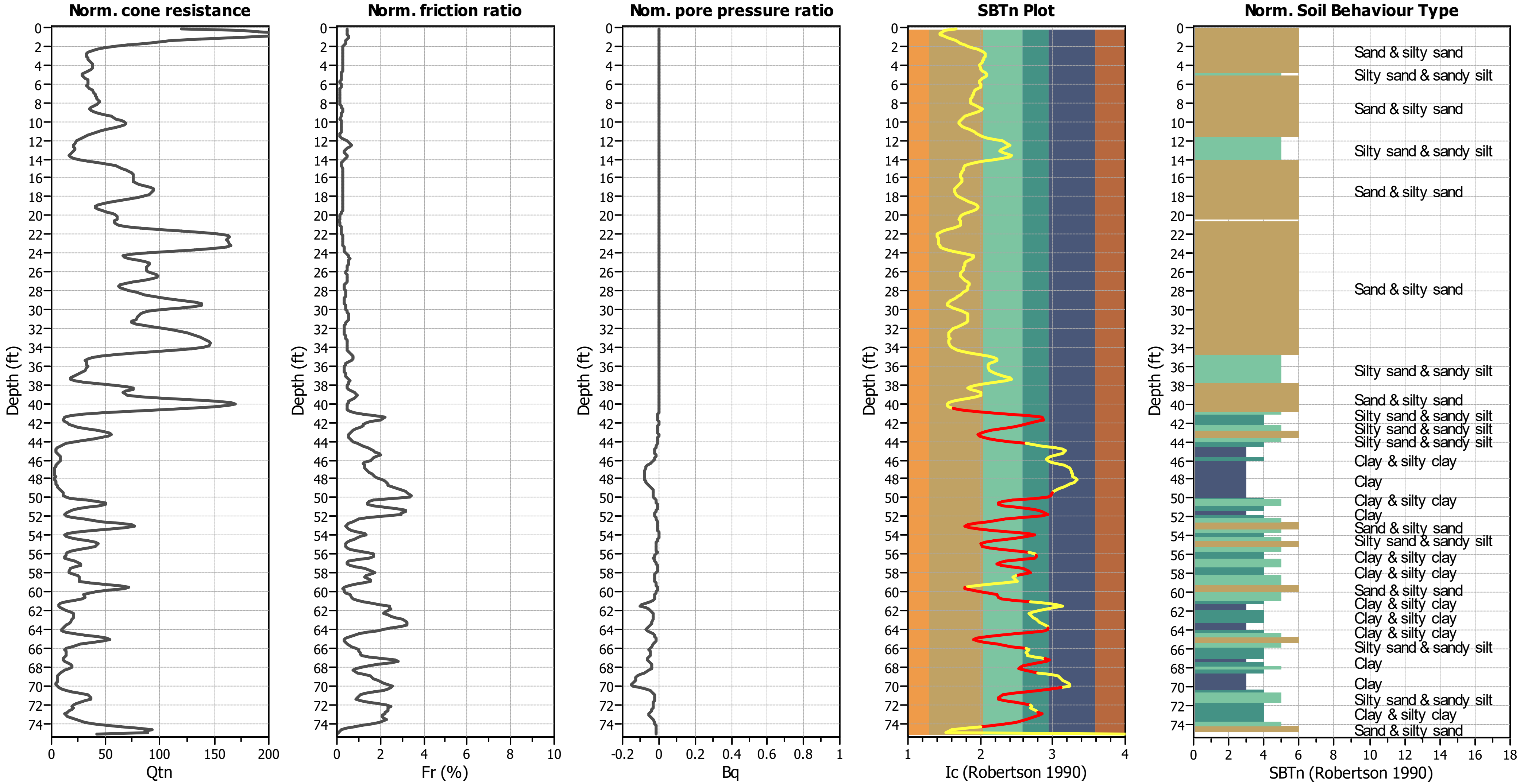
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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)



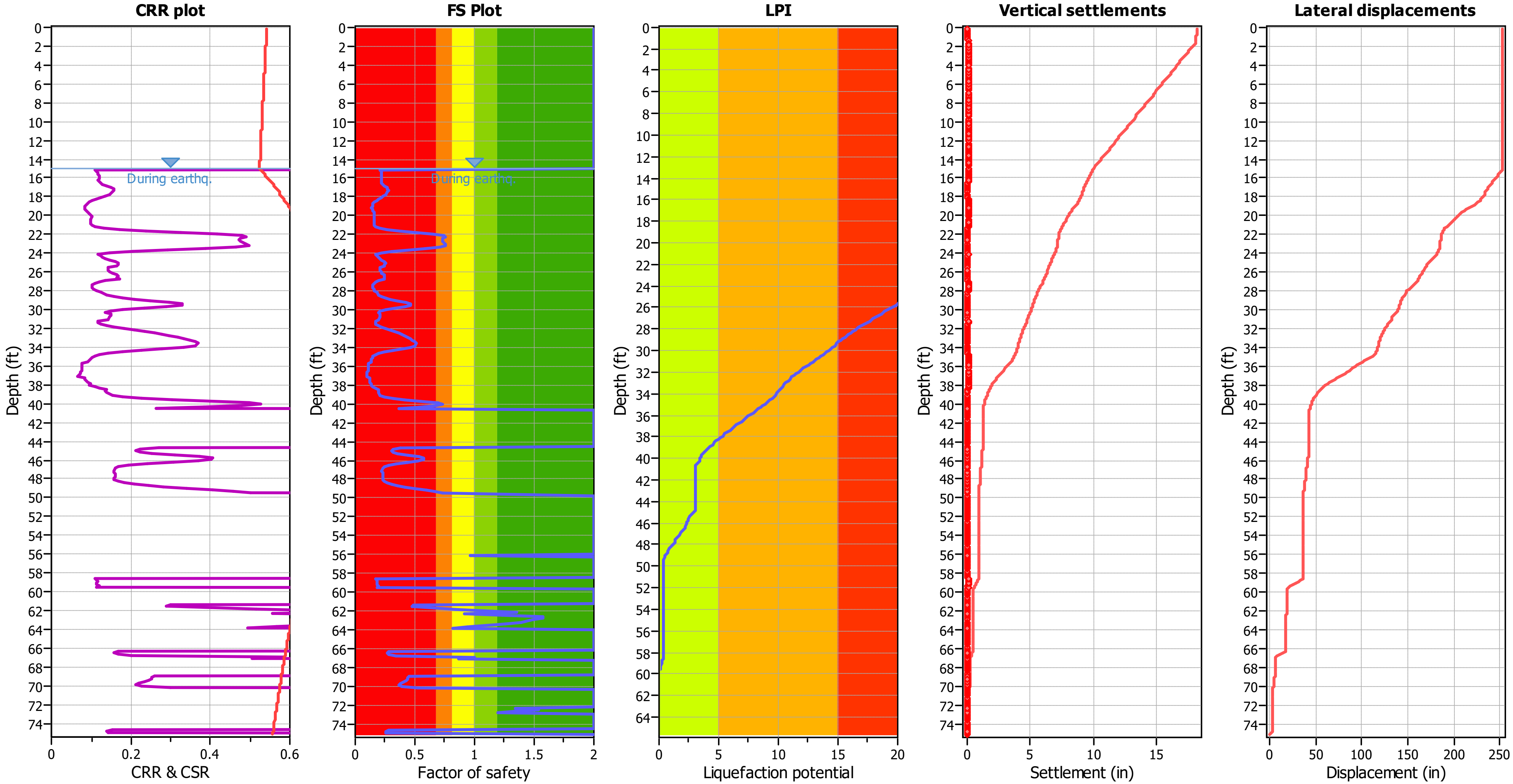
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

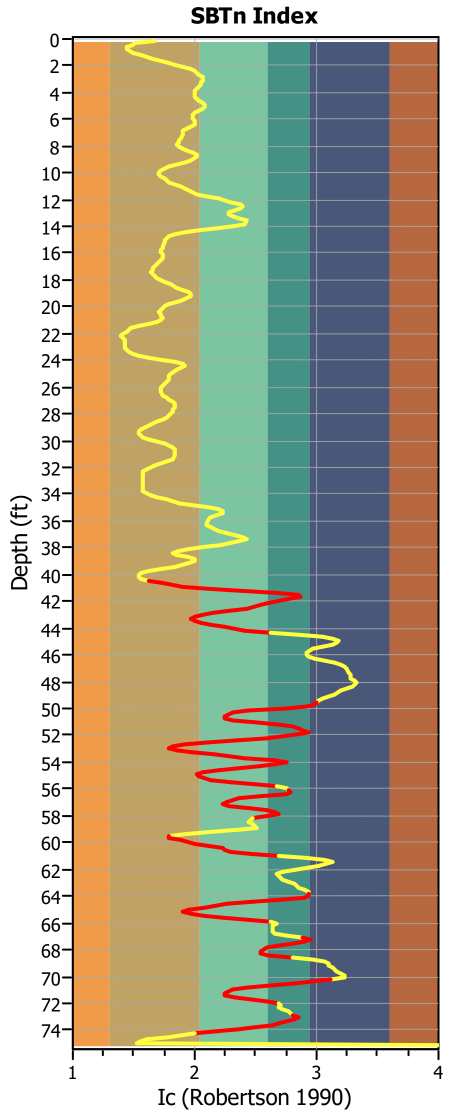
- Very high risk
- High risk
- Low risk

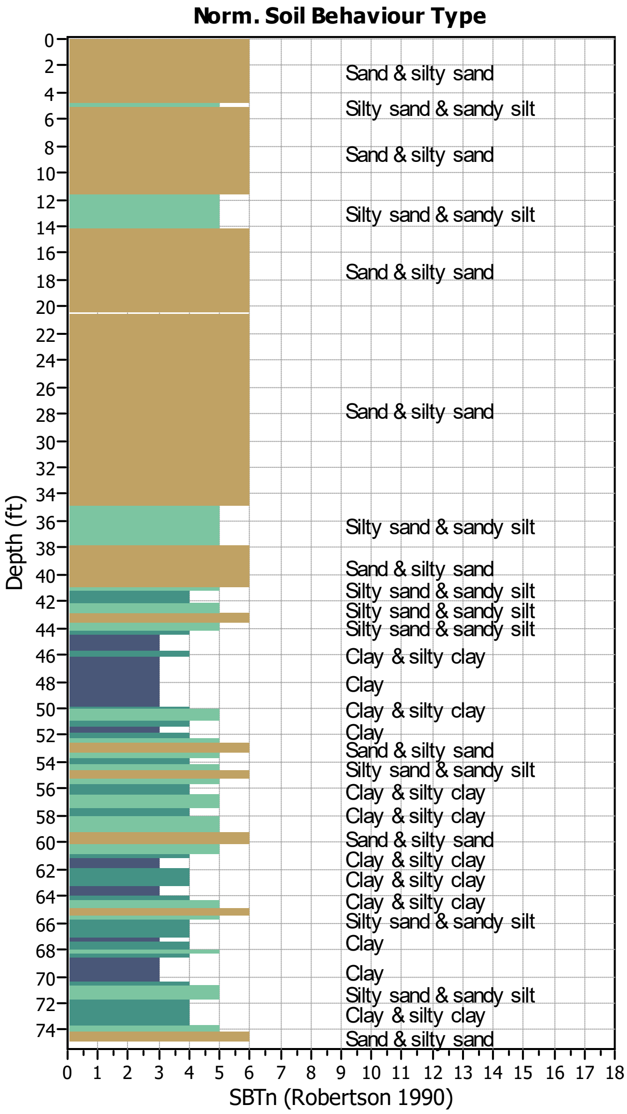
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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).

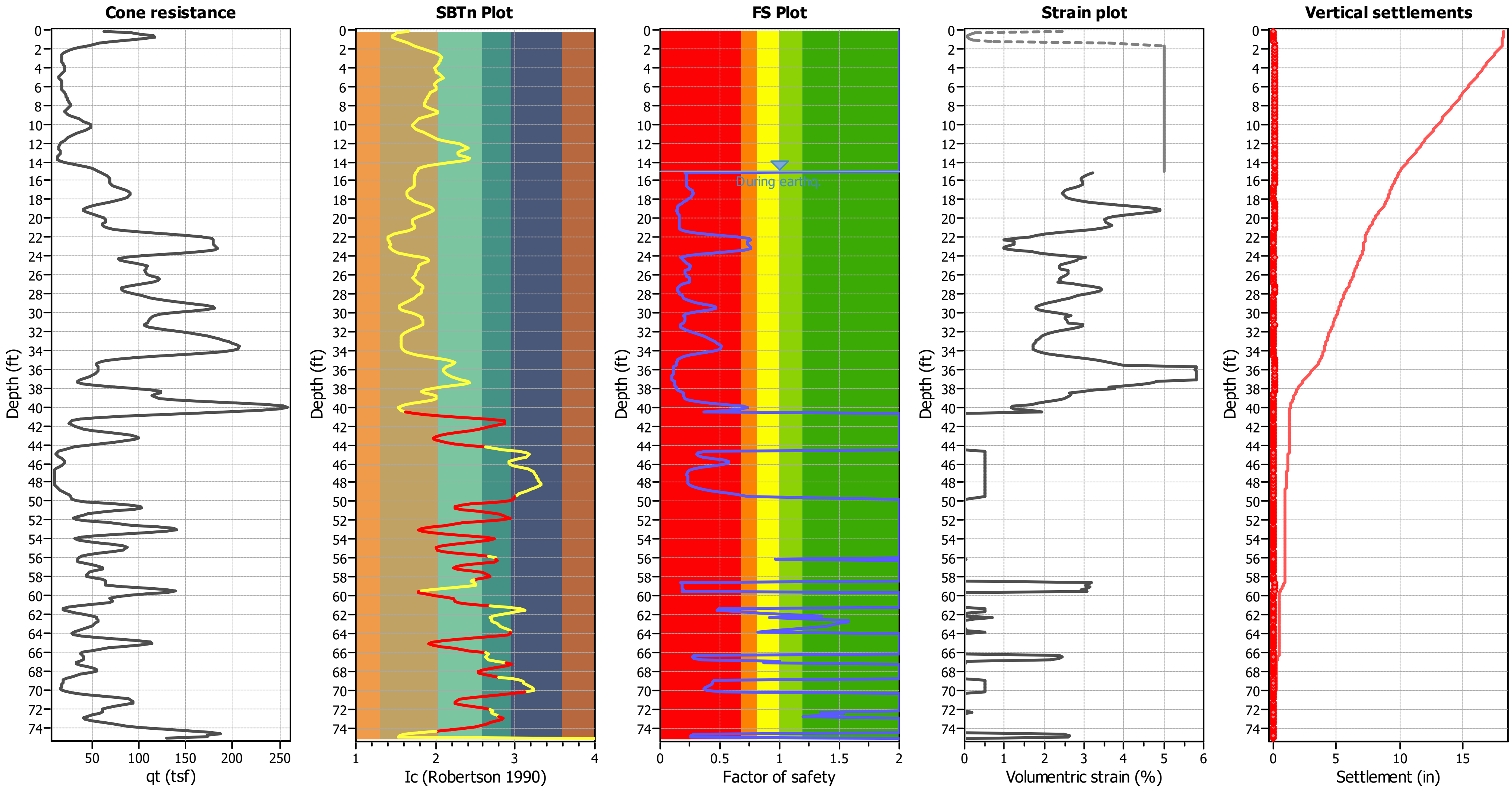
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	458
I_c maximum check value:	3.00	Total points excluded:	139
I_c change ratio value:	0.0100	Exclusion percentage:	30.35%
Minimum number of points in layer:	4	Number of layers detected:	20

Estimation of post-earthquake settlements

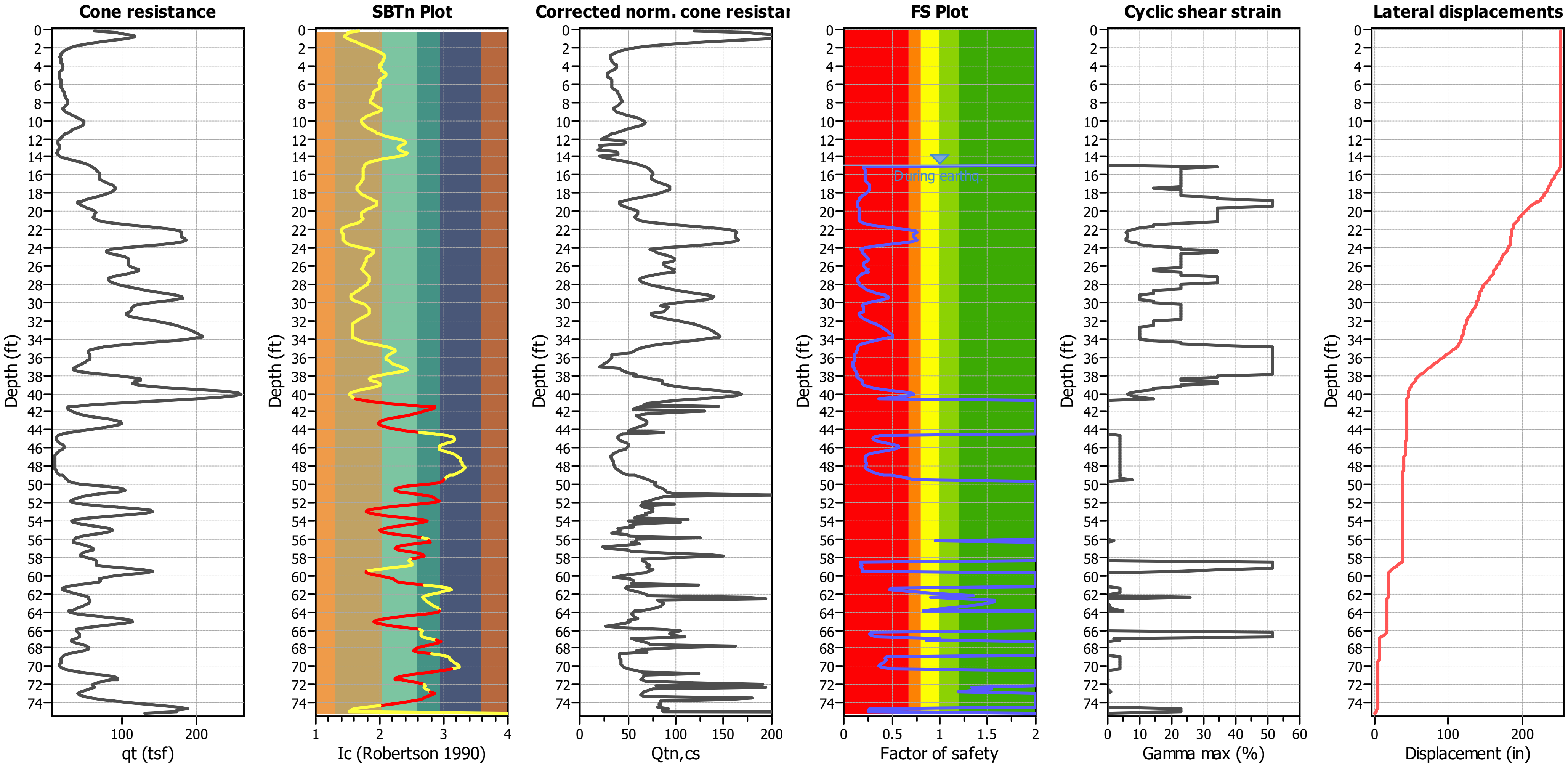


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 2.50 %)

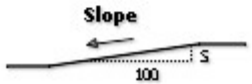


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



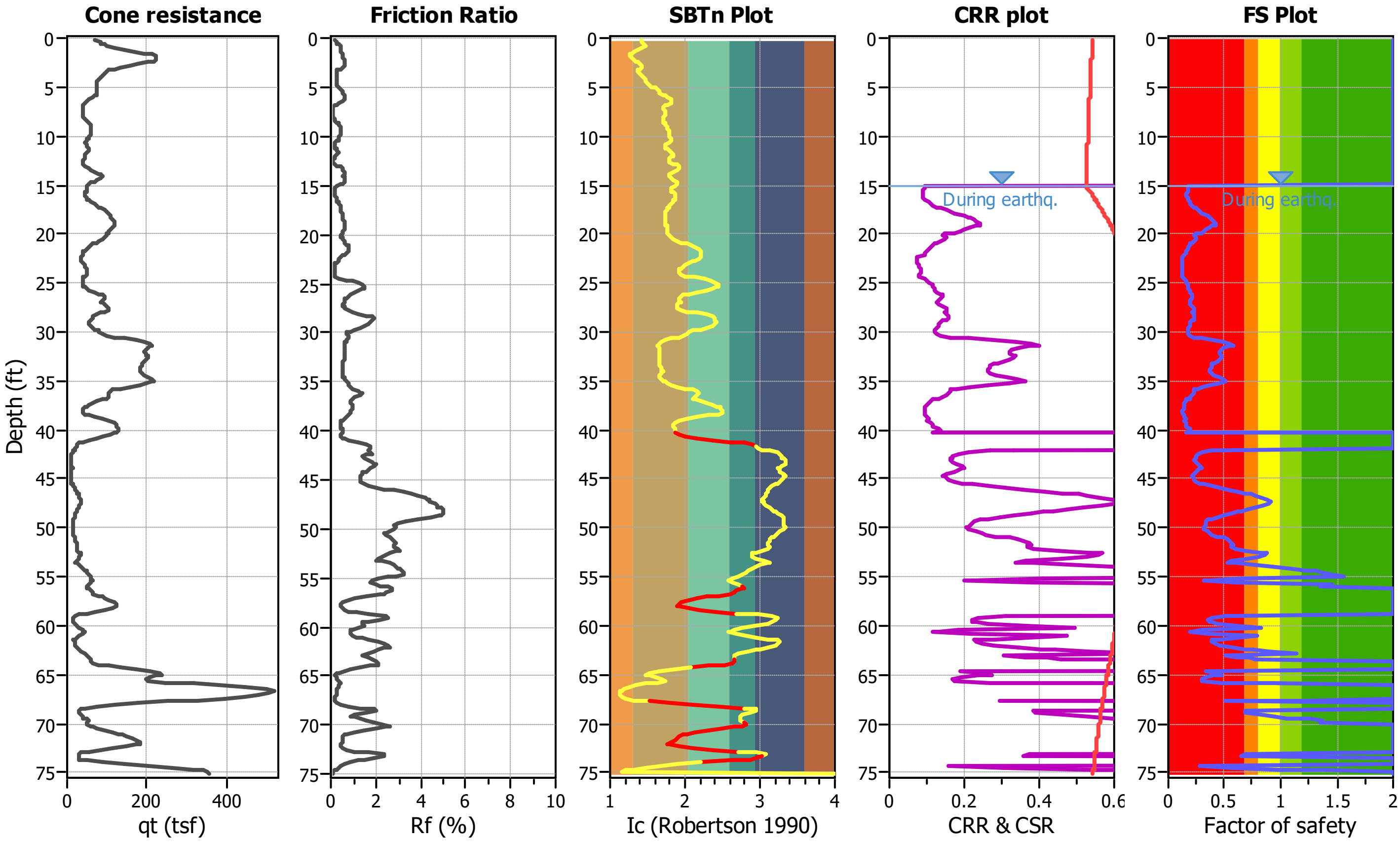
LIQUEFACTION ANALYSIS REPORT

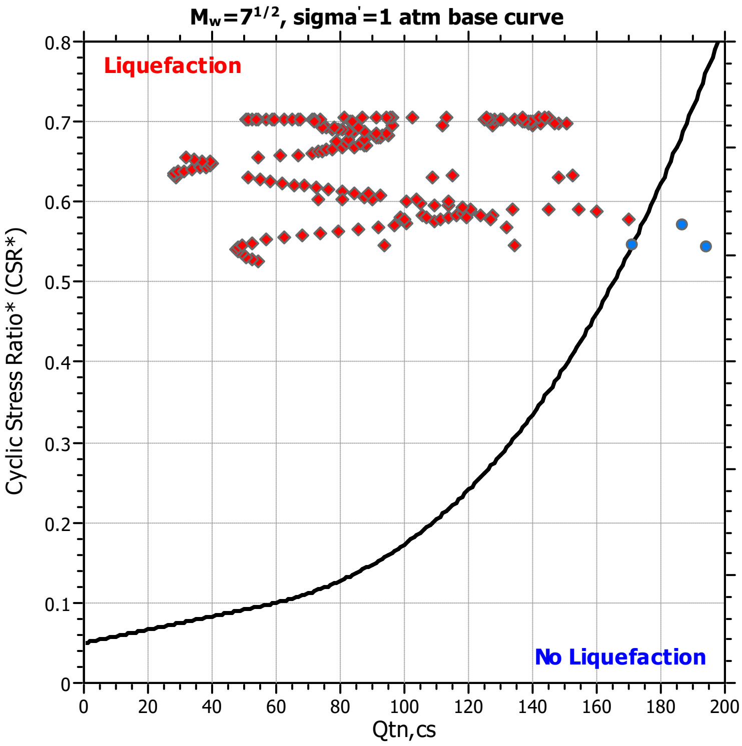
Project title : Moorpark Library **Location : High Street and Moorpark Avenue, Moorpark, Caliornia**

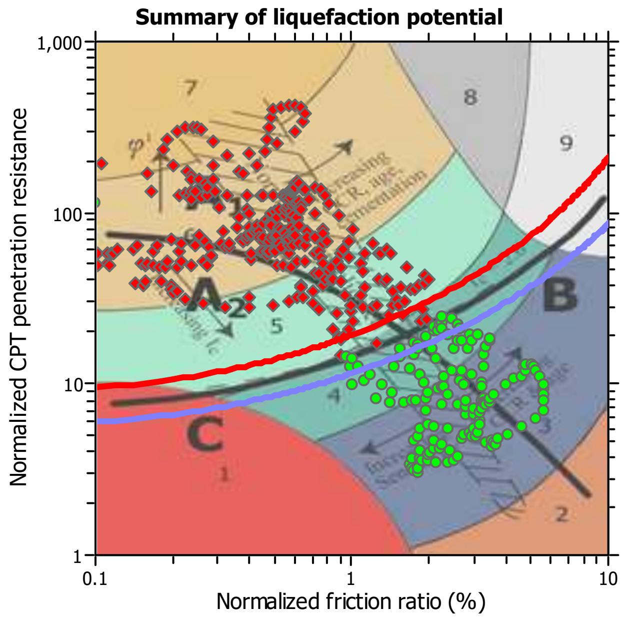
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	37.50 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	No		

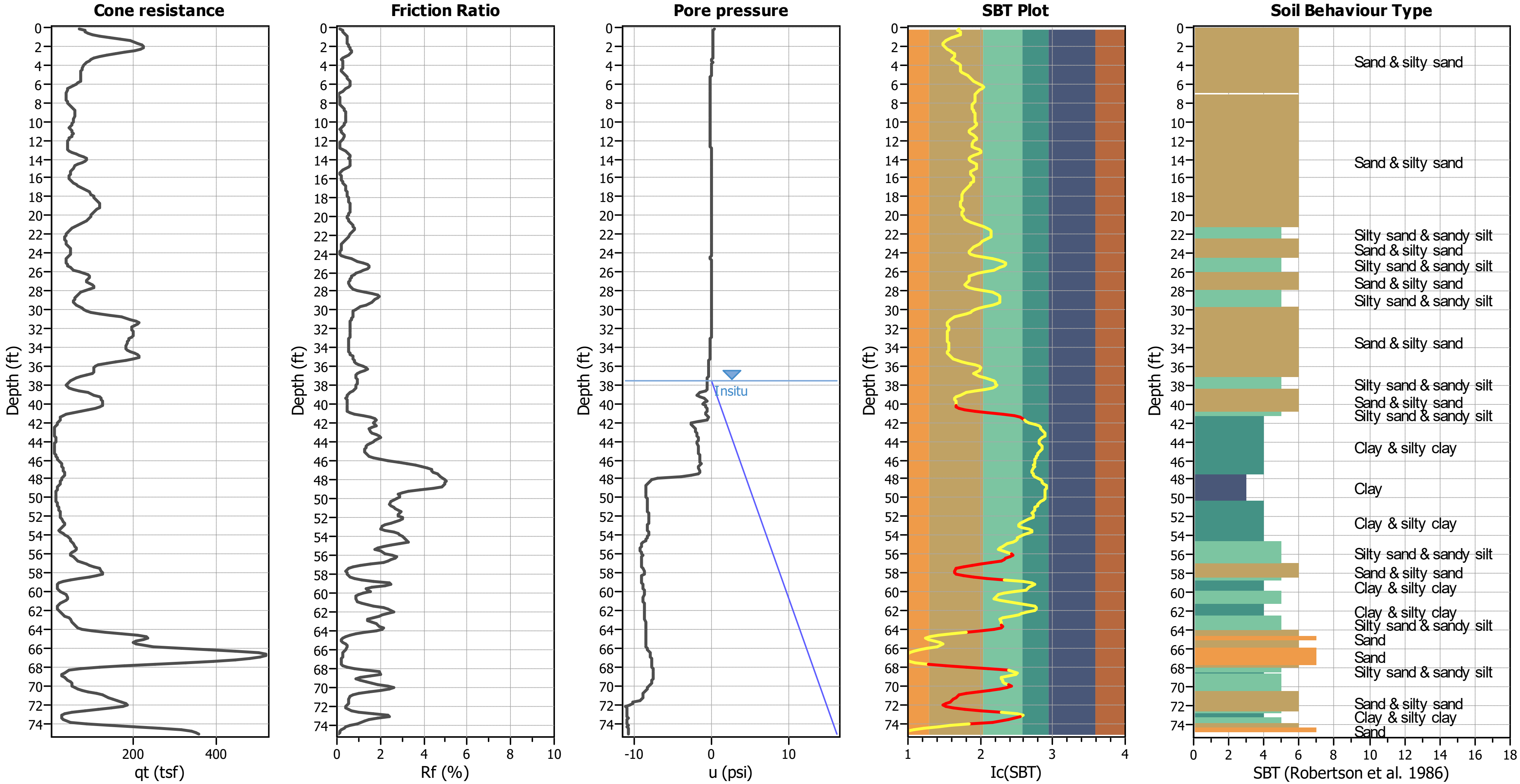






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



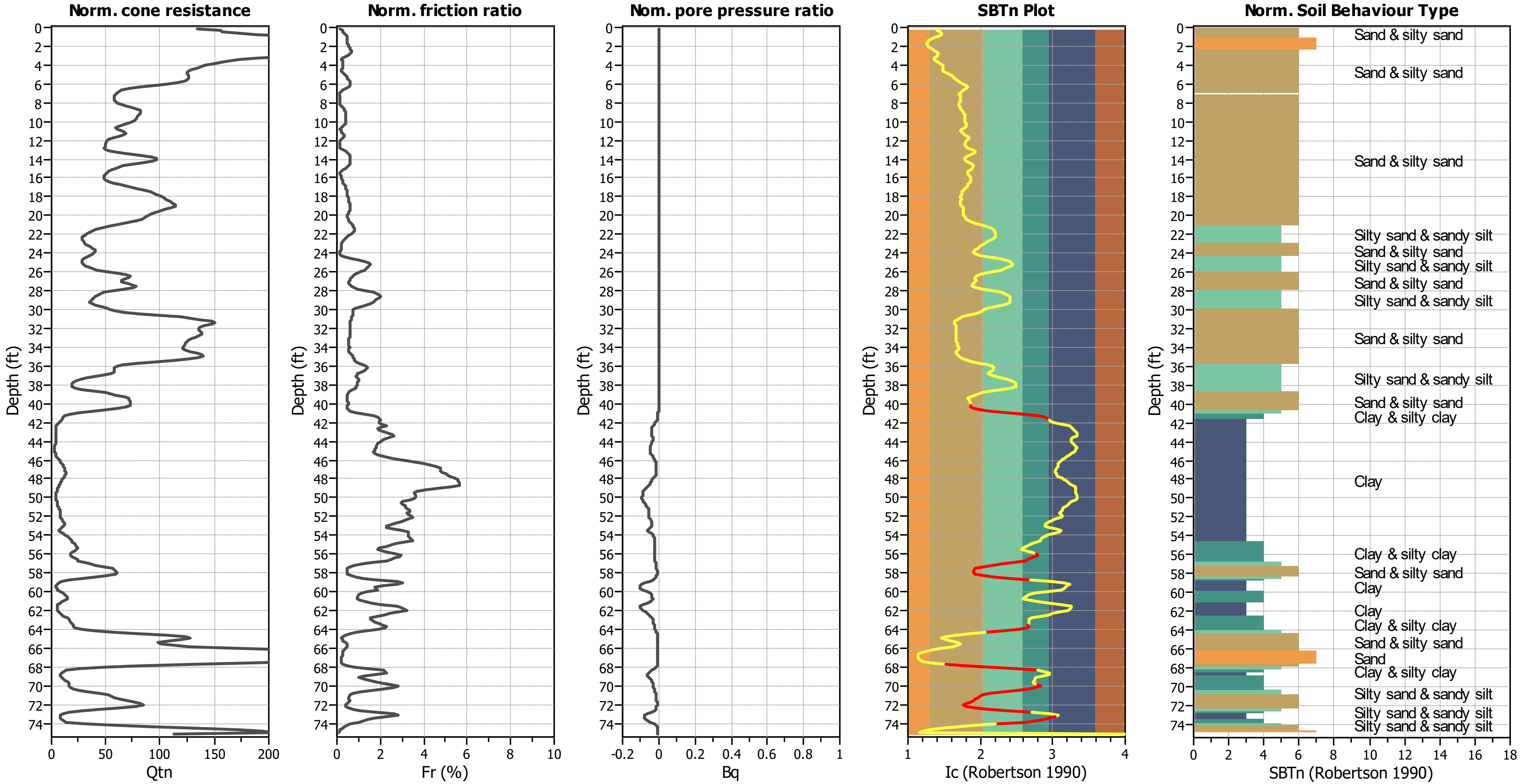
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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)



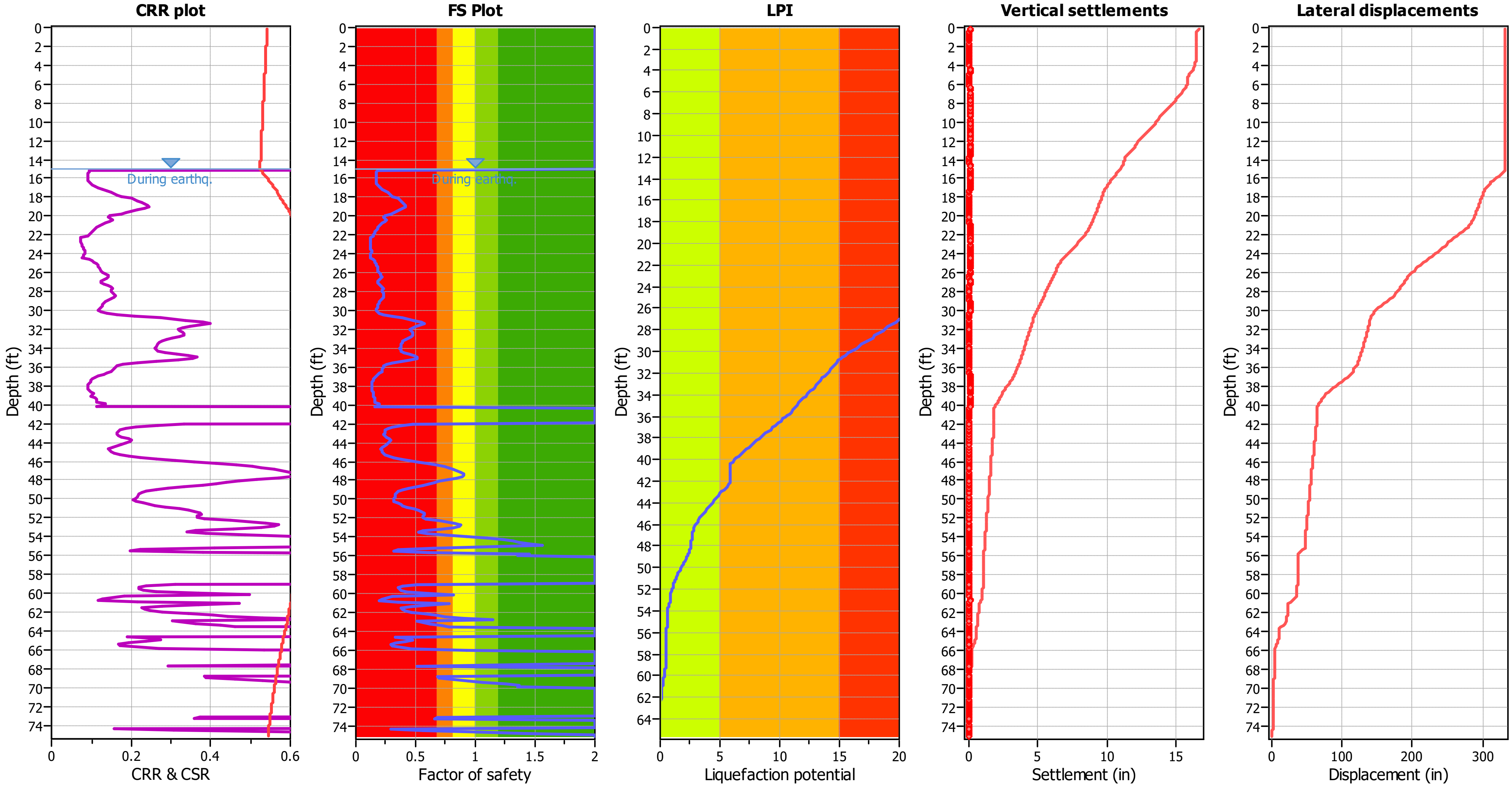
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

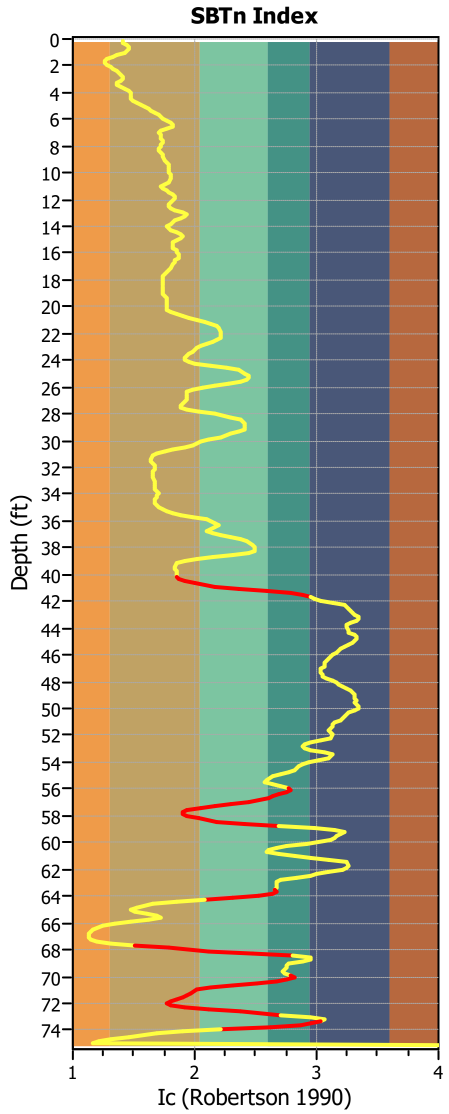
- Very high risk
- High risk
- Low risk

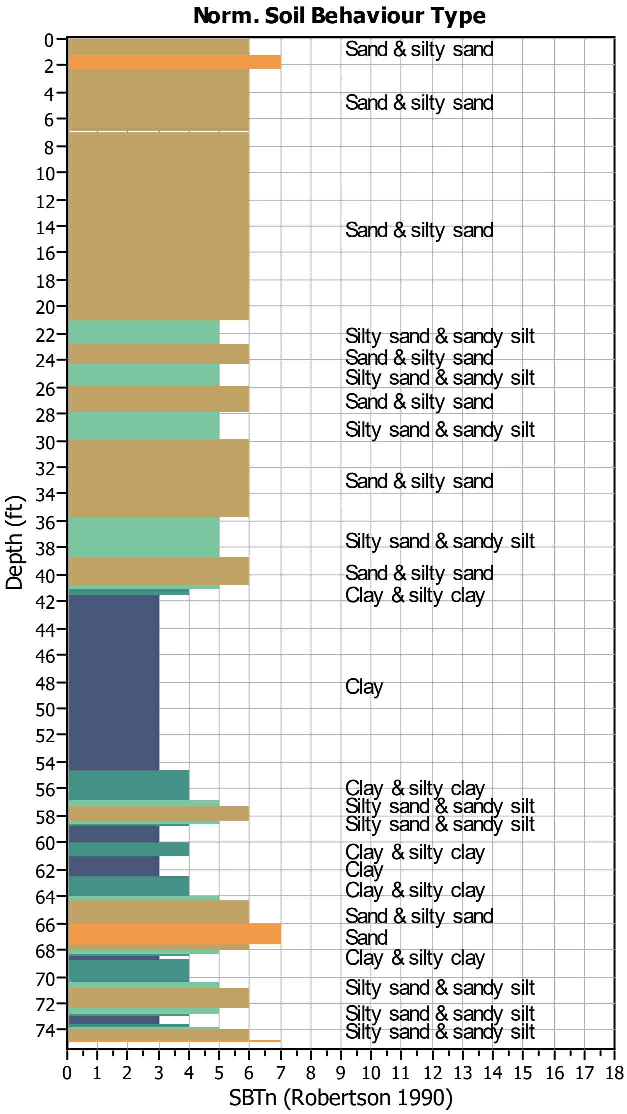
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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.

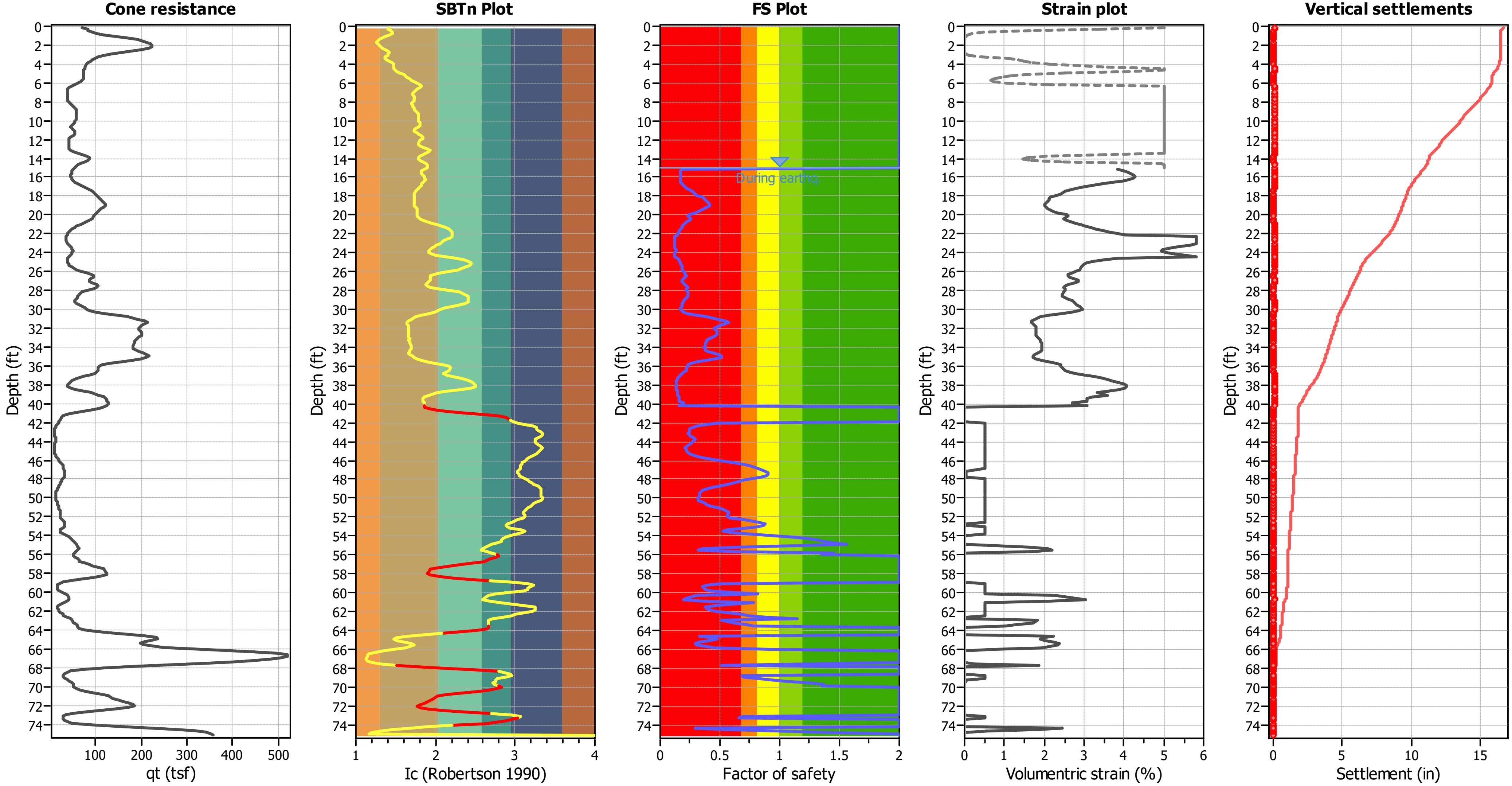




Transition layer algorithm properties	
I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0100
Minimum number of points in layer:	4

General statistics	
Total points in CPT file:	458
Total points excluded:	64
Exclusion percentage:	13.97%
Number of layers detected:	8

Estimation of post-earthquake settlements

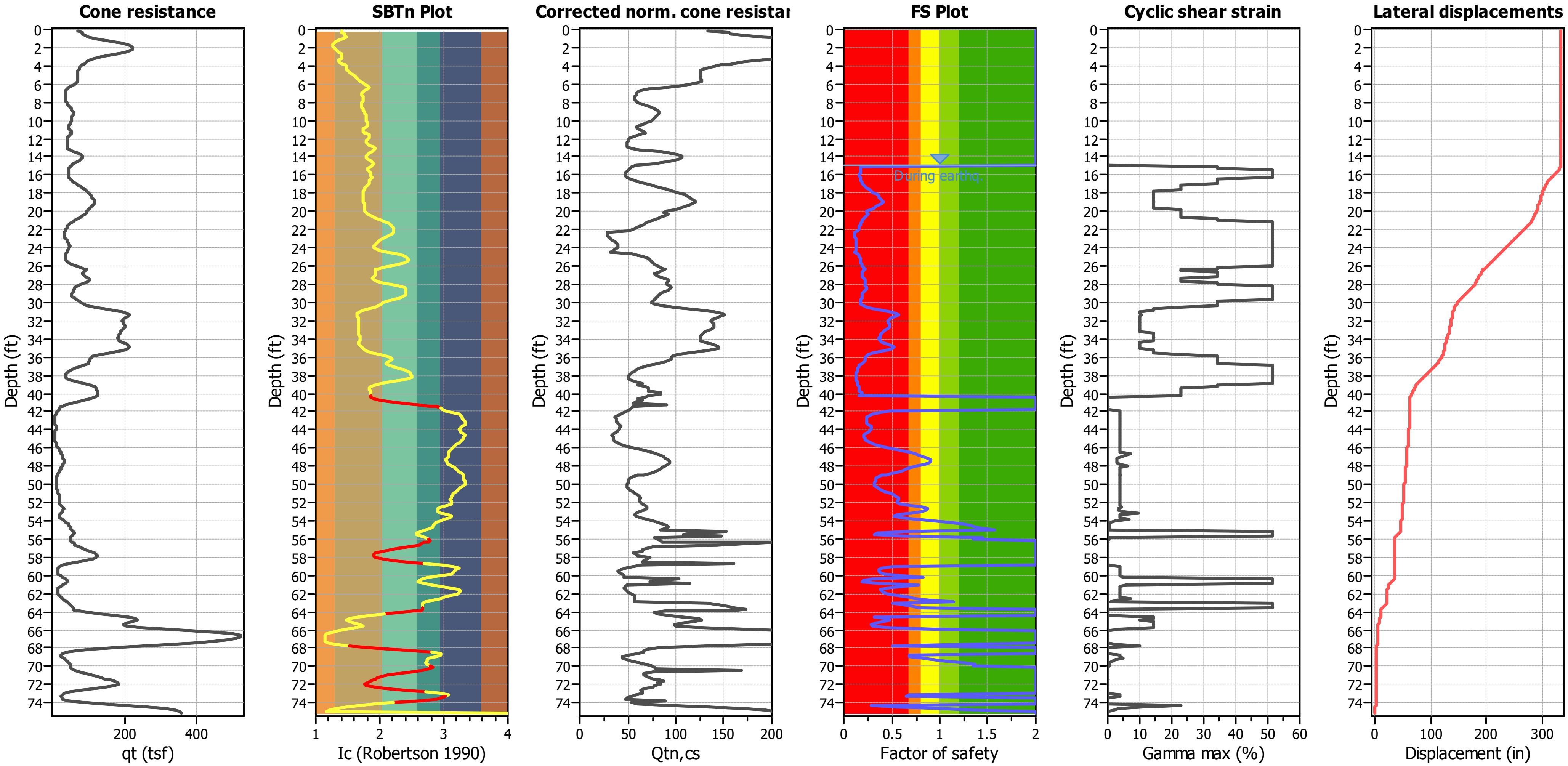


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 2.50 %)

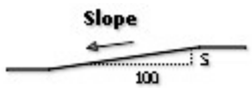


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



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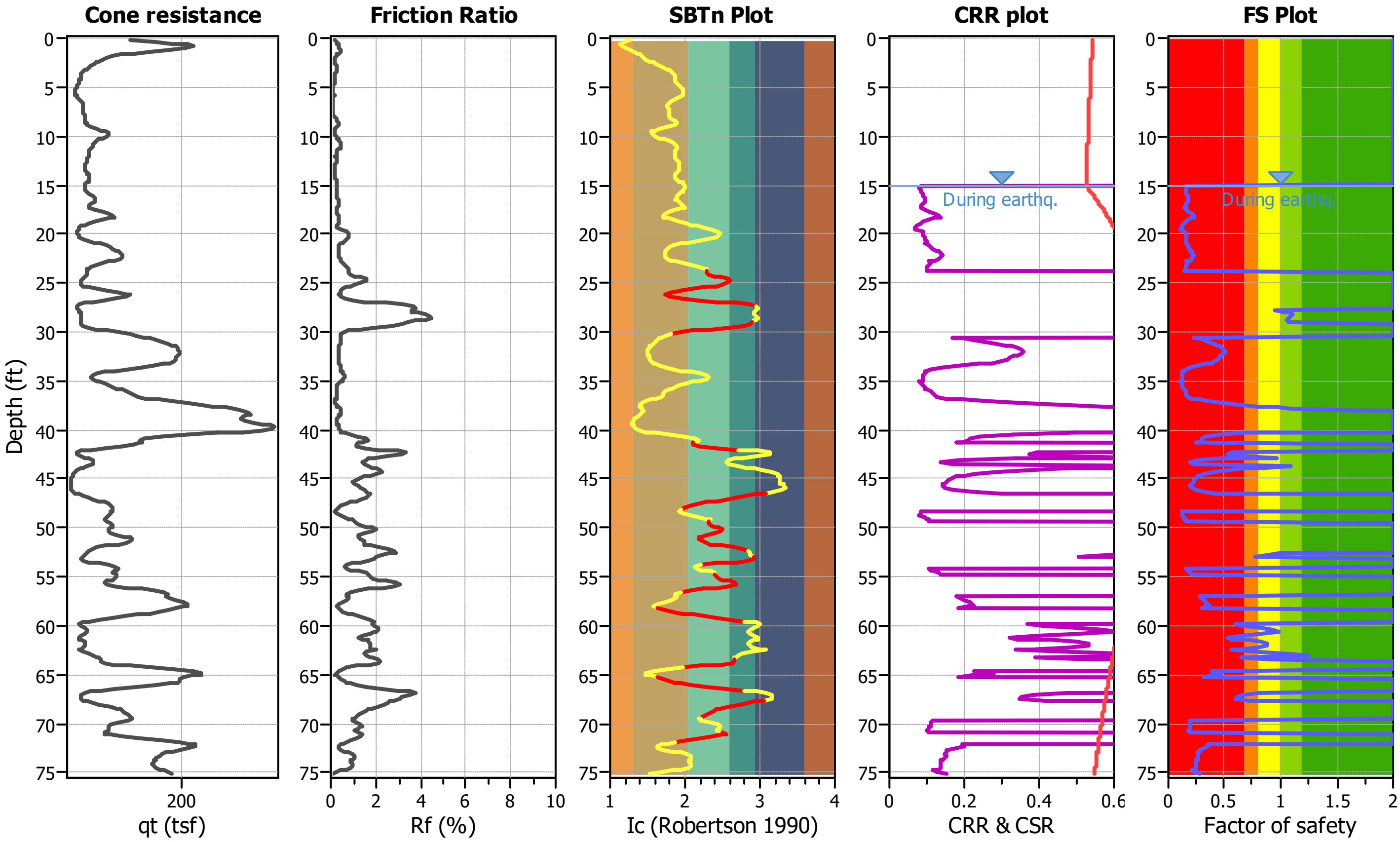
LIQUEFACTION ANALYSIS REPORT

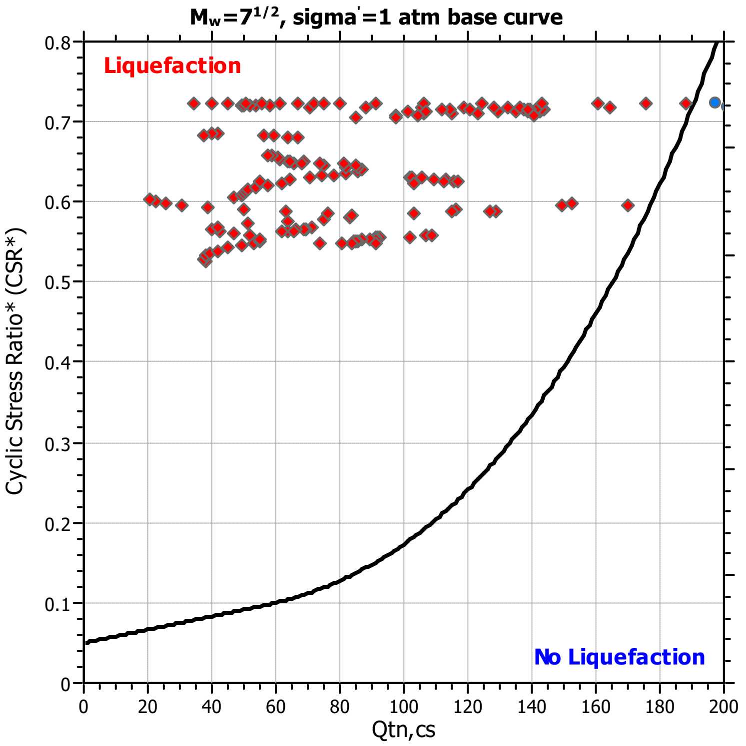
Project title : Moorpark Library **Location : High Street and Moorpark Avenue, Moorpark, Caliornia**

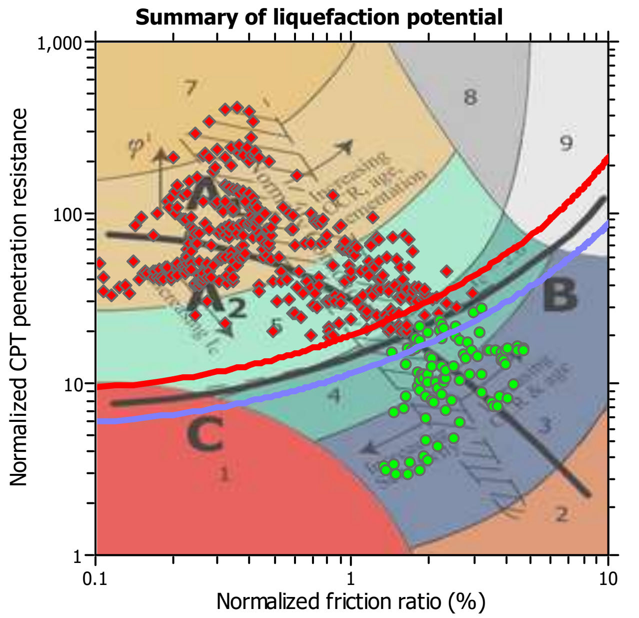
CPT file : CPT-5

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	37.50 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	No		

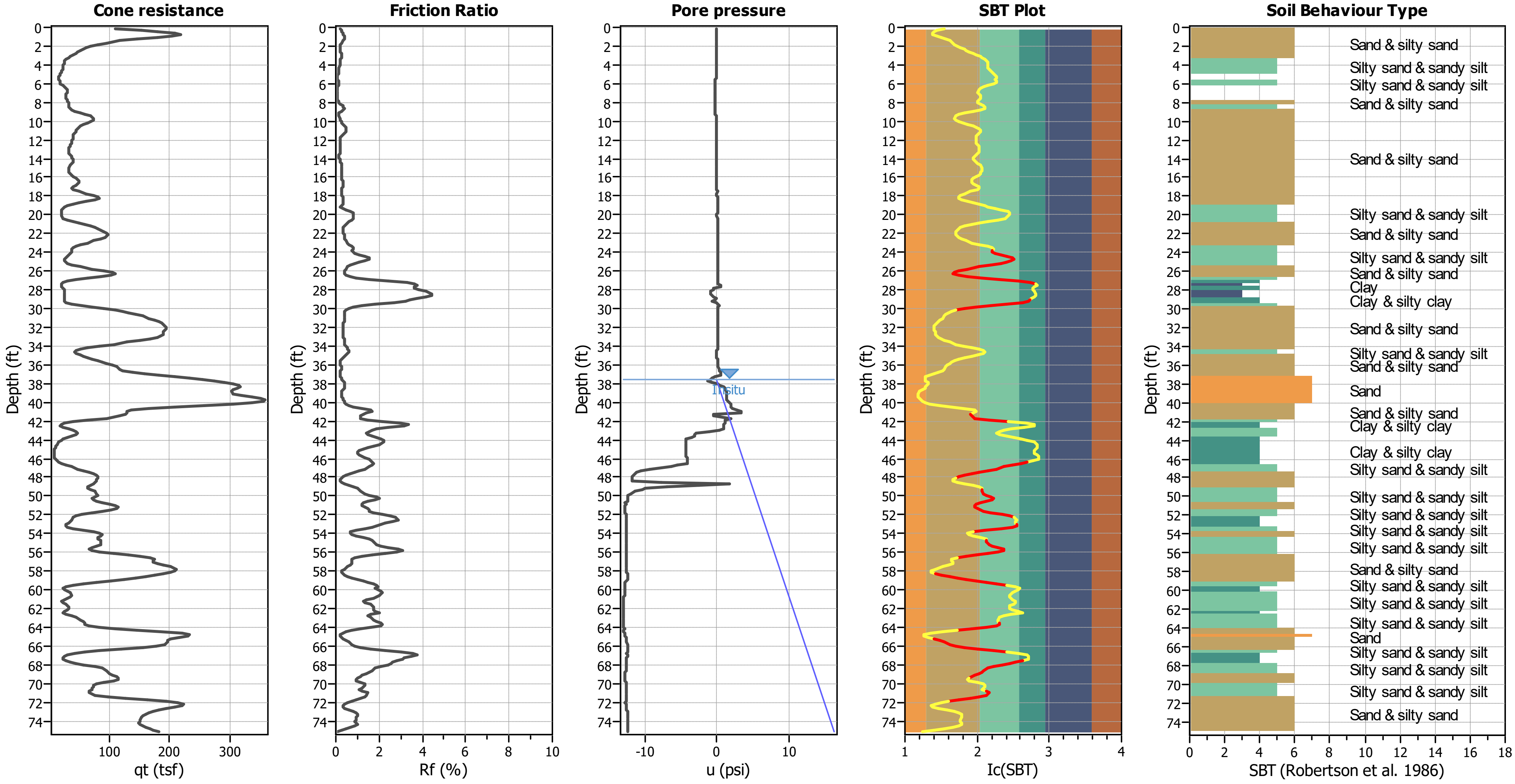






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



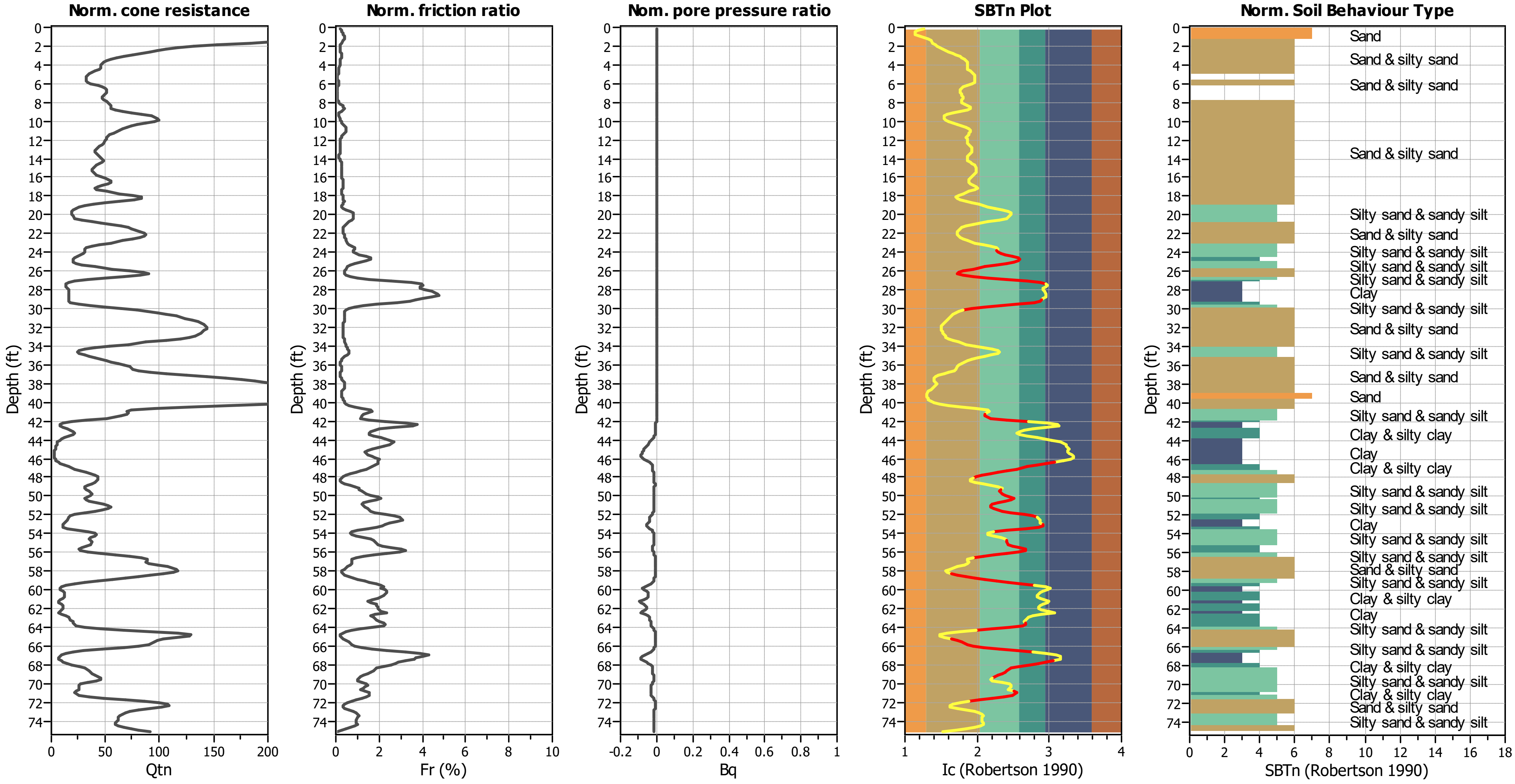
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

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)



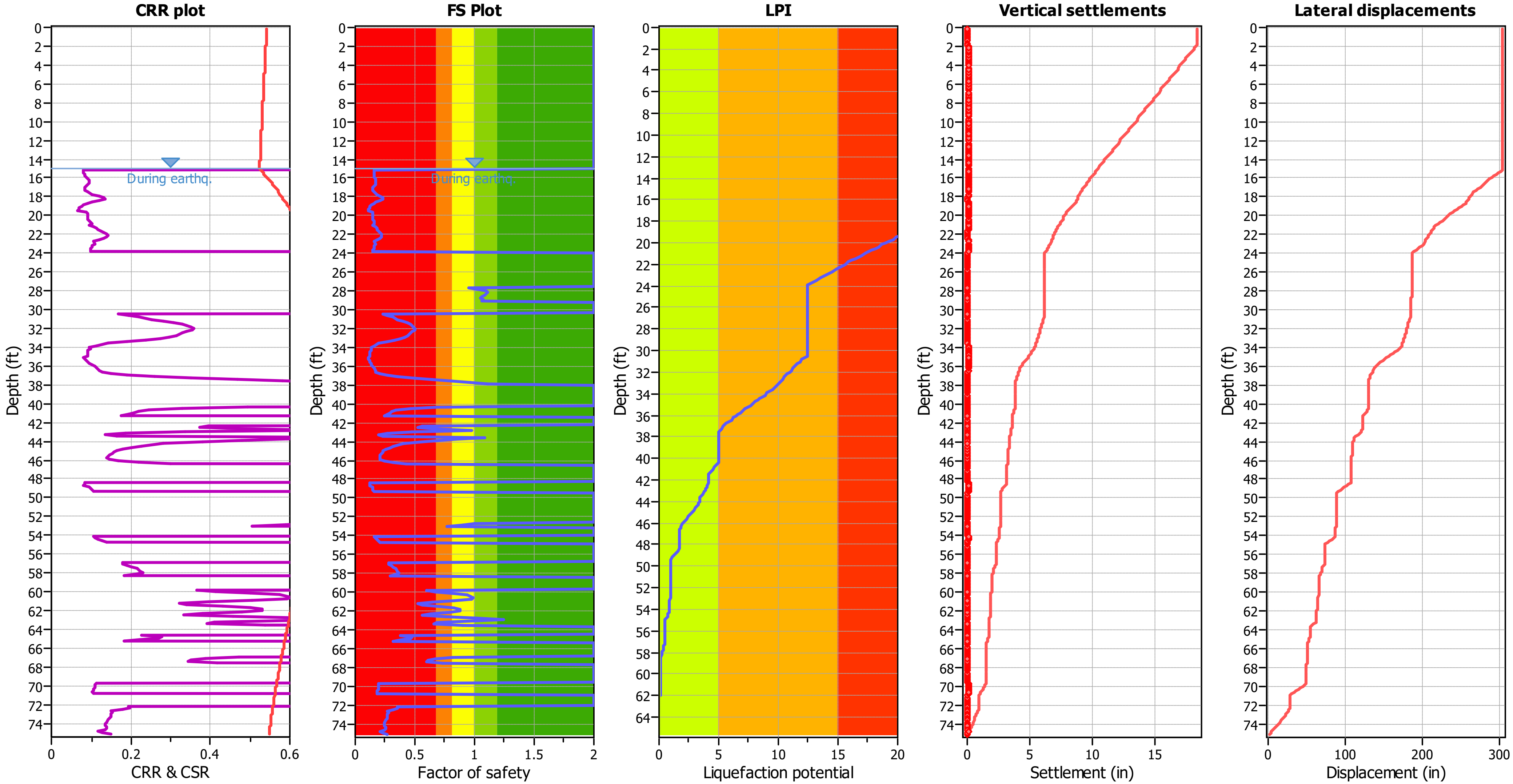
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
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.):	15.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	1.03	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	37.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

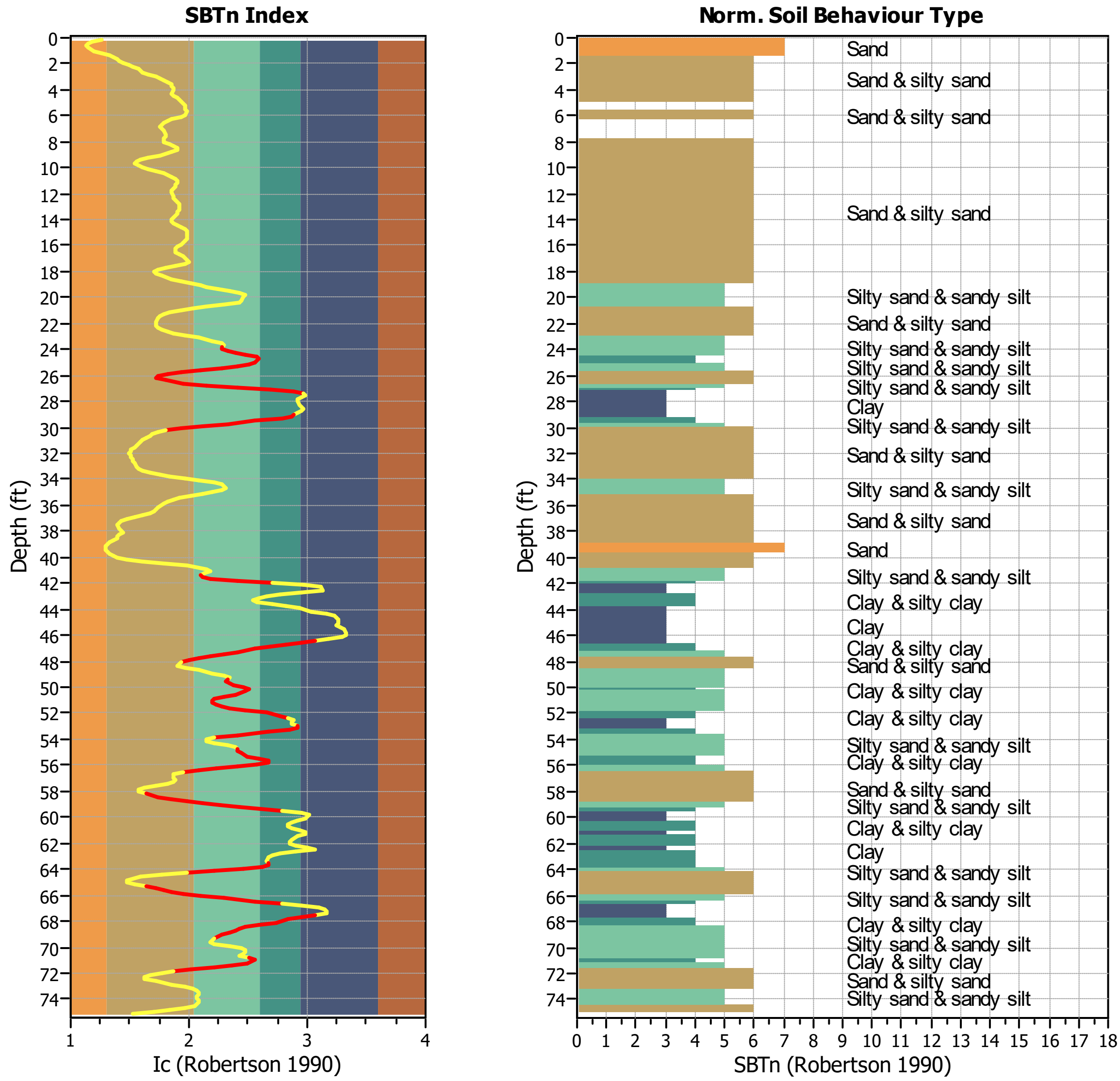
- Very high risk
- High risk
- Low risk

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.



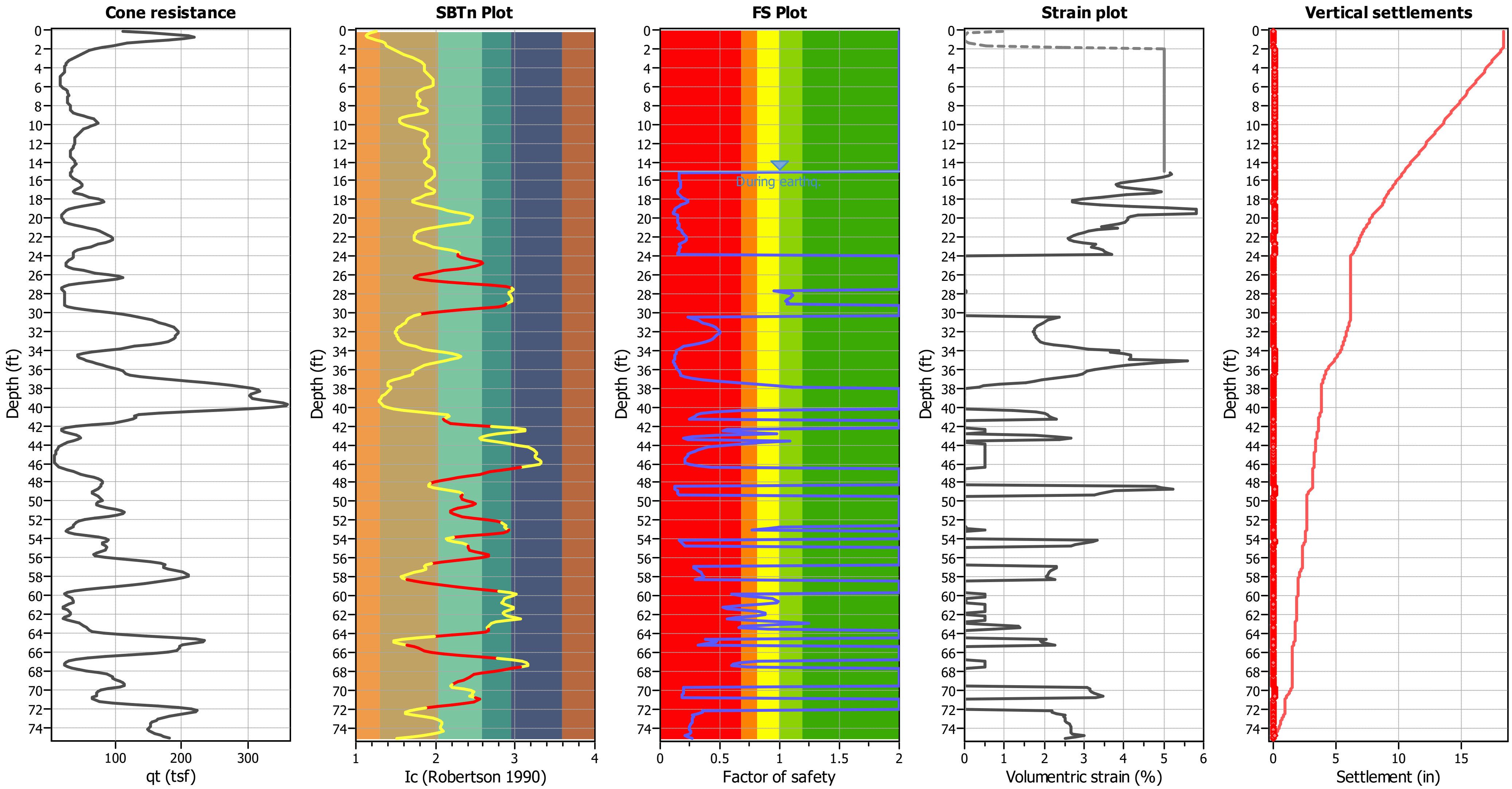
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0100
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	458
Total points excluded:	129
Exclusion percentage:	28.17%
Number of layers detected:	17

Estimation of post-earthquake settlements

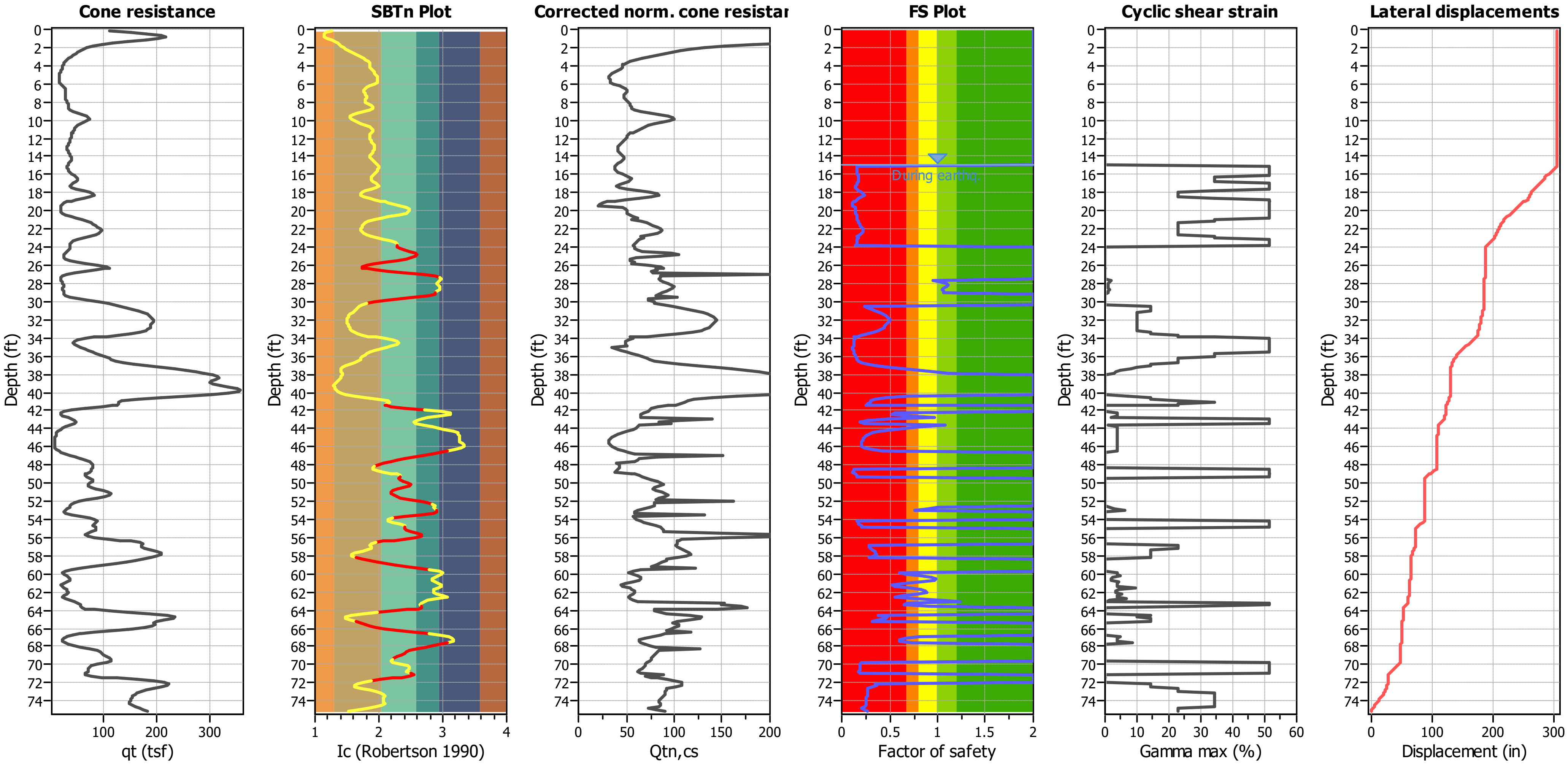


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 2.50 %)

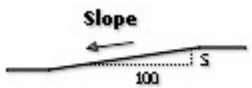


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



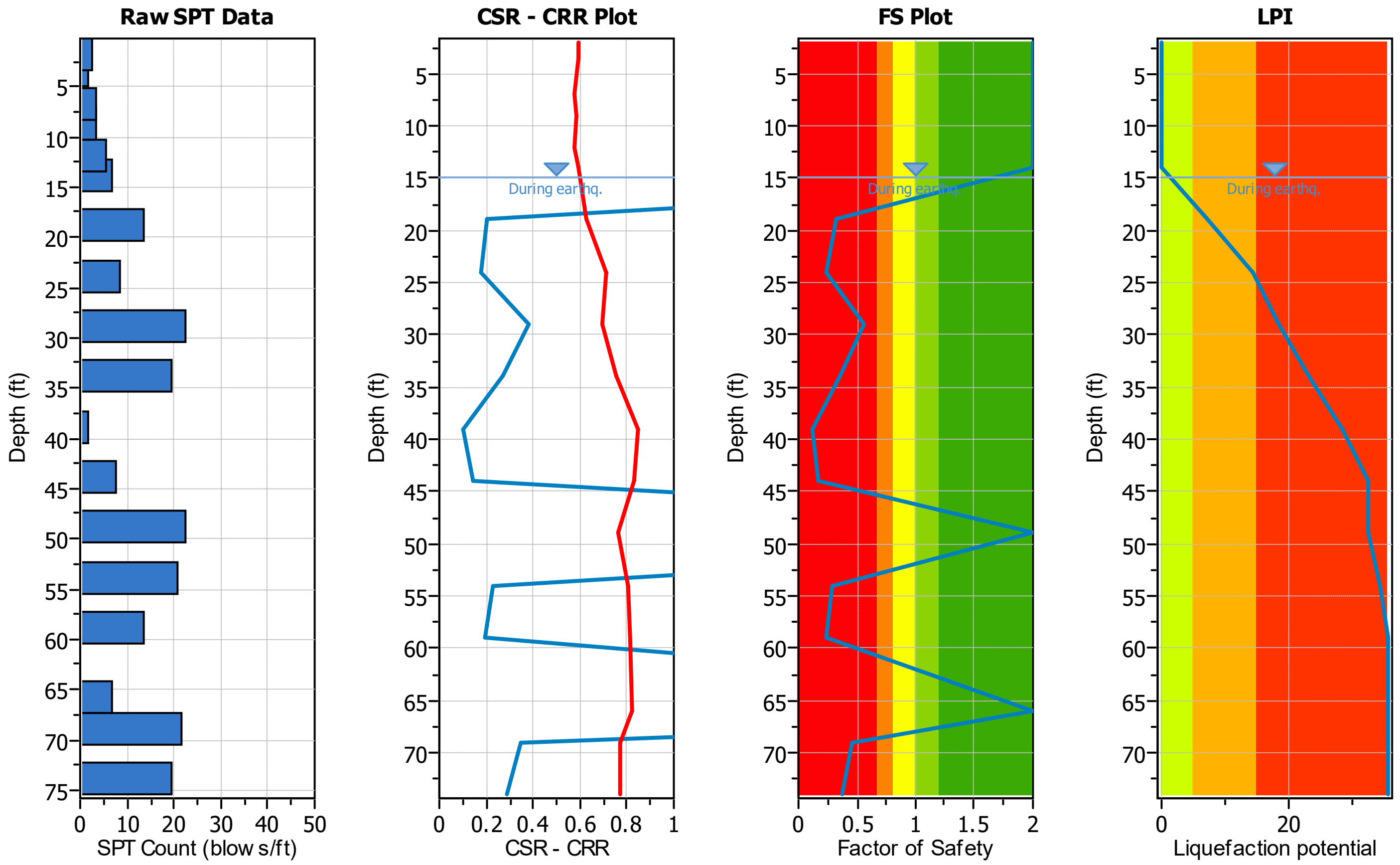
SPT BASED LIQUEFACTION ANALYSIS REPORT

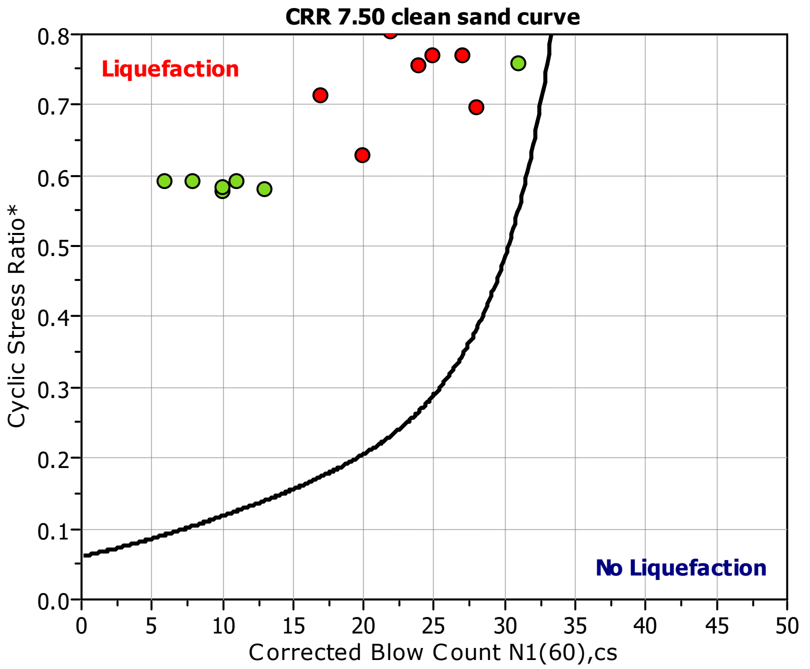
Project title : Moorpark Library | **SPT Name: DH #1**

Location : High Street and Moorpark Avenue

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	37.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	15.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.90 ft
Borehole diameter:	200mm	Peak ground acceleration:	1.03 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.30		





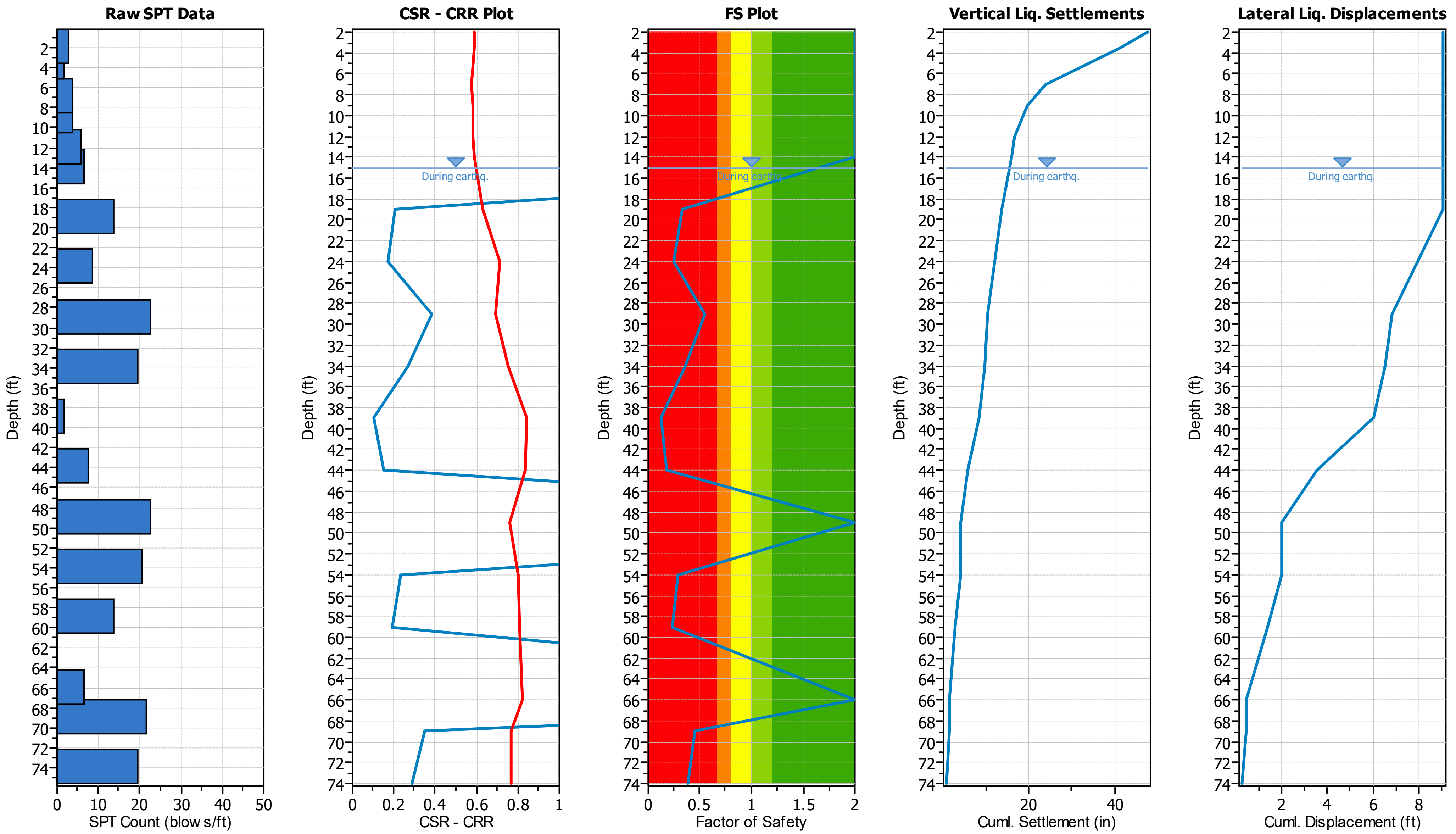
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
2.00	3	12.00	98.00	2.00	No
3.50	2	12.00	98.00	2.00	No
7.00	4	12.00	105.00	4.00	No
9.00	4	15.00	105.00	3.00	No
12.00	6	15.00	111.00	2.00	No
14.00	7	7.00	111.00	3.00	Yes
19.00	14	7.00	111.00	7.00	Yes
24.00	9	23.00	108.00	5.00	Yes
29.00	23	7.00	107.00	5.00	Yes
34.00	20	7.00	107.00	5.00	Yes
39.00	2	50.00	112.00	5.00	Yes
44.00	8	50.00	112.00	5.00	Yes
49.00	23	25.00	112.00	5.00	Yes
54.00	21	3.00	112.00	5.00	Yes
59.00	14	25.00	112.00	5.00	Yes
66.00	7	63.00	112.00	3.00	No
69.00	22	24.00	112.00	3.00	Yes
74.00	20	24.00	112.00	3.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
2.00	3	98.00	0.10	0.00	0.10	0.55	1.70	1.30	1.15	0.75	1.00	6	12.00	2.07	8	4.000
3.50	2	98.00	0.17	0.00	0.17	0.58	1.70	1.30	1.15	0.75	1.00	4	12.00	2.07	6	4.000
7.00	4	105.00	0.36	0.00	0.36	0.52	1.70	1.30	1.15	0.80	1.00	8	12.00	2.07	10	4.000
9.00	4	105.00	0.46	0.00	0.46	0.51	1.53	1.30	1.15	0.80	1.00	7	15.00	3.26	10	4.000
12.00	6	111.00	0.63	0.00	0.63	0.50	1.30	1.30	1.15	0.85	1.00	10	15.00	3.26	13	4.000
14.00	7	111.00	0.74	0.00	0.74	0.53	1.21	1.30	1.15	0.85	1.00	11	7.00	0.14	11	4.000
19.00	14	111.00	1.02	0.00	1.02	0.44	1.02	1.30	1.15	0.95	1.00	20	7.00	0.14	20	0.206
24.00	9	108.00	1.29	0.00	1.29	0.48	0.91	1.30	1.15	0.95	1.00	12	23.00	4.88	17	0.174
29.00	23	107.00	1.55	0.00	1.55	0.37	0.87	1.30	1.15	0.95	1.00	28	7.00	0.14	28	0.384
34.00	20	107.00	1.82	0.00	1.82	0.41	0.80	1.30	1.15	1.00	1.00	24	7.00	0.14	24	0.268
39.00	2	112.00	2.10	0.05	2.05	0.60	0.67	1.30	1.15	1.00	1.00	2	50.00	5.61	8	0.105
44.00	8	112.00	2.38	0.20	2.18	0.52	0.69	1.30	1.15	1.00	1.00	8	50.00	5.61	14	0.148
49.00	23	112.00	2.66	0.36	2.30	0.37	0.75	1.30	1.15	1.00	1.00	26	25.00	5.07	31	4.000
54.00	21	112.00	2.94	0.51	2.43	0.42	0.70	1.30	1.15	1.00	1.00	22	3.00	0.00	22	0.233
59.00	14	112.00	3.22	0.67	2.55	0.46	0.66	1.30	1.15	1.00	1.00	14	25.00	5.07	19	0.194
66.00	7	112.00	3.61	0.89	2.72	0.55	0.60	1.30	1.15	1.00	1.00	6	63.00	5.59	12	4.000
69.00	22	112.00	3.78	0.98	2.80	0.39	0.68	1.30	1.15	1.00	1.00	22	24.00	4.98	27	0.347
74.00	20	112.00	4.06	1.14	2.92	0.42	0.65	1.30	1.15	1.00	1.00	20	24.00	4.98	25	0.290

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.00	98.00	0.10	0.00	0.10	1.00	1.00	0.670	1.15	8	1.03	0.649	1.10	0.590	2.000
3.50	98.00	0.17	0.00	0.17	1.00	1.00	0.667	1.13	6	1.03	0.649	1.10	0.590	2.000
7.00	105.00	0.36	0.00	0.36	0.98	1.00	0.659	1.19	10	1.04	0.633	1.10	0.575	2.000
9.00	105.00	0.46	0.00	0.46	0.98	1.00	0.654	1.19	10	1.04	0.628	1.08	0.583	2.000
12.00	111.00	0.63	0.00	0.63	0.96	1.00	0.645	1.26	13	1.06	0.611	1.05	0.580	2.000
14.00	111.00	0.74	0.00	0.74	0.95	1.00	0.639	1.21	11	1.05	0.611	1.03	0.591	2.000
19.00	111.00	1.02	0.12	0.89	0.93	1.00	0.711	1.49	20	1.11	0.643	1.02	0.628	0.328
24.00	108.00	1.29	0.28	1.00	0.90	1.00	0.775	1.38	17	1.08	0.717	1.01	0.712	0.244
29.00	107.00	1.55	0.44	1.12	0.88	1.00	0.818	1.88	28	1.19	0.688	0.99	0.695	0.552
34.00	107.00	1.82	0.59	1.23	0.85	1.00	0.843	1.67	24	1.14	0.737	0.98	0.755	0.355
39.00	112.00	2.10	0.75	1.35	0.82	1.00	0.853	1.15	8	1.03	0.826	0.98	0.844	0.124
44.00	112.00	2.38	0.90	1.48	0.79	1.00	0.855	1.29	14	1.06	0.805	0.96	0.835	0.177
49.00	112.00	2.66	1.06	1.60	0.76	1.00	0.849	2.06	31	1.23	0.692	0.91	0.759	2.000
54.00	112.00	2.94	1.22	1.72	0.73	1.00	0.840	1.58	22	1.12	0.747	0.93	0.803	0.290
59.00	112.00	3.22	1.37	1.85	0.71	1.00	0.827	1.45	19	1.10	0.753	0.93	0.811	0.239
66.00	112.00	3.61	1.59	2.02	0.67	1.00	0.806	1.24	12	1.05	0.767	0.94	0.820	2.000
69.00	112.00	3.78	1.68	2.10	0.66	1.00	0.796	1.82	27	1.18	0.677	0.88	0.770	0.450
74.00	112.00	4.06	1.84	2.22	0.64	1.00	0.781	1.72	25	1.15	0.676	0.88	0.769	0.377

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted
FS: Calculated factor of safety against soil liquefaction

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
2.00	2.000	0.00	9.70	1.50	0.00
3.50	2.000	0.00	9.47	1.50	0.00
7.00	2.000	0.00	8.93	3.50	0.00
9.00	2.000	0.00	8.63	2.00	0.00
12.00	2.000	0.00	8.17	3.00	0.00
14.00	2.000	0.00	7.87	2.00	0.00
19.00	0.328	0.67	7.10	5.00	7.28
24.00	0.244	0.76	6.34	5.00	7.31
29.00	0.552	0.45	5.58	5.00	3.81
34.00	0.355	0.64	4.82	5.00	4.73
39.00	0.124	0.88	4.06	5.00	5.42
44.00	0.177	0.82	3.29	5.00	4.13
49.00	2.000	0.00	2.53	5.00	0.00
54.00	0.290	0.71	1.77	5.00	1.92
59.00	0.239	0.76	1.01	5.00	1.17
66.00	2.000	0.00	0.00	0.00	0.00
69.00	0.450	0.00	0.00	0.00	0.00
74.00	0.377	0.00	0.00	0.00	0.00

Overall potential I_L : 35.76

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
2.00	6	0.07	0.07	0.23	0.13	25789.58	0.05	0.16	10.08	13.35	2.00	6.406
3.50	4	0.11	0.11	0.28	0.13	18434.08	0.10	0.43	10.08	36.24	2.00	17.396
7.00	8	0.23	0.24	0.47	0.14	11908.57	0.02	0.05	10.08	4.45	4.00	4.275
9.00	7	0.30	0.31	0.53	0.14	10194.92	0.02	0.05	10.08	4.25	3.00	3.060
12.00	10	0.40	0.42	0.68	0.15	8470.78	0.01	0.02	10.08	1.72	2.00	0.824
14.00	11	0.47	0.49	0.70	0.15	7681.29	0.02	0.03	10.08	2.83	3.00	2.038

Cumulative settlemetns: 33.999

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
19.00	20	15.90	0.52	0.328	15.90	2.30	7.00	1.935	1.11
24.00	17	22.15	0.67	0.244	22.15	2.62	5.00	1.572	1.11
29.00	28	6.08	0.04	0.552	6.08	1.29	5.00	0.777	0.30
34.00	24	10.02	0.29	0.355	10.02	1.97	5.00	1.181	0.50
39.00	8	50.00	0.94	0.124	50.00	4.23	5.00	2.536	2.50
44.00	14	30.65	0.79	0.177	30.65	3.02	5.00	1.810	1.53
49.00	31	4.04	-0.16	2.000	0.00	0.00	5.00	0.000	0.00
54.00	22	12.67	0.41	0.290	12.67	2.13	5.00	1.275	0.63
59.00	19	17.78	0.57	0.239	17.78	2.40	5.00	1.441	0.89
66.00	12	0.00	0.00	2.000	0.00	0.00	3.00	0.000	0.00
69.00	27	6.92	0.11	0.450	6.92	1.53	3.00	0.549	0.21
74.00	25	8.88	0.23	0.377	8.88	1.90	3.00	0.683	0.27
							Cumulative settlements:	**13.760**	**9.05**

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

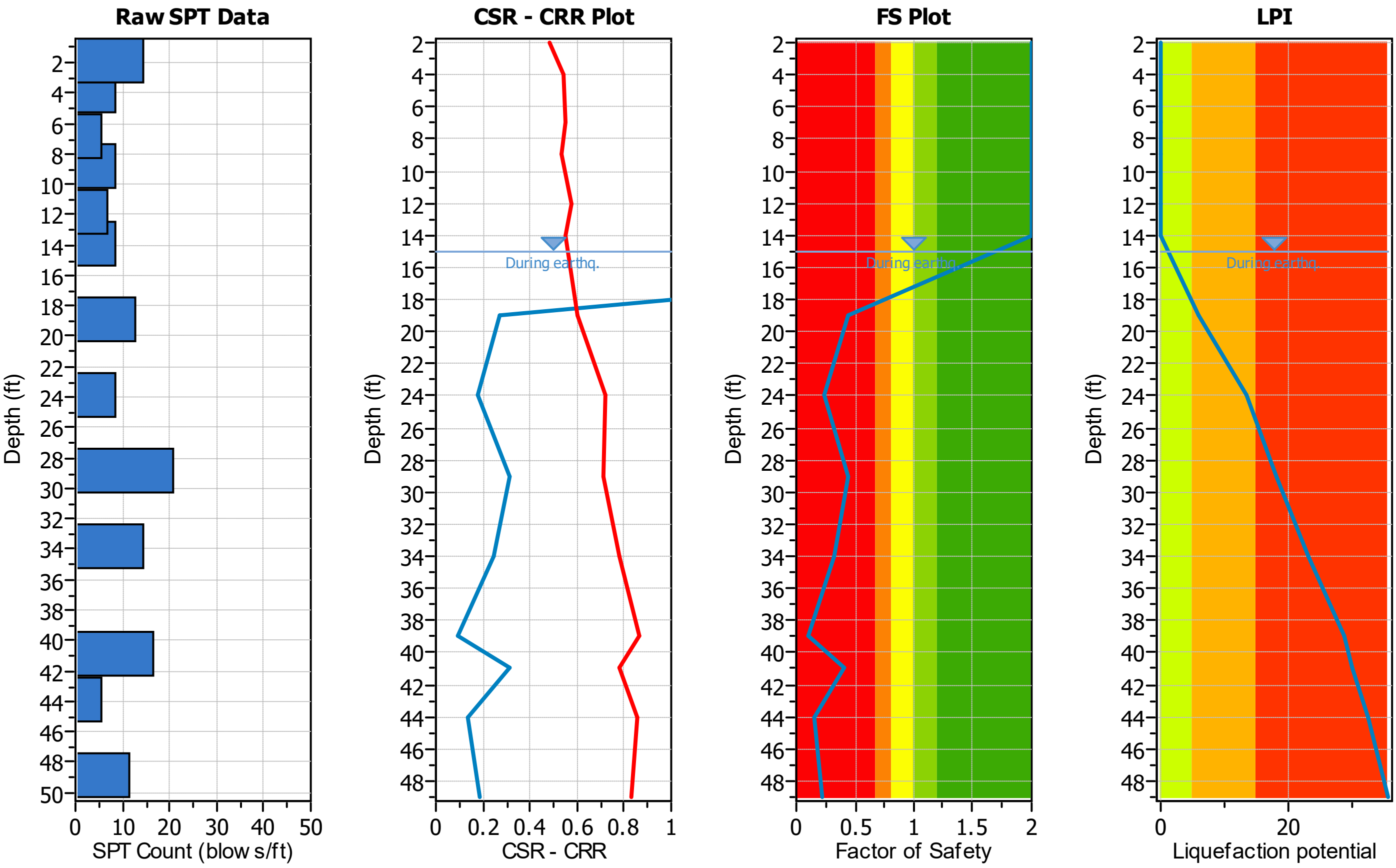
SPT BASED LIQUEFACTION ANALYSIS REPORT

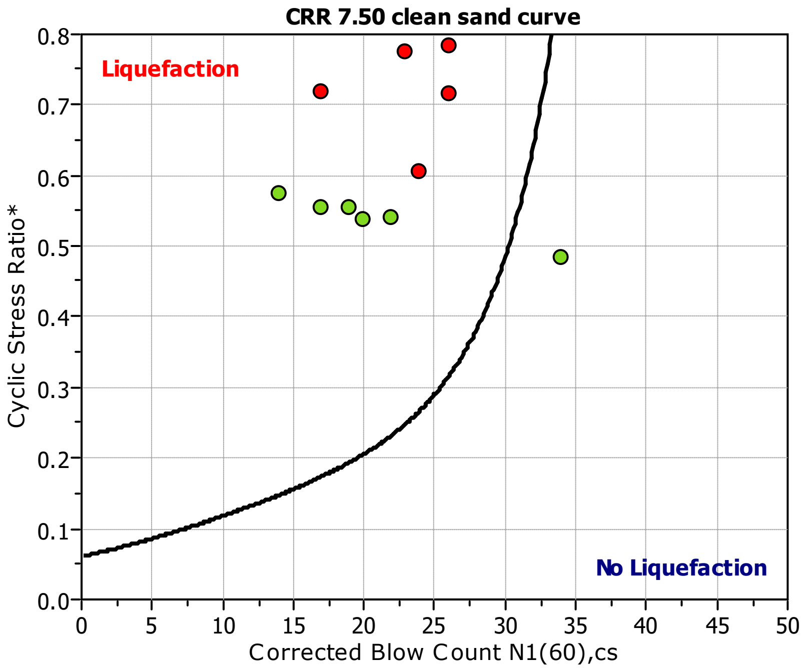
Project title : Moorpark Library **SPT Name: DH #2**

Location : High Street and Moorpark Avenue

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	37.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	15.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.90 ft
Borehole diameter:	200mm	Peak ground acceleration:	1.03 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.30		





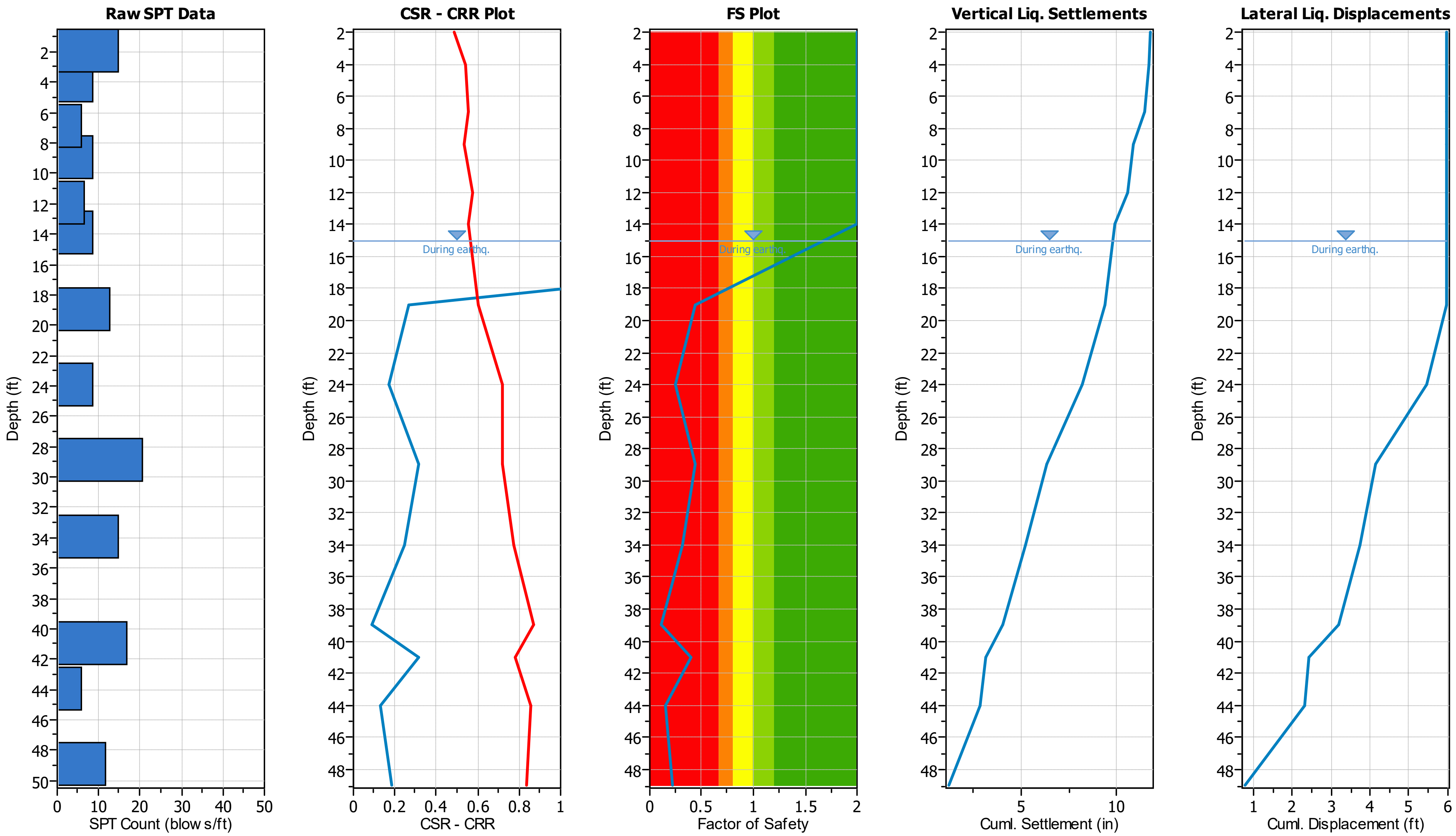
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
2.00	15	22.00	102.00	3.00	No
4.00	9	22.00	102.00	3.00	No
7.00	6	22.00	101.00	3.00	No
9.00	9	22.00	101.00	3.00	No
12.00	7	12.00	101.00	2.00	No
14.00	9	27.00	106.00	4.00	Yes
19.00	13	27.00	106.00	5.00	Yes
24.00	9	29.00	95.00	6.00	Yes
29.00	21	6.00	95.00	5.00	Yes
34.00	15	27.00	97.00	5.00	Yes
39.00	0	52.00	118.00	1.50	Yes
41.00	17	52.00	118.00	1.50	Yes
44.00	6	47.00	118.00	4.00	Yes
49.00	12	27.00	118.00	4.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
2.00	15	102.00	0.10	0.00	0.10	0.32	1.70	1.30	1.15	0.75	1.00	29	22.00	4.77	34	4.000
4.00	9	102.00	0.20	0.00	0.20	0.40	1.70	1.30	1.15	0.75	1.00	17	22.00	4.77	22	4.000
7.00	6	101.00	0.36	0.00	0.36	0.45	1.63	1.30	1.15	0.80	1.00	12	22.00	4.77	17	4.000
9.00	9	101.00	0.46	0.00	0.46	0.42	1.43	1.30	1.15	0.80	1.00	15	22.00	4.77	20	4.000
12.00	7	101.00	0.61	0.00	0.61	0.49	1.31	1.30	1.15	0.85	1.00	12	12.00	2.07	14	4.000
14.00	9	106.00	0.71	0.00	0.71	0.44	1.19	1.30	1.15	0.85	1.00	14	27.00	5.21	19	4.000
19.00	13	106.00	0.98	0.00	0.98	0.40	1.03	1.30	1.15	0.95	1.00	19	27.00	5.21	24	0.268
24.00	9	95.00	1.22	0.00	1.22	0.47	0.94	1.30	1.15	0.95	1.00	12	29.00	5.32	17	0.174
29.00	21	95.00	1.45	0.00	1.45	0.39	0.88	1.30	1.15	0.95	1.00	26	6.00	0.03	26	0.316
34.00	15	97.00	1.70	0.00	1.70	0.42	0.82	1.30	1.15	1.00	1.00	18	27.00	5.21	23	0.249
39.00	0	118.00	1.99	0.05	1.94	0.63	0.68	1.30	1.15	1.00	1.00	0	52.00	5.61	6	0.092
41.00	17	118.00	2.11	0.11	2.00	0.41	0.77	1.30	1.15	1.00	1.00	20	52.00	5.61	26	0.316
44.00	6	118.00	2.29	0.20	2.08	0.54	0.69	1.30	1.15	1.00	1.00	6	47.00	5.61	12	0.132
49.00	12	118.00	2.58	0.36	2.22	0.47	0.70	1.30	1.15	1.00	1.00	13	27.00	5.21	18	0.184

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.00	102.00	0.10	0.00	0.10	1.00	1.00	0.670	2.20	34	1.26	0.533	1.10	0.485	2.000
4.00	102.00	0.20	0.00	0.20	0.99	1.00	0.666	1.58	22	1.12	0.593	1.10	0.539	2.000
7.00	101.00	0.36	0.00	0.36	0.98	1.00	0.659	1.38	17	1.08	0.609	1.10	0.554	2.000
9.00	101.00	0.46	0.00	0.46	0.98	1.00	0.654	1.49	20	1.11	0.591	1.10	0.537	2.000
12.00	101.00	0.61	0.00	0.61	0.96	1.00	0.645	1.29	14	1.06	0.608	1.06	0.574	2.000
14.00	106.00	0.71	0.00	0.71	0.95	1.00	0.639	1.45	19	1.10	0.583	1.05	0.555	2.000
19.00	106.00	0.98	0.12	0.85	0.93	1.00	0.714	1.67	24	1.14	0.625	1.03	0.604	0.444
24.00	95.00	1.22	0.28	0.94	0.90	1.00	0.788	1.38	17	1.08	0.728	1.01	0.718	0.242
29.00	95.00	1.45	0.44	1.02	0.88	1.00	0.840	1.77	26	1.17	0.721	1.01	0.716	0.441
34.00	97.00	1.70	0.59	1.10	0.85	1.00	0.874	1.62	23	1.13	0.771	0.99	0.776	0.321
39.00	118.00	1.99	0.75	1.24	0.82	1.00	0.880	1.13	6	1.03	0.857	0.99	0.868	0.106
41.00	118.00	2.11	0.81	1.30	0.81	1.00	0.880	1.77	26	1.17	0.755	0.97	0.782	0.404
44.00	118.00	2.29	0.90	1.38	0.79	1.00	0.877	1.24	12	1.05	0.835	0.97	0.857	0.154
49.00	118.00	2.58	1.06	1.52	0.76	1.00	0.867	1.42	18	1.09	0.796	0.96	0.833	0.220

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted
FS: Calculated factor of safety against soil liquefaction

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
2.00	2.000	0.00	9.70	2.00	0.00
4.00	2.000	0.00	9.39	2.00	0.00
7.00	2.000	0.00	8.93	3.00	0.00
9.00	2.000	0.00	8.63	2.00	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
12.00	2.000	0.00	8.17	3.00	0.00
14.00	2.000	0.00	7.87	2.00	0.00
19.00	0.444	0.56	7.10	5.00	6.02
24.00	0.242	0.76	6.34	5.00	7.32
29.00	0.441	0.56	5.58	5.00	4.75
34.00	0.321	0.68	4.82	5.00	4.98
39.00	0.106	0.89	4.06	5.00	5.53
41.00	0.404	0.60	3.75	2.00	1.36
44.00	0.154	0.85	3.29	3.00	2.55
49.00	0.220	0.78	2.53	5.00	3.01

Overall potential I_L : 35.53

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
2.00	29	0.07	0.07	0.38	0.13	25177.92	0.00	0.00	10.08	0.09	3.00	0.067
4.00	17	0.14	0.14	0.46	0.13	16611.23	0.00	0.00	10.08	0.35	3.00	0.254
7.00	12	0.23	0.24	0.56	0.14	11903.54	0.01	0.01	10.08	0.78	3.00	0.560
9.00	15	0.30	0.31	0.67	0.14	10245.08	0.01	0.01	10.08	0.47	3.00	0.340
12.00	12	0.39	0.41	0.69	0.15	8626.56	0.01	0.02	10.08	1.36	2.00	0.651
14.00	14	0.46	0.48	0.82	0.15	7833.59	0.01	0.01	10.08	0.54	4.00	0.516

Cumulative settlemetns: 2.387

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
19.00	24	10.02	0.29	0.444	10.02	1.97	5.00	1.181	0.50
24.00	17	22.15	0.67	0.242	22.15	2.62	6.00	1.887	1.33
29.00	26	7.85	0.17	0.441	7.85	1.79	5.00	1.076	0.39
34.00	23	11.27	0.35	0.321	11.27	2.04	5.00	1.227	0.56
39.00	6	50.00	0.95	0.106	50.00	4.86	1.50	0.875	0.75
41.00	26	7.85	0.17	0.404	7.85	1.79	1.50	0.323	0.12
44.00	12	38.03	0.86	0.154	38.03	3.34	4.00	1.604	1.52

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
49.00	18	19.85	0.62	0.220	19.85	2.51	4.00	1.204	0.79
							Cumulative settlements:	**9.376**	**5.97**

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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